POAC 77
Proceedings
Vol. I

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The Honorary Chairman of the Fourth International Conference on Port and Ocean Engineering under Arctic Conditions was Professor P. Bruun of the Norwegian Institute of Technology, Trondheim, Norway.
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Several graduate students provided transportation for the delegates and helped out with the technical sessions, they were: Z. Abedin, P.V.T. Babu, I. Cobb, F.M. El-Hawary, H. El-Tahan, S. Gowda, A.K. Haldar, E. Kearley, R.R. Maclsaac, N.P. Riggs and J. Ransom. The help during registration and throughout the week of the Conference from the following people, mostly from the Office of the Registrar, is also gratefully acknowledged: G. Collins, D. Devine, M. Devine, S. Devine, A.M. O’Dea, Fried, C. Kielly, L. Mackey, M. O’Dea and W. Thistle.
PREFACE

This series of international conferences dealing with Ocean Engineering at high latitudes was started in 1971 at Trondheim, Norway. They have been held every two years since then, and in 1977 the meeting was in Canada for the first time, at Memorial University in St. John's, Newfoundland. Approximately 300 delegates from some twenty countries registered at the conference.

Eighty-four technical papers were presented in three concurrent sessions (A, B, and C). Six invited papers and three summary papers were given in plenary sessions. Each session considered a separate range of topics as follows: (A) platform design, port and ocean engineering and marine transportation; (B) sea ice, ice mechanics, ice forces and icebergs; (C) offshore development, environmental considerations, physical oceanography, remote sensing, data collection and processing.

All papers were available in preprint form at the beginning of the conference. This was possible through the excellent cooperation of authors in providing camera-ready text as requested by the technical programme chairman.

In the production of these proceedings, corrections submitted by the authors following the conference and minor typographical errors and the like detected by the editor have been incorporated. Although the changes in the preprint form of the paper have been quite small in the majority of cases, we take responsibility for some syntactical changes in one or two instances. The thanks of the organizing committee of POAC 77 are extended to the reviewers who assisted in the evaluation and selection of abstracts and the editing of the Proceedings.

The conference was organized with the general support (financial and otherwise) of the Faculty of Engineering and Applied Science, Memorial University of Newfoundland, The Centre for Cold Ocean Resources Engineering (C-CORE), Memorial University and NORDCO Limited. Financial support was also received from the National Research Council of Canada; The Government of Newfoundland and Labrador (Department of Industrial Development); the Offshore Services Association of St. John's, Newfoundland; and the Crosbie Group of Companies.

Many people worked long and hard to make POAC 77 a success. We should like to mention especially Dr. H.T. Renouf, the Executive Secretary, for his cheerful and competent handling of all the conference general arrangements; Mrs. H.M. Muggeridge, who was responsible for the cover design and layout of the POAC 77 material including these Proceedings; and Mrs. J. Henderson who handled with patience and thoroughness all the secretarial details including the final preparation of these Proceedings.

Finally, there were others - many volunteers - whose names may not be mentioned but who contributed much to the success of the conference. To every one of them, we extend our thanks.

D.B. Muggeridge

G.R. Peters

February, 1978
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ICE ENGINEERING FOR OFFSHORE PETROLEUM EXPLORATION IN CANADA

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ABSTRACT

Canada's remaining proven hydrocarbon reserves amount to about 2.5 billion* tonnes. By the mid-1980's, at present rates of consumption, shortages will occur unless imports are increased or indigenous supplies are developed. Undiscovered hydrocarbon resources probably amount to about 7 billion tonnes, mostly in the offshore frontier regions. These regions are covered by ice for most of the year.

The incentive to develop hydrocarbon resources which may exist beneath ice-covered waters, has led to significant advances in ice-engineering in Canada. During the last few years techniques have been developed for drilling year-round in the Beaufort Sea, drilling off-the-ice in the Arctic Islands, and drilling amongst the icebergs of the Labrador Sea.

This paper describes these operations, and the research and development in ice engineering which has made them possible.

Note  *  one billion = 1 000 000 000 (North American usage)
1.0 CANADA'S ENERGY NEEDS AND SUPPLIES

1.1 The International League

As a nation, we in Canada consume large quantities of energy. If we look at energy consumption per capita of various countries (Figure 1), Canada is second only to the United States in energy consumption. Furthermore, Canada consumes about twice as much energy per capita as various European nations with similar levels of Gross National Product. (Foley, 1976).

Regardless of the causes, it is sufficient to say that Canada's lifestyle and present prosperity are highly dependent on energy. Moreover, because we consume energy at about twice the rate as many of our trading partners, we are particularly vulnerable to increases in oil prices. Furthermore, Canada's degree of self-sufficiency in energy becomes a vital element in our balance of payments. Obviously, as world oil prices continue to increase it will be to Canada's advantage to be self-sufficient, and perhaps even to export. However, unless new supplies are brought on stream, Canada will soon be a net importer of hydrocarbons.

1.2 Demand and Supply

Up to about 1975 Canada was self-sufficient in oil. The imported oil supplying Eastern Canada was balanced by exports of Western Canadian crude to the United States. Since 1975 Canada has imported more oil than it has exported and this imbalance is growing.

Currently, export of natural gas more than offsets the oil deficit. However, by about 1980 Canada will become a net importer of hydrocarbons. Moreover, if oil and gas demand continues to increase at about 3% per year, and no major new discoveries are brought on stream, then by 1985 Canada will be importing the equivalent of about 140,000 tonnes/day of oil (one million barrels/day).

At a world oil price of $20 per barrel, such imports would add to Canada's balance of payments deficit by about $7 billion per year.

There are only two alternative solutions to this serious potential problem of large oil imports. Either we reduce consumption or we develop indigenous energy resources. Conservation is certainly desirable, but given the nature of man and our society, will be difficult to achieve on a voluntary basis.

The incentives for developing indigenous supplies are, in my opinion, fairly clear. First, we can avoid the cost of large oil imports (or at least offset their cost by the export of natural gas). Second, by developing indigenous supplies we help to reduce unemployment, and also develop special skills and knowledge which can ultimately be exported. Third, by using our own oil and gas, we help alleviate an international problem of predicted world-wide oil shortage by 1990.

1.3 Potential Supplies

Canada's Department of Energy Mines and Resources has recently compiled an inventory of oil and gas resources (EMR, 1977). The report tells us that
remaining hydrocarbon reserves in Canada amount to about 2.5 billion tonnes of oil equivalent (i.e. 18 billion barrels). Undiscovered resources, based on a 50% probability of existence, are estimated to amount to another 7 billion tonnes of oil equivalent (50 billion barrels). Most of these undiscovered resources lie in the remote areas of Canada called the frontier regions as can be seen from Figures 2 and 3. Furthermore, in all the frontier regions the largest potential occurs in the offshore areas, where the presence of ice makes exploitation both difficult and costly.

It will be seen from Figure 3 that each of the frontier areas contains significant potential hydrocarbon resources. Also Canadian consumption over the next ten years will largely exhaust the remaining reserves of Western Canada (not counting the oil sands). Yet if the frontier regions prove fruitful and can be developed economically, Canadian hydrocarbon supply can be assured for several decades.

The EMR report also refers to offshore 'inaccessible' areas but which are not included in the estimates of undiscovered resources. Much of this so-called inaccessible area lies off the E. Coast of Canada and in Baffin Bay where exists deep water and/or the presence of ice. Although this area was not included in the EMR estimates, most of it is already leased to the oil companies, and the technology for exploration and production is already being developed. As with all the frontier offshore regions, large pool sizes and high productivity wells will be needed to justify the very expensive technology needed for development. Possibly the so-called "inaccessible" areas may have a potential, three or four times as great as the 'accessible' E. Coast area. This could add about 3 billion tonnes of oil equivalent (22 billion barrels) to the undiscovered resources. However until drilling is undertaken in these deep water areas any estimate of potential is highly speculative. The above estimate is only included to indicate the incentive for developing the technology needed for exploitation of the deep ice covered regions off Canada's East Coast and in Baffin Bay.

To conclude this section on petroleum supplies, I would again like to emphasize that Canada consumes large amounts of petroleum. In the future, existing reserves will not meet demand. If indigenous supplies are not developed, then large balance of payments deficits will occur due to imported oil. Significant undiscovered resources are predicted to be present in the frontier regions of Canada, largely in offshore ice infested areas. It is this potential which has stimulated the rapid development of ice engineering in Canada during the last ten years. In this paper, I will describe the highlights of this development.

It will be convenient to divide the offshore frontier areas into the three regions listed below.

(1) The Beaufort Sea

(2) The Seas between the Arctic Islands

(3) Baffin Bay, Davis Strait and the Labrador Sea.

For each of these regions, I will give an overview of ice conditions, describe techniques for hydrocarbon exploration and production, and discuss
the 'ice engineering aspects' of these techniques.

First a brief review of conventional offshore petroleum operations, as practised in ice-free areas of the world, will be useful.

2.0 OFFSHORE PETROLEUM OPERATIONS

2.1 In Conventional Areas

In the last twenty years, offshore petroleum technology had advanced quickly. Exploration wells are now being drilled in 1000 m water depths and production platforms have been installed out to 260 m. Floating rigs are now available to drill in water depths up to 2000 m, and subsea production systems for these water depths are under development. There seem to be no technical reasons why drilling and production systems cannot be extended to the 4000 m water depth. However, costs are high, and large deposits of oil and high well productivities will be needed for such ventures to be worthwhile.

Today, the procedures for offshore petroleum operations in modest water depths (say about to 300 m) are well proven. Following seismic operations, exploratory 'wildcat' drilling penetrates potential hydrocarbon-bearing sediments. Exploratory drilling is usually conducted from floating vessels which are kept over the drill location either by mooring lines or by thrusters (dynamic positioning). In shallow water, bottom-founded mobile rigs such as jack-ups are often used. For ice infested waters none of these systems is suitable because the moving ice can easily push such rigs off location. Naturally in areas which are ice free in the summer, floating rigs can be, and indeed are being, used. But short and variable ice free periods diminish the efficiency of summer drilling.

For production operations, year-round surface access is desirable. In most current offshore operations this is achieved by installation of production platforms which are fixed to the sea bed. Their design is usually governed by wave forces but sometimes by earthquake criteria. Platforms are now quite common in water depths out to 200 m (North Sea and Gulf of Mexico) and a platform was recently installed in 260 m of water off California (Ocean Industry, 1977). Platforms using the guyed tower principle are predicted to be feasible out to about 600 m of water. In deep water depending on the area and sea conditions, floating vessels positioned over subsea production systems can be more attractive than fixed platforms. Also, these systems can be arranged so that oil is stored in the production vessel, and moved to market by shuttle tankers. For both fixed and floating platforms whether or not a pipeline is used to transport the hydrocarbons will depend largely on distance to shore and the market. Without a pipeline the movement of natural gas requires liquifaction.

In ice infested waters, bottom founded production platforms need to withstand the lateral forces generated by moving ice. Similarly, floating production vessels would need mooring systems or thrusters capable of maintaining the vessel on location against the moving ice. Much of the ice engineering I shall be describing in this paper has been developed to determine the forces generated by moving ice against fixed structures.
2.2 In Ice Covered Areas

Some precedents for petroleum operations in ice covered waters were established in the Cook Inlet of Alaska in the early 1960's (Visser, 1969). Here, ice can be present from November to May, can grow up to about one metre thick, and is subject to large movements caused by tidal currents. Exploration drilling activities were conducted in the ice-free period from floating rigs and jack-ups. Production platforms were designed to resist the lateral forces generated by the moving ice. A typical Cook Inlet production platform in about 20 m of water is shown (Figure 4). The four support legs are about 5 m in diameter and are piled into the sea bed. A Monopod type structure with a 9 m diameter leg has also been installed in the Inlet (Oil and Gas Journal, 1970). It is of interest to note that these structures were designed for an ice crushing stress of 2070 kPa (300 psi). Needless to say, no failures have occurred.

The pioneers of Cook Inlet have demonstrated one approach to offshore petroleum activities in ice covered waters; explore in the summer from conventional rigs, and produce from bottom-founded platforms designed to withstand the winter ice forces.

Depending on the ice conditions and the water depths, other approaches can be considered. For this discussion, it is convenient to divide petroleum activities into exploration and production. Again, I will emphasize that for exploration, all that is required is a stable platform for a limited period, whereas for production, year-round capability is almost mandatory.

Two types of ice cover, that is, landfast ice and pack ice will also be considered. The movement of landfast ice will be assumed to be limited to 3 m per day and less than 20 m per season. Pack ice is assumed to move several kilometres per day.

For exploratory drilling, the best approach for any area will depend on the ice conditions and the water depth, but there are basically three options.

1. Drill during the ice free period from a floating vessel.
2. Drill off the ice.
3. Drill from a bottom-founded platform or vessel capable of resisting the lateral ice forces.

The first option above will be the most attractive if, (a) the open water period is sufficiently long, (b) the water is too deep for cheap bottom-founded structures, and (c) the area is covered by pack ice in the winter. Such conditions apply to the region off Labrador where drilling has been conducted for a number of years. More recently, drillships have been operating during the short arctic summer in the Canadian Beaufort Sea.

Drilling off the ice using a more or less conventional land rig has been pioneered by Panarctic Oils operating on the seas of the Arctic Archipelago. A unique aspect of the ice in this area is the combination of fast ice and
deep water. The deep water mitigates the effects of any movements of the fast ice which may occur. This is because, as in floating drilling, the limit on lateral motion is governed by the angle of the drilling riser at the sea floor. Thus the allowable ice movement is a function of water depth. Drilling operations can normally continue up to a three degree riser angle which corresponds to a lateral ice motion in 200 m of water of 10 m. However, in 20 m of water the allowable lateral ice motion would only be one metre. It is for this reason that drilling off the landfast ice of the Beaufort Sea, which extends out only to about the 20 metre water depth, has not been attempted.

On the other hand, a large variety of different types of bottom-founded platforms have been considered for the Beaufort Sea. To date, drilling has been conducted from dredged islands in water up to 10 metres deep. As suggested by Figure 5, which categorizes drilling concepts as a function of ice conditions and water depth, mobile gravity-founded monopod and cone structures may also be viable for the Beaufort Sea in water depths up to 100 metres or so.

Islands, monopods and cones are designed to withstand winter ice conditions. Thus in addition to exploratory drilling, they can also be used for year-round production operations.

3.0 THE BEAUFORT SEA

3.1 Introduction

The continental shelf of the Beaufort Sea extends out to about the 200 m water depth, that is, about 150 km from shore in the MacKenzie Delta region. In 1973, the first offshore well was drilled from an artificial island in three metres of water. In 1976, summer drilling from floating vessels commenced at deeper locations.

When the exploration permits were leased, in the mid 1960's, few if any of the oil companies and government agencies knew very much about the physical environment of the Beaufort Sea, and even less about the technology which would be needed for petroleum exploitation. It says much for the initiative and enterprize of the organizations involved that during the last decade extensive research and engineering has been conducted to support arctic offshore operations.

3.2 The Arctic Petroleum Operators Association (APOA)

Following acquisition of leases it soon became obvious to many of the companies involved that the cost of studying ice conditions and conducting research in the Arctic would be extremely high. The extensive research needed, and the high costs involved, provided the incentive for an association which would act as a vehicle for cooperative research. Such an association was formed in 1969 and named, the Arctic Petroleum Operators Association (APOA).

Membership in APOA is open to all petroleum companies with interests in the Canadian Arctic; currently there are about 30 members. Research projects are operated by individual members. Participation in any project is volun
tary but participating companies share the cost of the project in return for
the data generated. Because participation in each research project is not
mandatory, it is necessary to implement a secrecy agreement which requires
that the results of each project not be published for a period of five years.

To date 136 cooperative research projects have been conducted by APOA for
a total value of about 21 million dollars. A list of these projects relating
specifically to 'ice engineering' is included in an appendix to this paper.

APOA has been very successful in promoting the development of ice engineer­
ing in Canada. Most of the research has been conducted by Canadian scient­
ists and engineers, who now form a nucleus of world expertise in this area
of technology.

3.3 Ice Conditions in the Beaufort Sea

Ice conditions vary from complete ice coverage to open water along a narrow
coastal strip during the brief polar summer. Ice drift is dominated by the
Beaufort Gyral which circulates in a clockwise direction. The open water
period naturally has a considerable influence on construction and logistics.
From our own experience over the last few years, plus Government ice maps
going back 15 years or more, we know that; in an average year, break-up will
be complete by mid-July near shore, clearing somewhat later further out;
freeze-up usually commences about mid-October giving a gross average open
water period of 90 days, near shore.

In bad years, as seems to occur every few years, break-up may be two or three
weeks late, and freeze-up two or three weeks early, cutting the season by
almost half.

Typical winter ice features are shown in Figure 6. Landfast ice extends out
to about twenty metres of water, but has a smooth surface only out to about
the five metre depth. Beyond that there are usually numerous first-year
ridges which are formed during the early winter before the ice becomes land­
fast. Many of these ridges are grounded, forming anchor points for the
landfast ice. The grounding action by these and the occasional multi-year
ridge probably causes the seabed scours which are numerous in the area
(Hnatiuk, Brown, 1977).

Freeze-up usually commences during October and by the end of October, new
ice has generally formed in the sheltered shallow bays, and at the edge of
the pack. By early November a thin ice cover usually extends over the whole
area, but except in the bays, it is easily moved and broken by the Autumn
storms. Gradually, however, a stable sheet of fast ice extends out from
shore. By mid-November fast ice extends out to about the 6 m water depth,
and is 30 - 60 cm thick.

During the dark months of November and December the relatively thin ice
which is not yet landfast, is continually broken and ridged by wind action.
It is during this period that most of the first year ridges in our area of
interest are formed. Many of these ridges ground on the sea bed and help
stabilize the ice so that by January a sheet of fast ice extends out to
about 20 m of water, the remains established at this position for the rest of
the winter.

3.4 Ice Ridges

The annual ice reaches a maximum thickness of about two metres by late April, but it is heavily covered by ice ridges. These ridges present obstacles for ice-surface transportation and can also cause significant loads if they move against bottom-founded structures.

Several studies of the geometry and strength of ridges have been conducted during the past few years (see APOA Projects in Appendix).

One study recently published (Hnatiuk, Kovacs and Mellor, 1976) contains several measured profiles of first-year and multi-year ridges. A typical profile of a first-year ridge is shown in Figure 7. This first-year ridge had a sail height of 5 m and was grounded in 19 m of water. First-year ridges are largely an accumulation of unconsolidated ice blocks and do not represent as severe a loading condition for marine structures as do multi-year ridges.

The aforementioned study examined a multi-year ridge with a keel depth of 14 m and a sail height of 6 m. The ridge was fully consolidated and the ice was of low salinity. Aerial surveys suggest that multi-year ridges of 20 m thickness are relatively common in the Beaufort Sea. Statistical techniques can be used to define the extreme ridge thickness at any particular location. However, in shallow water (say less than 20 m) the water depth will probably determine the ridge thickness for the design of marine structures in the area.

The design of offshore drilling and production platforms to withstand the forces imposed by moving multi-year ridges has been a significant ice engineering problem and will be discussed later under ice forces.

3.5 Ice Movement

The landfast ice is relatively stable but movements of several metres can occur due to deformation of the ice under wind stress or due to thermal strains. This amount of movement is sufficient to impose significant loads on fixed structures. Also, as discussed earlier, such movements are too large to allow off-the-ice drilling in the water depths covered by the landfast ice.

The movements shown in Table 1 were obtained using survey techniques in 1970 and show average movements of up to about two metres per day. Measurements since then using a wire reel system (Croasdale, 1973), indicate maximum movements of several metres per hour but never more than about 30 cm per hour in shallow sheltered locations. Total movements increase with water depth or distance from shore (Croasdale and Marcellus, 1976).

One important aspect of ice movement at sheltered shallow locations is that there can be stationary periods when the ice can freeze to the periphery of an island or structure. The adfreeze bond has to be broken once ice movement commences again and this can represent a severe loading condition.
TYPICAL LANDFAST ICE MOVEMENTS

BEAUFORT SEA - 1970

<table>
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<td>(Metres)</td>
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<td>MOVEMENT</td>
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17 | 33 | 19 | 28 | 5
8  | 20 | 29 | 39 | 19
9  | 14 | 6  | 27 | 2

TABLE 1

4.0 BEAUFORT SEA DRILLING AND PRODUCTION CONCEPTS

4.1 General

Drilling in the Beaufort Sea during the ice-free summer period has already been accomplished. In shallower water, exploratory drilling is possible from temporary bottom-founded platforms e.g. artificial islands. Regardless of the technique used for exploratory drilling, production will most likely be accomplished from fixed platforms.

We can generalize therefore, and note that an important concept for drilling and/or production in the Southern Beaufort Sea is the bottom-founded structure; the main criterion for design being an ability to withstand ice action. Thus the problem of ice forces on structures to be placed in the Beaufort Sea has become a major area of 'ice engineering' research in Canada.

4.2 The Monopod

Perhaps because of the precedent of Cook Inlet, one of the first configurations of bottom-founded structure considered for use in the Beaufort was the Monopod (Figure 8). The Monopod has the following advantages:

(i) Minimizes the frontal area of structure exposed to the moving ice action. This area does not change with the water depth in which the structure is placed.
(ii) Involves ice crushing failure, for which structures in subarctic regions have been successfully designed.
(iii) Ice freezing to the structure will not increase the ice forces by as much as might be expected with adfreeze on a sloping structure.
(iv) No chance of ice riding up onto the working surface of the platform.

At that time the precedents for structures in ice infested waters were all for the subarctic where the ice was thinner and weaker than in the arctic. Furthermore, ice design criteria were based on empirical relationships which could not easily be extrapolated to more severe ice conditions; certainly not
with the confidence we desired, considering the delicate environment at risk and the large investments anticipated for offshore arctic structures. Thus an extensive research program to study ice forces on Monopod type structures was commenced.

4.3 Ice Research: Monopod

The effective ice pressure or stress acting on a narrow vertical pier can be defined as:

\[ p = \frac{F}{td} \]  

(1)

where \( p \) is the effective ice stress  
\( F \) is the ice force  
\( d \) is the width of the pier  
\( t \) is the ice thickness 

The essence of the ice force problem on a narrow vertical pier is to be able to specify the appropriate value of \( p \) for the particular ice conditions expected.

Early work (Korzhavin, 1962) to address this problem (primarily for bridge piers) suggested the following empirical relationship for \( p \).

\[ p = Im\sigma \]  

(2)

where \( \sigma \) is the ice strength in compression  
\( k \) is a contact coefficient which equals 1.0 for perfect contact  
\( m \) is a shape factor which is close to 1.0 for circular piers  
\( I \) is an indentation factor which tends to 1.0 for a wide structure and is equal to 2.5 for narrow structures \((d/t = 1.0)\)

In Korzhavin's original equation a velocity term was included, but it is generally accepted that this can be omitted if the ice strength is specified for the appropriate velocity or strain rate.

The engineer is faced with many uncertainties when trying to apply equation (2). First, the ice strength in compression is difficult to specify. It is well known that the compressive strength of ice is a function of temperature, strain rate, crystal structure, salinity, degree of confinement, the size of the sample and the skill of the tester. Carefully tested laboratory samples give strengths as high as 7000 kPa (1000 psi) at temperatures of \(-10^\circ C\). If such a value is inserted into equation (2) with all other coefficients equal to unity then the effective ice pressure becomes.

\[ p = 7000 \text{ kPa (1000 psi)} \]

Yet it is well known that most structures installed in ice infested areas have been designed for much less than the above figure. For example, all the Canadian, U.S. and U.S.S.R. codes for design of vertical piers quote effective ice pressures no greater than 2800 kPa (400 psi) (Neill, 1976). To the best of my knowledge few if any structures designed to these codes have been destroyed by ice. However, the problem facing the arctic offshore engineer
is to establish a value for the effective ice pressure for the much thicker and perhaps stronger arctic ice.

As a first step to rationalize the problem, large in-situ ice crushing tests were conducted on Arctic coastal ice in 1970 (Croasdale, 1970, 1974). In these tests, effective ice pressures on circular piles were measured by jacking apart hinged piers of 0.75 and 1.5 m in diameter. (Figure 9)

Later tests were conducted with similar indentation equipment but which was designed to be portable so that a larger number of tests could be conducted per winter. In 1971 a series of tests were conducted on lake ice at varying strain rates using flat indenters 0.75 m in width (Croasdale, Morgenstern and Nuttall, 1976). Subsequent work (not yet published) was conducted with indenters up to 3.7 m wide which were actuated using hydraulic rams with a total load capacity of 17800 kN (2000 tons). (Figure 10)

In all these early tests, the objective was to investigate the validity of equation (2) and therefore small scale compressive strength tests were also conducted on ice taken from the test locations. Also the tests were conducted with a good fit between the indenter and the ice so that the contact coefficient could be assumed equal to unity. Various diameters and ice thicknesses were tested to investigate the indentation factor in equation (2). All tests were conducted on fresh ice. During the course of the work it was established that Arctic coastal ice near the river mouths is essentially fresh and has the same strength characteristics in terms of temperature and strain rate as other fresh S2 ice.

Composite plots of effective ice pressures obtained using the indentation equipment is given in Figure 11. No strong effects of width or strain rate can be seen in these data. Furthermore, the data suggests that a Monopod type structure should be designed for at least 4000 kPa effective ice pressure.

There are at least two possible mitigating factors leading to lower effective ice pressures, these are;

(1) A reduction in ice strength with volume (size effect)

(2) Contact factors less than 1.0

In addition, because of the visco-elastic nature of ice, if strain rates are very low then ice pressures will be low even with perfect contact conditions.

Space does not allow a full discussion of these topics but some evidence supporting these mitigating factors will be presented.

In recent work conducted at Laval University (Michel and Toussaint, 1976) Indentation tests were performed in the laboratory with ice up to 10 cm thick and indenters up to 20 cm. wide. Peak indentation pressures versus strain rates obtained from this work are shown in Figure 12. The tests conducted in the field with much wider indenters are also presented. The comparison suggests a size effect which if extrapolated would lead to even
lower effective ice pressures on full size structures. The Laval work also demonstrates the phenomena of 'continuous crushing'. As shown in Figure 13, the peak ice pressure obtained during the initial indentation with perfect contact is always much higher than the subsequent peaks. One can think of the subsequent peaks during the continuous crushing phase as exhibiting contact factors less than 1.0. In the example shown the contact factors would be about 0.4.

The range of 'contact factors' to be expected during the life of a structure is to some extent a function of the pattern of ice motion and in particular the strain rate. The problem can perhaps be better defined in statistical rather than deterministic terms. It is in this direction that much recent research is now aimed.

In noting that ice forces due to continuous crushing are of an oscillatory nature (as shown in Figure 13), the dynamic response of structures should be considered. This problem was first considered in relation to the Cook Inlet platforms (Peyton, 1968, Blenkarn, 1970). More recently, considerable work on this topic has been done at Memorial University (Reddy et al 1973, 1974, 1975, 1976).

A Monopod structure which was designed for use in the shallow Beaufort Sea has been described by Jazrawi and Davies, 1975. A side view of this structure is given in Figure 8. For design, the maximum sheet ice load was considered to be 100 MN (22.5 x 10^6 lbf) which corresponds to a nominal effective ice pressure of 5200 kPa (750 psi) with 2 m of ice. In water depths greater than about 10 metres it was realized that thick multi-year ridges might impose loads up to 300 MN (67 x 10^6 lbf). Furthermore, parallel research indicated that conical shaped structures could better resist such ice features. It was concluded that Monopod type structures have limited use in the Beaufort Sea and designs for conical structures were evolved.

4.4 Cone and Monocone Structures

Conical shaped light piers have been in use in the St. Lawrence for many years (Danys, 1975). The main reason for using a conical shaped structure is to reduce the ice force by causing bending rather than crushing failure. In the Beaufort Sea, this is of particular importance at locations where thick multi-year ridges can be present.

The Monocone structure is the subject of a separate paper at this conference (Jazrawi and Khanna, 1977). The Monocone is a Monopod with a conical collar at the ice line. For deeper water this configuration is expected to be less expensive than a simple conical structure. In addition, the Monocone has the advantage of being able to minimize the diameter at the waterline so that forces due to friction and ice adhesion can be kept low.

4.5 Ice Research: Cone

Early work on ice forces was conducted in a model basin in 1971 (Edwards and Croasdale, 1976). Ice modelling for flexural failure of ice had already been
successfully demonstrated in ship model testing (Edwards and Lewis, 1970).

Scaling criteria for successful modelling of ice interaction involving flexural failure have been discussed by many (e.g. Michel, 1970). It can easily be demonstrated that for valid testing, the ice strength and modulus should be reduced by the linear scaling ratio. Thus to conduct model tests at say one fiftieth scale requires ice weaker than real ice by the same factor. Weak ice can be achieved by high salinity, and model basins using this technique exist in the U.S.S.R., Europe and N. America. The use of a synthetic ice was pioneered in Britain (Crago, Dix and German, 1970). More recently a model basin has been built by Arctec Canada Ltd. where ice interaction can be studied using a wax based synthetic ice.

To provide confirmation of theoretical and small scale modelling, a large open air ice test basin was constructed in Calgary in 1973 (Robbins et al, 1975). This Canadian facility, which is the largest in the world, is 60 m long by 30 m wide (see Figure 14). Structures to be tested are mounted on a three point support system which is instrumented to measure horizontal and vertical forces. The ice is pulled into the structure by an ice boom and towing system with a capacity of 900 kN (100 tons). For the first few winters tests were conducted with a 45° steel conical structure in place (Figure 15). The structure was 3 metres wide at the water line, sheet ice and ridges up to 0.7 m thick were tested against it. Most of the tests conducted in the basin have been cooperative industry programs through APOA.

In addition to work aimed at studying the ice interaction process, field programs to measure the actual flexural strength of arctic ice have been conducted. Of particular interest has been the strength of multi-year ice in the Beaufort Sea. Multi-year ice was quarried from the ridge and tested as beams 2 m long by 0.3 m wide. (Figure 16).

The Ministry of Transport has also performed extensive studies in relation to conical light piers. Conical light piers have been instrumented (Danys, 1975), and theoretical studies have been conducted (e.g. Danys, Bercha and Carter, 1976).

4.6 Typical Ice Forces: Cone

For ice not frozen to the structure, the total horizontal force on a cone (R) can be separated into two basic components:

- The force necessary to break the ice. \( R_1 \)

- The force necessary to move the broken ice up and around the structure \( R_2 \)

It is suggested that:

\[ R_1 = f(\sigma_f, t^2, b, \beta, \mu) \]  \hspace{1cm} (3)

\[ R_2 = f(\rho_{\text{ice}}, d, t, \beta, \mu) \]  \hspace{1cm} (4)

where; \( \sigma_f \) is the flexural strength of the ice.
t is the thickness of the ice (or ridge).
b is the width of the ridge if applicable
\( \rho \) is the ice density
d is the cone diameter
\( \beta \) is the cone angle
\( \mu \) is the ice-to-structure friction.
E is the elastic modulus of the ice

It should be noted that for wide structures, the diameter will also probably enter the first component.

Calculation of the ice breaking component can be accomplished for a uniform ice sheet using Nevel's (1972) theory for floating ice sheets.

The significant effects of both friction and cone angle have been well documented by Danys, Bercha and Carter (1976). Some results from their paper are reproduced here as Figure 17. It will be seen that for a 45° cone if the friction increases from 0.1 to 0.4 the ice force approximately doubles. Also, depending on the friction, when the cone angle becomes steeper than about 45° (from the horizontal) the ice force starts to increase significantly.

Recently published data (Edwards and Croasdale, 1976) were obtained from model tests on a 45° cone. A correlation for lateral ice force was derived from the tests, namely;

\[
R = 1.6 \sigma_f t^2 + 6 \rho g D t^2 \quad (5)
\]

The first term can be considered as the force necessary to fail the ice in bending, the second term is the force necessary to push the broken ice up and around the cone. Implicit in both terms is the friction coefficient which was about 0.1 in these tests.

Consider a typical example for the Beaufort Sea with ice 2 m thick and a cone 25 m diameter at the water line, assume also that the ice has a flexural strength of 700 kPa (100 psi).

Then \( R_1 = 4480 \) kN
\[
R_2 = 5650 \text{ kN}
\]

In other words the ice breaking and ice clearing components are about equal.

Ridge loads on a cone can be calculated approximately using published theories for beams on elastic foundations (Hetenyi, 1946, Croasdale, 1975). As illustrated in Figure 18, for a ridge 15 m thick by 30 m wide, a ridge load of about 116,000 kN can be generated. This is considerably more than the 10,000 kN force due to 2 m thick sheet ice. But less than the force against
a Monopod structure which could be as high as 300,000 kN.

The ridge loads quoted above are only approximate and are applicable to long ridges ignoring the attached sheet ice which might be present. Furthermore, recent studies (Ralston, 1977) have indicated that shorter ridges could impose greater loads.

A cone is only a good shape for ice breaking as long as the ice slides over its surface. If the ice remains stationary long enough for it to freeze to the structure, an adfreeze bond will develop. When ice movement starts again, this adfreeze bond will have to be broken before ice can continue to fail in bending on the structure.

Obviously the adfreeze force is a function of the area of contact and the bond strength between ice and steel (or concrete). Little data has been published on ice bond strengths. But if we assume a nominal 700 kPa, a conical structure 30 m diameter at the ice line and an ice sheet 2 m thick frozen around it; the adfreeze force works out to be greater than 180,000 kN.

Obviously to reduce adfreeze forces, the cone diameter should be minimized. The use of heat, or anti-friction coatings currently being considered for ice breakers (Makinen, 1975), are other possible ways of reducing these potentially high forces to manageable values.

4.7 Artificial Islands

To date as many as sixteen islands have been built in the Beaufort Sea for exploratory drilling. Most of the islands have been constructed during the summer using dredges. Some have been built in winter by trucking gravel over the ice. Islands have been successfully built out to the 13 m isobath; islands in deeper water are considered feasible. Typical island profiles are shown in Figure 19. Islands generally need to have a diameter of about 100 m at the working surface and usually have freeboards of around 5 to 6 m. Ice and wave action on islands have been discussed in a recent paper (Croasdale and Marcellus, 1977).

4.8 Ice Action on Islands

For a wide structure, such as an island, the problem of ice clearing tends to dominate the ice interaction pattern. In our experience in the Beaufort Sea, exposed islands subject to extensive movements of thin ice in the Fall collect large areas of ice rubble around them (Figure 20).

A typical sequence of ice action on the beach of an exposed island is shown in Figure 21. Initial movements of thin ice fail at low loads in bending. The ice is too thin to ride-up the island beach so rubble piles are formed.

The mechanics of rubble pile formation are uncertain. In some cases, the rubble forms a ramp up which the oncoming ice sheet advances and fails in bending. At other times the ice penetrates the rubble but again is probably failed in bending by differential buoyancy and weight forces as suggested by
Parmenter and Coon (1972) for ice ridge formation. The rubble often grounds, and once a certain height is reached, grows seawards. In the early winter, the rubble height rarely exceeds 7 m (Kry, 1977). Ice forces at this time might be predicted with the model for pressure ridge formation mentioned above, to be less than 100 kPa. In any case, the rubble probably protects the island from most of these forces.

Because of the rubble resistance to lateral forces the active zone remains on the outside of the rubble which consolidates and freezes into a rigid ice annulus around the island (Figure 22).

Eventually the ice becomes too thick to fail in bending and begins to fail in crushing at much higher ice stresses. However, by now the refrozen rubble is competent enough to maintain the active zone at its outer boundary; and at the same time, transmit the ice forces to the frozen island surface.

For a particular ice stress (p) in the active zone, the ice force (F) generated on the island rubble-pile combination as shown in Figure 22 is given by:

\[ F = pWt \]  \hspace{1cm} (6)

where \( W \) is the width of the rubble in the direction of ice movement, \( t \) is the ice thickness.

If \( R \) is the sliding resistance of the rubble then the force on the island is given by:

\[ Q = pWt - R \]  \hspace{1cm} (7)

Simple estimates of rubble sliding resistance based on typical friction angles indicate that usually the force on the island is increased by the presence of the refrozen rubble around it (Croasdale and Marcellus, 1976). This is because the larger width of the active zone increases the lateral force due to ice thrust more than is gained from the sliding resistance of the rubble. (Kry, 1977).

The ice crushing stress in the active zone is difficult to predict. It is likely that due to the effect of width (size), the average stress in the active zone is much lower than would be felt by a narrow structure such as a Monopod. Research into this problem is continuing.

4.10 **Ice Ride-up**

The potential problem of ice ride-up has been described by Bruun and Johannesson (1971) with respect to Baltic experiences; examples of ice encroachment in Canada were reported by Tsang (1975).

To date, no significant problems of ice ride-up onto islands in the Beaufort Sea have been experienced. Observations have suggested that when large ice movements occur in the Fall, the ice is too thin to ride up and forms the extensive rubble piles described earlier in the paper. On the other hand,
during summer break-up, when the ice is thicker, it appears to be so deteriorated that it disintegrates on the beach, without ride-up (Garratt, Kry 1977).

Despite these favourable experiences so far, it is recognized that at more exposed locations, where for instance, polar ice may invade, the potential for an ice ride-up exists. Current research is aimed at better understanding of the mechanisms which limit ice ride-up and makes use of theory, model testing and tests with real ice in the large ice test facility in Calgary.

The ice properties which are thought to affect the ride-up problem are shown in Figure 23. Ice strength dominates the potential ice push, whereas friction between the ice and the beach has a large effect on resistance to ride-up.

4.11 In-Situ Ice Stress Measurements Around Islands

Because of uncertainties in predicting the extreme ice forces which might act on islands, instrumentation has been developed to give real time indications of both ice stresses (Metge et al 1975) and island stability (Hayley and Sangster, 1974).

The arrangement of the ice pressure sensor is shown in Figure 24. The sensor consists of a three plate capacitor about 0.6 cm. thick, 1.2 m wide and 1.8 m long. The metal plates are separated by elastomer buttons, under pressure the capacitance of the sensor changes. The sensors are installed in a slot cut in the ice and allowed to freeze in.

The ideal stress sensor should be invisible to the ice, that is, it should have the same stiffness and thermal expansion coefficient as ice. This is impossible because the modulus of ice changes with strain rate, and temperature. As a compromise, the sensors have been designed to have a smaller modulus than ice to give adequate sensitivity. But they are thin and wide, to make them less sensitive to changes in ice modulus. The relevant equation for an inclusion in an elastic material has been given by Coutinho (1949).

\[
\sigma = \frac{E_1}{D} \left(1 + \frac{h}{D} \right) \left(\frac{\sigma_1}{E_1} - \frac{h}{D} \right) + \frac{h}{D}
\]

In this equation, \(\sigma\) is the stress in the ice, \(\sigma_1\) is the average stress felt by the sensor. \(E\) and \(E_1\) are the elastic moduli of the ice and sensor respectively, \(h\) is the sensor thickness and \(D\) is the sensor width. Equation 8 shows that if \(\frac{h}{D} \ll 1\), then \(\sigma\) is not sensitive to \(E_1\) even if \(\frac{E_1}{E} < 1\).
5.0 THE ARCTIC ISLANDS

5.1 Introduction

The seas between the High Arctic Islands of Canada are covered by ice most of the year. Open water periods are too short to allow conventional drillships to enter the area to conduct exploratory drilling. On the other hand, the ice cover for most of the year is landfast. Taking advantage of this unique characteristic, ice engineering techniques have been developed by Panarctic Oils Ltd., and their consultants, to allow off-the-ice drilling. (Baudais, Watts and Masterson, 1976). The first well was drilled off-the-ice in 1974 in 130 m of water, 13 km from shore. Since then, wells have been drilled in water depths up to 300 m.

The offshore drilling system is shown in Figure 25. To support a drilling rig weighing 766 tonnes (845 tons) the natural ice sheet is thickened to 5 m by flooding.

5.2 Ice Platform Design and Construction

Ice platforms have been designed and monitored during construction by Fenco Consultants Ltd. of Calgary. Several design criteria have been established for construction and operation (Hood, Strain and Baudais, 1975). Ice platforms are treated as elastic plates on elastic foundations to calculate stresses and deflections. Ice platform thickness is determined to satisfy two conditions. First, the maximum long term deflection is limited to avoid flooding of the ice surface. Second, the maximum tensile stress in the ice is limited to 350 kPa (50 psi).

Experience has shown that the deflection criterion can be satisfied by using a reduced elastic modulus of 175,000 kPa (25,000 psi) in the equation for deflection. Calculations by Frederking and Gold (1975), indicate that this approach limits deflections to the primary stage of creep. A typical ice platform thickness is 5 m.

Flooding usually commences in November using electric submersible pumps. Typical build-up rates are 10 cm. per day (Masterson, Baudais and Wasilewski, 1975). Temperatures and salinity are extensively monitored across the pad. Temperatures in the built-up ice are maintained at -5°C or less by controlled flooding. Ice strengths are also monitored periodically using an in-situ bore-hole jack (Kivisild, Iyer, 1976).

5.3 Ice Movement

A critical factor in the success of ice platform drilling is the amount of ice movement during the drilling period. Confidence in the concept of ice platform drilling was supported by several years ice movement measurements. Both survey and acoustic techniques have been used. Typical ice movements have been less than 3 m during the drilling period from February to April (Baudais et al, 1976).

5.4 Production Systems

For near-shore wells an arrangement using subsea wellheads with flowlines
to shore is being developed (Panarctic Oils, 1976). It is planned to install such a system 1.5 km off shore Drake Point on the Sabine Peninsula of Melville Island. The well will be completed for gas production and a two month production test will be carried out in 1978.

6.0 THE EAST COAST

6.1 General

This paper would not be complete without some discussion of techniques being developed for petroleum operations off Canada's East Coast.

Average ice-free periods off the East Coast vary between 365 days per year in the south to about 100 days in the Davis Strait. These ice-free periods are sufficient to allow the use of more-or-less conventional floating drilling equipment. The big difference is in the need to cope with icebergs.

6.2 Icebergs

It is well known that the icebergs carried down the East Coast by the Labrador current originate mostly from the glaciers of West Greenland. It is estimated that up to 40,000 bergs per year can calve from West Greenland, although a more typical figure is about 15,000 per year (Murray, 1968). As the bergs drift south they melt and ground so that only about 400 per year reach 48°N latitude. Icebergs are influenced more by currents than by winds. The average drift rate off Labrador being about 14 km/day (8 nautical miles/day). The duration of the journey from Greenland to the Grand Banks is between one and three years. The average mass of icebergs decreases from North to South. In the Davis Strait icebergs are reported to have an average mass of about 1.5 million tonnes, and about 0.2 million tonnes on the Grand Banks (Smith, 1931). The seasonal aspect of iceberg calving plus the variability of sea ice cover, leads to seasonal variations in iceberg flux. The number of bergs passing any latitude is at a maximum in May or June (Nordco, 1975). Moreover, the best time for summer drilling from the point of view of zero sea ice and moderate seas is in the period May to September off Labrador, and July to October in the Davis Strait. Obviously during these months any drilling operation has to cope with potential iceberg collisions with the drillship.

6.3 Iceberg Handling

Techniques for iceberg avoidance and handling have been successfully developed by EastCan and their consultants Marex, and by Memorial University. Each drillship has an experienced iceberg observer who identifies the presence of icebergs near the drillship by radar and tracks their movement. Drift models based on previous observations and local currents are used to identify those bergs which may be heading towards the ship. The radius for initial alert is usually about 9 km. If a berg gets within about 3 km of the ship a towing operation is initiated. Various towing systems have been developed (Ainslie and Duval, 1973), all make use of suitably equipped supply ships. It has been suggested that bergs up to about 20 million tonnes can be handled. For larger bergs the drillship may have to leave location
if a collision is imminent. All drillships now being operated in iceberg areas do not rely on mooring lines, but are dynamically positioned by thrusters. Such an arrangement is ideal for rapid move-off. In an emergency, the drill pipe can be sheared at the sea bed, the blow-out preventers closed and the marine riser released from the subsea wellhead in a matter of minutes. In reality, several hours will be available between an untowable berg entering the alert radius and a potential collision. During this period, drilling can be stopped and a more orderly abandonment procedure can be implemented (leading to release of the riser from the wellhead if it is necessary for the ship to move). In most years at most locations the probability of having to avoid a collision by moving the drillship is estimated to be less than one percent.

6.4 Possible Production Systems

Production operations off the East Coast will be more difficult than exploratory drilling. This is because for economic operation year-round surface access is desirable, even if subsea production systems are utilized. Year-round surface access is made difficult by sea ice, icebergs and rough seas. Most of the continental shelf is covered by water greater than 300 metres deep. Thus, even if they could be designed against iceberg collision, fixed platforms are not suitable for most of the area. Subsea production systems being developed for other parts of the world will be needed.

It is envisaged that these systems will be linked to a massive floating caisson structure at the surface which will store the produced hydrocarbons for removal by shuttle tanker to market (See Figure 26). Caisson systems are presently being considered which could stay on location for most of the year. Thrusters would provide enough resisting force to combat moving ice, waves, winds and currents. Ice breaking work boats would be assigned around the caisson to break up large ice features and tow away icebergs. In the event of an iceberg which is too large for towing, threatening the caisson, a quick disconnect arrangement on the production riser with automatically closing valves would enable the caisson to avoid the berg by moving away using its thrusters. This sophisticated concept appears feasible, but it will require large reservoirs of hydrocarbons and high well productivity for it to be economical.

7.0 CONCLUDING REMARKS

Canada will be facing potential oil and gas shortages by the mid-1980's. Yet, Canada could avoid high oil import bills by developing indigenous supplies. However, much of Canada's potential oil and gas reserves lie offshore covered by ice infested water. These facts have provided the incentive for rapid development of ice engineering in Canada.

In the space of a few years, techniques have been developed for drilling year-round in the Beaufort Sea, drilling off-the-ice in the Arctic Islands, and drilling amongst the icebergs of the Labrador Sea.

Given the opportunity to earn a fair return on investment, the Canadian Oil
Industry is now in a strong position to exploit the potential hydrocarbon resources beneath Canada's icy seas.

8.0 ACKNOWLEDGEMENTS

In assembling a review paper such as this, space often dictates the breadth of coverage that can be included. The author is well aware that certain aspects of arctic offshore technology have been omitted. In particular apologies are offered for not including any material on ice breaking vessels and ice cutting technology -- two areas which promise to contribute significantly to future arctic offshore development.

The author thanks Imperial Oil Ltd. for permission to publish this paper. Any opinions expressed in the paper are those of the author.

The author also wishes to thank the many colleagues who have conducted the research and developed the technology described in this paper.

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ENERGY CONSUMPTION AND GNP - SELECTED NATIONS 1972

FIG. 1

MAP SHOWING MAJOR HYDROCARBON REGIONS

FIG. 2

BILLION BARRES OF OIL EQUIVALENT

V. CANADA (REMAINING RESERVES) UNDISCOVERED

MACKENZIE - BEAUFORT

ARCTIC ISLANDS

E. COAST "ACCESSIBLE" E. COAST "INACCESSIBLE"

APPROX. WATER 10 YEAR OIL AND GAS CONSUMPTION

FIG. 3

FIG. 4

PACK ICE

FAST ICE

OPEN WATER

TYPICAL DRILLING CONCEPTS FOR ICE INFESTED WATERS

FIG. 5
LAND FAST ICE (TO 20m BY MARCH) — MOBILE POLAR PACK — SHEAR ZONE

TYPICAL WINTER ICE CONDITIONS, BEAUFORT SEA

FIG. 6

PROFILE OF ARCTIC FIRST YEAR RIDGE

FIG. 7

PRINCIPLE OF OPERATION: NUTCRACKER TESTERS

FIG. 9

CONCRETE MONOPOD

FIG. 8

SCHEMATIC OF LAKE ICE TESTER

FIG. 10
INDENTATION STRENGTH (p) VERSUS STRAIN RATE (t) (FIELD TESTS)

Fig. 11

FIELD TESTS
ICE THICKNESS 100-150CM

LABORATORY TESTS
(ICE THICKNESS 5-100CM)

INDENTATION STRENGTH (p) VERSUS STRAIN RATE (t)

Fig. 12

INDENTATION PRESSURE VERSUS DISPLACEMENT

Fig. 13

PLAN VIEW OF ICE TEST BASIN

Fig. 14

Fig. 15
FIG. 16

FIRST CRACK

H_{f1} = 32000 \text{ kN}

HINGE CRACKS

\[ 2H_2 = 186,000 \text{ kN} \]

MONOPOD CRUSHING

\[ H = 300,000 \text{ kN} \]

TYPICAL RIDGE LOADS

FIG. 18

TOTAL HORIZONTAL FORCE FOR 120 cm ICE SHEET.

(BERCHA et al., 1976)

FIG. 17

FLEXURAL STRENGTH - TOUGH MATERIAL

\[ \mu = 0.3 \text{ - solid lines} \]

\[ \mu = 0.4 \text{ - dashed lines} \]

\[ \mu = 0.5 \text{ - dash-dotted lines} \]

\[ \mu = 0.6 \text{ - dotted lines} \]

\[ \mu = 0.7 \text{ - dash-dot-dotted lines} \]

\[ \mu = 0.8 \text{ - dash-dot-dot-dotted lines} \]

\[ \mu = 0.9 \text{ - dash-dot-dot-dot-dotted lines} \]

INERTIA MOMENT - \( A = \frac{a}{2} \)

FIG. 19

TYPICAL SACRIFICIAL BEACH ISLAND

TYPICAL SAND-BAG RETAINED ISLAND
SEQUENCE OF ICE ACTION ON ISLAND BEACH
(EXPOSED 1St AND)

ICE ACTION ON ISLAND AND ANNULUS OF REFROZEN ICE RUBBLE

FIG. 20

FIG. 21

FIG. 22

INTERPRETIVE MODEL FOR ICE RIDE UP

FIG. 23

ICE STRENGTH (σ)
ICE MODULUS (E)
ICE THICKNESS (h)
BUOYANCY (ρ)

HEIGHT OF ICE UP BEACH (z)
ICE THICKNESS (h)
FRICITION (μ)
ANGLE OF BEACH (θ)
IN SITU ICE STRESS SENSOR

FIG. 24

HECLA ICE DRILLING PLATFORM

FIG. 25

POSSIBLE DEEPWATER PRODUCTION SYSTEM FOR EAST COAST CANADA

FIG. 26
### APPENDIX

List of A.P.O.A. Ice Studies

<table>
<thead>
<tr>
<th>A.P.O.A Project No.</th>
<th>Description</th>
<th>Total Cost</th>
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<td>1</td>
<td>&quot;Nutcracker&quot; Large Scale Ice Strength Tests</td>
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<td>2</td>
<td>Beaufort Sea Ice Survey</td>
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<td>9</td>
<td>Large Scale Ice Strength Test Phase II</td>
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<td>All Season Exploratory Drilling System - 0 to 200 Feet of Water</td>
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<td>Seasonal Drilling from a Barge</td>
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<td>Summer Ice Reconnaissance Beaufort Sea</td>
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<td>Theoretical Analysis of Ice Failure</td>
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<td>Pressure Ridge and Ice Island Scouring</td>
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<td>Model Test Simulating Ice on Drilling Barge</td>
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<td>Aerial Reconnaissance of Ice - Beaufort Sea</td>
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<td>Landfast Ice Movement - Mackenzie Delta</td>
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MODEL SIMULATION OF NEAR SHORE ICE DRIFT, DEFORMATION AND THICKNESS

W.D. Hibler III*
USA Cold Regions Research and Engineering Laboratory, Hanover, N.H. 03755

ABSTRACT
Simulation results for sea ice drift, deformation and ice thickness variations in the Arctic Basin are presented using a dynamic-thermodynamic model which treats the ice as a rigid plastic continuum. Using available observed atmospheric and oceanic forcing data, numerical model simulations are made over a four year long period employing one day time steps in a finite difference code with a resolution of 125km. Drift, deformation, stress and ice thickness time series from the simulation results in the near shore region off the Alaskan and Canadian North slope are reported and briefly examined in light of available observations.

INTRODUCTION
Modeling of near shore pack ice motion, stress, and thickness characteristics on the Geophysical scale is a subject of relevance to commercial and Naval offshore operations. From work to date it has become apparent that to adequately model this region requires consideration of both the nature of the ice interaction with itself and variations in the ice thickness characteristics. The essential idea is as the ice builds up against the shore the ice interaction with itself becomes stronger and hence the motion and stress will change even though the wind and water driving forces may be the same.

For considering the interaction of the ice with itself the use of plastic rheologies (e.g. Pritchard 1975) has seemed particularly appropriate since such rheologies can naturally produce shear zone effects and are easily coupled to ice thickness variations (Thorndike, et. al. 1975; Rothrock, 1975). However, present simulations (Pritchard, 1977) using plastic type rheologies have generally been localized in space and are not suitable for longer term simulations including seasonal effects because of short time step limitations and formulation in moving Lagrangian grids. Such longer term simulations are, however, valuable since they supply estimated records of the build up and decay of the near shore ice cover and of onshore ice stress variations.

In this paper a plastic type simulation model usable in longer term simulations is presented and used to carry out a 4 year simulation at one day time steps. The

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model is formulated in a manner similar to geophysical fluid dynamics ocean and atmospheric circulation models (Bryan, et. al., 1975; Manabe, et. al., 1975) and makes use of finite difference techniques developed in such work. The rheology used in his model is rigid plastic for normal deformation rates and viscous for very small deformation rates. This viscous-plastic approach allows the essential features of plastic interaction to be modeled without time step limitations.

For the ice thickness characteristics continuity equations for both compactness and ice thickness are used which include advection and thermodynamic source and sink terms. The plastic strength (P) of the ice is taken to depend linearly on the ice thickness (h) in any grid cell and exponentially on the compactness (A) according to

\[ P = P^* h \exp[C(A-1)] \]

where \( P^* \) and \( C \) are empirical constants. This formulation makes the strength strongly dependent upon the amount of thin ice (characterized by the compactness A) while also allowing the ice to strengthen under ridging which will increase the thickness h. The coupled system of equations are formulated in a fixed Eulerian grid (with a small biharmonic diffusion term added to the ice thickness equations to control non linear instabilities) and consequently can be integrated over unlimited time intervals.

**ESSENTIAL FEATURES OF MODEL**

To give some feeling for the nature of this work the essential aspects of the model are briefly outlined. More details on both the model and the numerical scheme are presently being written up and will be submitted for publication in the near future.

**A. Viscous-Plastic Constitutive Law**

At each grid cell the following general constitutive law is used

\[ \sigma_{ij} = 2\eta(\dot{\epsilon}_{ij}, P) \dot{\epsilon}_{ij} + [\zeta(\dot{\epsilon}_{ij}, P) - \eta(\dot{\epsilon}_{ij}, P)]\delta_{ij} \]

where \( \sigma_{ij} \) is the two dimensional stress tensor, \( \dot{\epsilon}_{ij} \) the strain rate tensor, \( P \) a pressure (or ice strength term depending on one's point of view) and \( \eta \) and \( \zeta \) are nonlinear bulk and shear viscosities. The dependence of \( \eta \) and \( \zeta \) on \( \dot{\epsilon}_{ij} \) and \( P \) is normally taken so that the stress state lies on an elliptical yield curve of ellipticity 2 going through the origin (explicit functional equations are given in Hibler (1977)). However, for very small strain rates the viscosities become arbitrarily large. To avoid this the viscosities are taken to be the minimum of the plastic value and some large limiting value. In practice the limiting value is reached only for deformation rates smaller than \( \sim 10^{-4} \) days\(^{-1}\).

**B. Ice Thickness Evolution**

Denoting partial derivatives \( \partial / \partial t \) by \( \partial_t \) etc., the following continuity equations are used

\[ \partial_t h = - \partial_x (uh) - \partial_y (vh) + G(h,A) + \text{Diffusion} \]

\[ \partial_t A = - \partial_x (uA) - \partial_y (vA) + \frac{f(0)}{h_0} (1-A) H[f(0)] + \frac{A}{2h} G(h,A) H[-G(h,A)] + \text{Diffusion} \]

34
where \( u \) and \( v \) are the x and y components of ice velocity, \( h \) is the average thickness of ice over a given grid cell, \( A \) the compactness, \( f(h) \) the growth rate of ice of thickness \( h \), and \( H(x) \) a step function (0 for \( x<0 \), 1 otherwise). Small biharmonic and harmonic diffusion terms have been added for stability over long time integrations. In the equation for \( A \), \( h \) is chosen to be a small demarcation thickness between thin and thick ice (0.5m has been used here); and the second source term is derived by assuming the ice is uniformly distributed between 0 and \( 2h/A \) in thickness.

C. Thickness - Dynamics Coupling

The plastic strength \( P \) is taken to be a function of compactness and thickness:

\[
P = P^* h \exp[C(A-1)]
\]

In the simulations performed here \( P^* \) is taken to be \( 10^4 \) N/m and \( C=20 \). As mentioned before, this makes the strength strongly dependent upon the amount of thin ice while also allowing the ice to strengthen under ridging. \( P^* \) is the key constant and is chosen to give reasonable velocity values.

D. Governing Momentum Equations

In the momentum equations a balance between Coriolis force, wind stress, water stress, internal ice stress, Ocean tilt and acceleration is used. The components of the momentum equations are given in Hibler and Tucker (1977) with the exception of the acceleration term which is taken to be the local ice mass times the acceleration. (For this particular calculation momentum advection terms are neglected, e.g. \( \frac{\partial u}{\partial x} \)), consequently we will not repeat the equations here. Linear wind and water stress terms are used although the numerical scheme will handle more complex nonlinear cases if desired. The particular wind and water drag parameters are the same as used in Hibler and Tucker (1977) with the exception of the turning angles which were taken to be \( 20^\circ \).

E. Numerical Scheme

The momentum and continuity equations are numerically solved as an initial value problem. For conservation purposes a staggered spatial grid configuration similar to that used in Ocean models (Bryan et. al., 1975) is used. For solution of the momentum equations point relaxation techniques are used together with a semi-implicit procedure to handle the nonlinear terms. The continuity equations are integrated by a splitting technique.

SIMULATIONS

To obtain seasonally varying simulation results essentially independent of initial conditions the model was applied to the Arctic Basin and integrated for four years at a one day time steps and a resolution of 125km. The grid is shown in Figure 1. The speckled cells and circled grid points near the North slope denote locations used for near shore time series results. To obtain a natural outflow condition, the ice thickness (for estimating strength) was taken to be zero in the shaded area between Spitzbergen and Greenland. For input wind data to drive the simulation, observed surface pressure data (obtained from NCAR) over the time period May 1962 to May 1963, was used to construct 8 day averaged Geostrophic winds. This wind record was then interpolated into itself in May to create a one year periodic
record. For calculation of Geostrophic currents Coachman's (Coachman and Agaard, 1974) dynamic topography values were used. (Equations for calculating geostrophic currents and wind are given in Hibler and Tucker 1977). Thermodynamic growth rates (no geographical variation) as a function of thickness and time were taken from Thorndike et. al. (1975). After several years of integration using this data the model approaches a cyclic equilibrium with thickness and velocity characteristics taking on similar values on corresponding days of successive years. All the results reported on here were taken from the 4th year of integration.

**Basin Wide Results**

To give some feeling for the overall results, Figure 2 shows ice velocities and contour plots of compactness and ice thickness on Julian day 225. The salient characteristics are the thin and low compactness ice off the North slope and the heavy ice thickness off the Canadian Archipelago which is in general agreement with observed ridging characteristics (Fig. 3), obtained from laser profilometer data (Hibler et. al., 1974b) and from recent submarine sonar data (Wadhams, 1977). The low compactness is indicative of good summer navigation which is in agreement with Barnett's (1977) observational study for this year. Note that these variations are caused primarily by advection since the growth rate curves do not vary geographically.

**Near Shore Ice Characteristics Time Series**

Geographical variations -- To examine geographical effects, ice characteristics of the three grid cells (see Fig. 1) nearest the coast were examined. The locations approximately correspond to the Mackenzie delta, Prudhoe Bay and Barrow. In Figure 4 are shown the year long time series of ice thickness, compactness, compressive stress ($\sigma_{xx}$), x and y velocities (x axis perpendicular to North slope, right handed coordinate system) and the bulk viscosity which in a plastic model is a function of the deformation rate as well the ice strength. The dominant temporal variation in all these locations is a seasonal variation with the ice becoming thinner and less compact in summer with a commensurate decrease in stress and viscosity. However, superimposed are fluctuations on scales of 10 days with the greatest effects occurring in the stresses and the viscosities. There is also a general correlation between the time series at a given location with compactness changes correlating positively with ice thickness, viscosity and negatively with stresses and velocity. This type of behavior is intuitively expected since when the ice pulls away (positive x velocity) the shearing motion (y velocity) increases, the onshore stresses are reduced, and the ice thickness and compactness decrease. This general behavior was observed from remote sensing imagery by Hibler et. al., (1974a) and by Reimnitz et. al., (1977).

With regard to geographical variations, they are not pronounced. The Barrow region does appear to have slightly greater changes in thickness. Also the Barrow and Prudhoe stress records are somewhat different as can be seen from Fig. 5, where more detailed plots of the Barrow and Prudhoe stress and thickness time series are given. However, it is emphasized that a plane boundary is a very crude representation of the true North slope boundary geography. Calculations with a step in the boundary extending from Prudhoe to past Barrow have been performed and do yield a greater build up of ice around Prudhoe. However, such a large discontinuous step is a fairly drastic boundary irregularity. More detailed simulations at 62.5km allowing reasonable boundary resolution are currently planned and should yield better insight into geographical effects.
Variation Versus Distance from Shore -- To examine the effects of going away from the shore, the ice characteristics were plotted for the three speckled grid cells off Barrow and the results are shown in Fig. 6. The results show simulated deformation events similar to those observed in LANDSAT studies by Hibler et al. (1974a). The basic characteristic is that as the ice moves out, the near shore layer develops a very low viscosity layer so that the pack can slip along the shear zone in a manner similar to a cohesive wheel. This type of behavior is particularly apparent in the spring when a large deformation event took place. Although, partially due to the weakening of the near shore ice strength due to lower compactnesses, this effect is strongly dependent on the plastic nature of the flow. Simulations without ice thickness variations, for example, yielded a lower viscosity in the near shore region under similar forcing fields due to the greater deformation rates here.

Another effect, which would certainly be expected intuitively, is that the fluctuations in compactness are much more reduced further from shore, indicating a shore lead developing and then closing.

DISCUSSION

In this paper a plastic type simulation model, usable both in long and short term simulations, has been presented and used to carry out a year long model study of the evolution of ice thickness and deformation characteristics in the Arctic Basin. Typical near shore deformation events obtained from the simulation follow the pattern observed from remote sensing imagery with the formation of a thin low viscosity layer near shore as the ice moves away allowing the pack ice in the gyre to move more cohesively. During the summer a low compactness region formed off the North slope in general agreement with observed ice edge characteristics for the particular year simulated. Stresses were found to fluctuate appreciably even without large changes in compactness and all near shore parameters showed a significant seasonal variation. Basin wide variation of ice thickness were found to be in general agreement with observed ridging variations.

Detailed comparisons between simulations and observations at a finer resolution are certainly needed to assess the usefulness of this type of model in aiding the understanding of near shore ice processes. Better resolution to approximate geographical boundaries is especially needed. The results obtained here, however, indicate that the near shore simulated behavior has characteristics similar to observations and suggests that such simulations should have utility in applied near some operations.

ACKNOWLEDGMENTS

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REFERENCES


Figure 1. Fixed square mesh grid used for numerical calculations. The speckled areas denote location of time series plots of nearshore ice characteristics.
Figure 2. Ice velocity field (a), thickness contours (b), and compactness contours (c), on day 225 of 4th year of simulation. A velocity vector one grid space long represents 0.05 m/s.
Figure 3. Ridging intensity (a parameter proportional to the volume of deformed ice) observations and contours obtained from laser profilometer measurements taken during Feb. 1969. (Hibler et. al. 1974).
Figure 4. Ice thickness, compactness, onshore compressive stress, bulk viscosity, and x and y ice velocity time series for the nearest shore grid cells located close to the McKenzie delta, Prudhoe Bay and Barrow.
Figure 5. Ice thickness and compressive stress curves for Barrow (solid) and Prudhoe (dashed) grid cells.
Figure 6. Ice thickness, compactness, and bulk viscosity, as a function of distance from shore off Barrow. The solid, dashed and dotted lines represent results respectively from the near shore grid cell, a cell 125km from shore and a cell 250km out from shore.
NEW DEVELOPMENTS IN MODELING ICE PROBLEMS

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Abstract

Ice modeling has started almost simultaneously in Leningrad and in Hamburg in the 1950s. The discovery of oil, gas, and other resources in the Arctic has pushed the ice technology considerably forward. New ice model basins were established in Columbia, Md., USA, Helsinki, Finland, and Hamburg, W-Germany. In present model technology saline ice is mainly being used whereby the problem of reducing strength and elasticity by the same scale is still hampering the results. The Hamburg Model Basin (HSVA) only recently succeeded in developing a method by which the ratio $E/\sigma$ can be kept within the range of sea ice in nature.

The occurrence of several different equations for predicting the icebreaker resistance is due to the lack of theoretical approaches, incomplete physical understanding of the icebreaking phenomena, disregard of fundamental similarity laws in case of model tests, and inadequate testing methods in full scale. In case of full scale tests, the obtained data scatter so widely that they easily can be used to prove different prediction equations. There are several reasons for the scattering of full scale data.

Recent model test results on the icebreaker resistance obtained by HSVA seem to prove the theoretically developed prediction of Milano (1975), including the "Milano hump".

In order to elucidate the fundamental relationship of parameters involved in icebreaking, a research project has been carried out at HSVA in which the different components of resistance have been analysed separately by model tests on a simple body such as an inclined plane plate. The results have shown that the resistance of an icebreaking body is governed by simple physical relationships. As a consequence of these investigations, a new icebreaking concept has been developed, which breaks the ice cover mainly by shear and provides an icefree channel.
Introduction

Physical modeling of icebreaking phenomena has become increasingly important due to the discoveries of oil, gas and other resources in arctic regions.

In addition to the existing ice model basins in Leningrad, Helsinki, Hamburg and Columbia/Maryland at least six more facilities are presently planned or already under construction: two in Canada, two in USA, one in the Soviet Union and one in Japan. The model material mainly being in use is saline ice. By varying the brine content within the ice, almost any required strength reduction can be obtained. The disadvantage of saline ice is, however, that by reducing the strength over a certain limit the elasticity decreases far more than required by scale. The ratio $E/\sigma$ of saline model ice drops to 200 to 500 while in nature this ratio is 2000 to 5000.

As an alternative to saline model ice synthetic ice has been developed by Michel in Canada and by Tryde in Denmark. The problem with synthetic ice is, however, that the friction coefficient is too high compared with real ice.

The task of the Hamburg Model Basin was therefore directed to develop a model ice on the basis of saline ice but with improved similarity of its mechanical properties.

Similarity Considerations

In order to obtain similarity between open water model tests and corresponding full scale conditions, where inertia forces, viscous forces, and gravity forces are acting, the Froude number

$$\frac{v}{\sqrt{gL}} = Fr$$  \hspace{1cm} (1)

and the Reynolds number

$$\frac{v \cdot L}{\nu} = Re$$  \hspace{1cm} (2)

have to be equal in both cases. Since this is not possible, the compromise of modelling according to Froude's law is common practice. The error is kept small by carrying out the model tests at Re-numbers as high as possible and by correcting the results by specially established correction methods.

In case of model testing in ice in addition to the aforementioned gravity, viscous and inertia forces, friction and breaking phenomena as well as the two phase flow (water and ice) have to be considered.

The rupture of ice depends mainly on its material properties. It has been shown by several authors (Michel, 1970; Poznyak, 1968, and Atkins, 1975, for example) that strength and elasticity of the ice have to be scaled by the same scale factor $\lambda$. This follows from the requirement that in addition to the Froude number also the Cauchy number

$$\frac{v^2 \rho}{E} = Ch$$  \hspace{1cm} (3)
has to be equal in model and full scale tests.

Recently Atkins (1975) has introduced a new non-dimensional group

\[ I = \text{Ch}^2 \cdot \sqrt{\frac{EL}{R}} \]  

This so called "Ice number" is based on the square of the Cauchy number and includes also the fracture toughness R, i.e. the resistance to the propagation of cracks.

Last, but not least, the friction coefficient has to be equal in model and full scale.

The frictional resistance results not only from the interaction between the hull of the considered body and ice but also from the friction between ice and ice. Therefore it is not allowed to use a model material with a high friction coefficient (synthetic ice) and to try to compensate its effect by a low friction coating of the model.

The question now is, which of the similarity laws must and which can be fulfilled in ice model tests?

Due to the importance of the effect of gravity, inertia, and elasticity of the ice, Froude's and Cauchy's similarity requirements must be fulfilled. Common practice, however, was and is even today at some laboratories, to scale just by Froude's law. This means, the geometry values as well as the ice strength are reduced by \( \lambda \) and the velocity by \( \sqrt{\lambda} \). The elasticity of the ice is not thought to be an important parameter (Edwards, 1975). How erroneous and serious the disregard of Cauchy's law is, will be shown later in this paper.

The Reynolds law can be disregarded in case of icebreaking tests, if the viscous forces are small compared with the sum of all the other components. In fact this is the case, especially, if the scale factor is not too large (\( \lambda < 20 \)).

To satisfy Froude's and Cauchy's model laws simultaneously by using saline ice is indeed difficult, because the proper reduction of strength results in a far too small elasticity, if the scale factor \( \lambda \) is too large.

In 1975 the Hamburg Model Basin (HSVA) has carried out a research program to investigate the relationship between strength, elasticity, salinity and temperature (Schwarz, 1975). The result of this investigation is plotted in Fig. 1 as the normalized strength versus the normalized elasticity for two different temperatures (\( 0^\circ \) and \( -15^\circ \) C). It proves the validity of the relationship between strength, elasticity and brine volume as it was theoretically developed by Weeks and Assur (1967) and it also shows that cold saline ice with a large brine content has a far too small elasticity compared with its flexural strength (E/\( \sigma \) < 500). By warming the model ice just before testing, the ratio E/\( \sigma \) can be increased significantly and by choosing model scales not larger than \( \lambda = 20 \), the E/\( \sigma \) ratio can be kept in the range of sea ice in nature. This result has been utilized successfully by HSVA for modeling icebreaking phenomena.
The effect of the E/σ ratio on the breaking energy is demonstrated in Fig. 2 in which the force-deflection curve of the new HSVA-model ice is being plotted as well as the equivalent curve of a high saline, cold model ice. The comparison between both curves shows that the energy required to break the HSVA-model ice is much smaller than that required to break the low elastic ice with its high residual plasticity.

The importance of the "Ice Number" on the model test results has not yet been evaluated, because of the difficulty to measure the fracture toughness of ice, especially of saline ice with its numerous void spaces and brine cells. It has been claimed by Atkins, however, that the fracture toughness of ice with a too low E/σ ratio is smaller than reduced by scale. In order to compensate for this lack of fracture toughness it could be advisable to scale the strength by less than λ, the exact value, however, is difficult to establish.

In order to avoid scale effects in both the frictional and the breaking forces, the size of the ice crystals should be reduced by the scale factor λ. This is commonly being obtained by seeding ice nuclei at the beginning of the ice formation.

Implications with Icebreaker Testing

In 1968 Kasteljan (1968) has presented a semiempirical equation for the calculation of the icebreaker resistance, in which the total resistance is subdivided into three different portions: the breaking portion R_b, the gravity portion R_g and the inertia portion R_i. Based on this procedure of separating the different physical processes, several scientists in the western hemisphere (Lewis and Edwards (1970), Edwards and Lewis (1972), Vance (1975), Kotras et al (1973), Mäkinen and Ross (1973), Enkvist (1972)) have presented a number of equations during the last seven years. They are all based on results from model and full scale tests and have been derived by regression analysis. Some of these equations are presented in Table 1, which shows the different parameters considered in the three portions and the different exponents of these parameters. The friction effects as well as the shape of the icebreaker are supposed to be taken into account by the constants C_0, C_1 and C_2.

Since the physical phenomena of the icebreaking process are not yet well understood and since questionable data from model tests as well as from full scale measurements, are being used, the regression analysis is not the proper tool to establish resistance equations for icebreakers. By this approach different resistance equations are obtained whenever a new icebreaker is being tested. Mass production of equations has caused confusion in the icebreaker technology.

The wide range of scattering is demonstrated also in Fig. 3, in which a variety of total resistances is plotted versus ship speed for the U.S. Coast Guard icebreaker MACKINAW as predicted by various formulations.

One of these resistance curves has been established theoretically by Milano (1975) on the basis of an energy approach. After Milano, the total energy required for breaking an ice cover in the continuous mode consists of the following parts:

\[ E_1 = \text{Energy lost for removing the broken ice pieces} \]
\[ E_2 = \text{Energy lost due to ship impact with the unbroken ice sheet} \]
\[ E_3 = \text{Energy lost due to the ship climbing onto the unbroken ice sheet} \]
<table>
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<th>Total Resistance of Icebreakers</th>
<th>Fracture Portion</th>
<th>Gravity Portion</th>
<th>Inertia Portion</th>
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<td>$R = C_0 \sigma_f Bh + C_1 \varphi_E g Bh^2 + C_2 B^{1.65} h v$</td>
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<td>Lewis and Edwards</td>
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<td>$R = C_0 \sigma_f Bh + C_1 \varphi g Bh^2 + C_2 \varphi_E^{L h^{0.65} \theta^{0.35}} v^2$</td>
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<td>$R = C_0 \sigma_f Bh + C_1 \varphi g Bh , T + C_2 \varphi_E Bh , v^2$</td>
<td>Wärtsilä</td>
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Table 1  Several Formulas for Predicting the Resistance of an Icebreaking Ship
\[ E_4 = \text{Energy lost due to the ship falling after ice failure} \]

\[ E_5 = \text{Energy lost due to submersion and moving away of ice pieces.} \]

Since the pitching motion of the ship and thereby the energy losses \( E_3 \) and \( E_4 \) decrease with increasing speed, the total resistance curve of Milano gets the unconventional shape, which implies that the resistance does not increase significantly between 2 and 8 knots. In case of thicker ice, the resistance at 8 knots can even be smaller than at 5 knots. Although Milano claims that the correlation between his prediction and full scale measurements of the icebreaking resistance of the MACKINAW is good, some reservations can be made, as for example in respect to friction which is estimated too low.

In contrast to other ice laboratories the Hamburg Model Basin, using its newly developed model ice with the \( E/\sigma \) - ratio being in the range of real sea ice, has obtained repeatedly results which more or less show the same tendency as Milano's theoretical predictions (see Fig. 4). The "Milano hump" cannot appear in the results of model tests, in which the ratio \( E/\sigma \) is too small due to improper scaling of \( E \). In this case the model ice deforms too plastically. When the icebreaker model tries to climb up onto the unbroken ice cover the low modulus ice gives way so that the pitching motion is far less than in full scale. Therefore the resistance is reduced by the energy required for the vertical motion of the ship. This occurs especially at low speeds.

Another important question is related to model test results which show an extremely small breaking resistance compared with the submergence and inertia portions. Following these results, it is widely accepted that the breaking portion is only 5 to 20 % of the total resistance, which has stimulated Mäkinen and Ross (1973) to neglect the breaking term completely in their equation for predicting the resistance of icebreaking ships.

Full scale measurements on the other hand show a much higher (30 to 70 %, depending on the velocity and ship size) and more intelligible contribution of the breaking portion to the total resistance. This difference between model tests and full scale measurements is given in Table 2 for three different icebreaking ships (after Vance, 1975) which indicates that the model technique omitting the proper scaling of the elasticity does not produce correct results.

<table>
<thead>
<tr>
<th>Velocity</th>
<th>Resistance Portion</th>
<th>Mackinaw Mod.</th>
<th>FS</th>
<th>Moskva Mod.</th>
<th>FS</th>
<th>Finncarrier Mod.</th>
<th>FS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Ft/sec</td>
<td>Fracture</td>
<td>43</td>
<td>71</td>
<td>6</td>
<td>68</td>
<td>32</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td>Gravity</td>
<td>57</td>
<td>29</td>
<td>94</td>
<td>32</td>
<td>68</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td>Inertia</td>
<td>00</td>
<td>00</td>
<td>00</td>
<td>00</td>
<td>00</td>
<td>00</td>
</tr>
<tr>
<td>7 Ft/sec</td>
<td>Fracture</td>
<td>34</td>
<td>44</td>
<td>5</td>
<td>57</td>
<td>24</td>
<td>43</td>
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<tr>
<td></td>
<td>Gravity</td>
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<td>77</td>
<td>27</td>
<td>53</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>Inertia</td>
<td>20</td>
<td>22</td>
<td>18</td>
<td>16</td>
<td>23</td>
<td>18</td>
</tr>
<tr>
<td>21 Ft/sec</td>
<td>Fracture</td>
<td>13</td>
<td>20</td>
<td>2</td>
<td>25</td>
<td>8</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>Gravity</td>
<td>18</td>
<td>9</td>
<td>31</td>
<td>12</td>
<td>19</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Inertia</td>
<td>69</td>
<td>71</td>
<td>67</td>
<td>63</td>
<td>73</td>
<td>67</td>
</tr>
</tbody>
</table>

Table 2: Resistance distribution in model and full scale of three different icebreakers (after Vance, 1975)
Agenda for Table 2:

<table>
<thead>
<tr>
<th>Ice thickness</th>
<th>= 2 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mod.</td>
<td>= Model</td>
</tr>
<tr>
<td>FS</td>
<td>= Full Scale.</td>
</tr>
</tbody>
</table>

In order to prove this statement the Hamburg Model Basin has carried out icebreaker model tests, in which the ice strength was reduced in several steps from 140 to 20 kN/m². The results, plotted in Fig. 5 as the "effective strength" R/h² versus the flexural strength \( \sigma_f \) obtained by cantilever beam tests, provide the following information:

When the bending strength of the model ice is being reduced from 140 to 50 kN/m², the "effective strength" decreases. However, if the flexural strength is reduced further, the "resistance" increases again. This can be explained by the unnatural increase of the plasticity or the unproportional decrease of the elasticity of the model ice which is associated by an increasing energy required to break the ice.

If these experimental data are then being used for the evaluation of the constants \( C_0', C_1 \) and \( C_2 \) in the equations given in Table 1, the regression analysis shows a much too small² effect of the ice strength on the total icebreaking resistance \( (C_0' \ll C_1, C_2) \). The results presented in Fig. 5 are also in agreement with the basic investigations on the relationship between strength, elasticity and brine volume mentioned earlier in this paper (Fig. 1). It can be shown that the elasticity starts to deviate from proper scaling, when the strength is reduced below values of about 50 kN/m².

Since most of the resistance equations for icebreakers are based on model tests which were carried out in saline ice at strength levels below \( \sigma_f = 30 \) kN/m², Young's Modulus has not been scaled properly. This has caused (1) a too high total icebreaker resistance, (2) a much too small breaking portion of the total resistance, and (3) incorrect resistance equations.

As a consequence of these results the Hamburg Model Basin is not reducing the ice strength below \( \sigma_f = 50 \) kN/m². In case a smaller scale (\( \lambda > 20 \)) is unavoidable the percentage of the breaking resistance is established and than corrected by the ratio of the required strength to the actual strength.

Possibility of Comparison between Model Test Results and Full Scale Measurements

It has been claimed by most of the groups who have established formulae for the prediction of the icebreaker resistance that their model test results are in good agreement with full scale data. This statement surprises, if we consider the big differences in the icebreaker resistance as demonstrated in Fig. 3. In spite of this objection, none of the research groups claims something wrong because the full scale test results scatter over such a wide range that they can be used to verify even controversial predictions of the icebreaker resistance.

The scattering of full scale results is caused by various reasons: First of all, it is difficult to measure ice thickness, ice strength and ship velocity as accurately as necessary. In most cases the environmental conditions such as ice thickness and snow cover vary from place to place so much that only a continuous record could provide adequate information. Secondly, the testing methods are different from investigator to investigator. This causes different results for nominally the same condition. Realizing the lack of conformance in testing methods, the IAHR (International Association of Hydraulic Research) and also the ITTC (International Towing Tank Conference) have formed working groups who are preparing recommendations.
on uniform methods for testing ice and how to execute full scale tests with icebreakers (see Appendix B, Proc. 3rd. Inter. Symp. on Ice Problems, Hanover, N.H. 1975, Frankenstein editor).

Icebreaker designers are waiting urgently for the trial tests of the U.S. Coast Guard icebreaker POLAR STAR, which is equipped with most sophisticated instruments, as for example with an impulse radar device which allows the continuous recording of the ice thickness. Unfortunately this icebreaker returned from its first voyage without data, but with a damaged propulsion plant.

Another reason why full scale data scatter so widely is due to the problem of converting the propulsion measured in full scale trials into resistance measured in model tests. In order to convert propulsion into resistance one needs information on the thrust deduction fraction and the wake fraction.

These values are specially difficult to establish due to the existence of broken ice around the propeller, which seems to cause a wider scatter of the propeller thrust.

Most of the remarks on the reason for the scattering of full scale data are based on the experience which the author has gained from a research expedition with an icebreaking offshore supply ship to the arctic ice of Spitzbergen in April 1977. The results of these trial tests will be published later this year.

Basic Investigations on the Resistance Components in Icebreaking

As a reaction on the variety of controversial results, opinions and formulae describing the icebreaking resistance of ships, the Hamburg Model Basin has carried out basic investigations of the resistance components in icebreaking. Instead of using ship models, which seem to be much too complicated for analytical and physical modeling of the icebreaking phenomena, the investigation was carried out on the simple body of an inclined plane plate, (Schwarz, Kloppenburg, 1976). This simplification allowed the experimental and theoretical determination of the different resistance components separately.

In order to determine the breaking portion the lower edge of the plane plate touched the ice surface only as much as necessary in order to break the ice cover without any horizontal displacement of the broken ice pieces. By another run the broken ice was submerged and pushed out of the cross-section of the inclined plane plate. Parameters being investigated were the ice thickness $h$, the velocity $v$, the inclination angle $\alpha$, the width of the plate $b$, the friction coefficient of the plate $\mu$, and the depth of the lower edge of the plate $t$.

The results of the experimental investigation can be summarized as follows:

1. The fracture portion of the resistance depends on the square of the ice thickness (Fig. 6).

2. This breaking resistance is almost independent of the velocity and the width of the plate (Fig. 7 and 8), which indicates that the failure is mainly due to shear at both sides of the plate.

3. The same breaking resistance depends linearly on the tangent of the sum of the inclination angle $\alpha$ plus the friction angle $\mu$ (Fig. 9). Since the results for
different friction coefficients do not fall on one line, the friction effect seems to be more complicated and certainly a topic for further investigations.

4. The normalized resistance due to removing the broken ice out of the cross-section of the plate depends on the square of the Froude number, in which the unit of length is given by the depth by which the ice pieces have to be submerged (Fig. 10).

By simplifying the icebreaking model from the icebreaker bow to the plane plate, it was possible to approach the problem also theoretically. The fracture portion of the resistance was calculated on the basis of the elastic theory for the case of two linear loads on an elastic foundation. The theoretical approach considered the distance between the linear loads as well as a horizontal force applied at the free end of the semi-infinite ice cover.

The most important information from this theoretical investigation is that the resistance depends on $h^{7/4}$, which is close to the experimental results of $h^2$. The theoretical work can certainly be improved by using an energy approach on this basis of a three dimensional failure criterion. Such a criterion has, however, not been established for saline ice yet.

By these investigations it has been shown that the resistance of an icebreaking body depends on simple physical parameters. The shape of the body and the friction effect must be taken into account by constants, which are subject to experimental determination. Any additional parameters like the ship length, a lateral pressure, or a snow cover on the ice should be considered by an additional term of a sum rather than by manipulating the exponents within the dimensionless groups.

The experiments with the inclined plane plate have shown that the ice cover breaks in two phases: at first, shear lines occur at both lateral edges of the plate, whereby a cantilever beam is being formed; in the second phase this cantilever beam breaks off by bending. The shear failure dominates the amount of resistance. Since the shear strength is the smallest strength of all (half of the flexural strength), it suggests itself to look for icebreaker concepts which break the ice by shear. The new Canadian Coast Guard icebreaker POLAR 7, designed by German and Milne, takes advantage of this idea and so does a new icebreaker concept, developed at the Hamburg Model Basin.

This new icebreaking concept - called WAAS-Icebreaker - has a pontoon shaped bow with gliders on both sides of the plane inclined stem. The forebody is slightly wider than the midship in order to minimize the frictional resistance along both sidewalls of the ship. The ice cover is being broken by shear failure at both sides of the plane stem. The bottom of the midship section has a V-shape, which allows the buoyant ice pieces to float sideways underneath the remaining icecover at both sides of the icebreaker.

This motion of the ice floes is supported by laterally directed water jets situated in the bottom of the midship. Mainly due to the shape of the icebreaker the aft portion of the icebreaker and the channel behind it is free of broken ice. This means that the possibility of propeller damages due to the impact of broken ice is reduced.

The model tests have shown that the resistance of this WAAS-icebreaker is smaller than that of a conventional icebreaker.
In addition to the propeller at the stern, a mechanical propulsion device (Stempelantrieb) situated at both sides of the plane stem was found to be most effective. By using the mechanical propulsion devices on one side only, the manoeuveribility of the ship in ice was improved.

This new icebreaking concept is presently being further developed, including a pilot study.

Closing Remarks

The ice model technology certainly has made progress during the last years, without being complete yet. The testing methods of the Hamburg Ice Tank seems to provide the most logical results which are also in agreement with theoretical approaches. The lack of reliable full scale results as well as thorough theoretical investigations leaves still space for different opinions.

In order to place the whole ice technology on a sounder foundation, it is urgently suggested to perform full scale tests as carefully as possible. Besides, it is necessary to establish theoretical calculations, possibly along the line of Milano (1975) for ships resistance or of Reinicke and Ralston (1976) for offshore structures. With respect to those theoretical approaches, more basic investigations on the mechanical properties of ice and on friction effects are needed.

There is plenty of work for all of us!

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Schwarz, J., Kloppenburg, M., "Untersuchung über das Widerstandsverhältnis zwischen Modell und Großausführung eisbrechender Schiffe", HSVA-Bericht Nr. 76/82, Hamburg 1976

Vance, G.P., "A Scaling System for Vessels Modelled in Ice", Ice Tech 75, SNAME-Ice Symposium, Montreal, Canada, 1975


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![Diagram](Fig. 1 Normalized strength vs. normalized elasticity of saline ice)
Fig. 2 Force-Deflection Curves of two Different Types of Model Ice

Fig. 3 Total resistance versus ship speed for icebreaker MACKINAW in two feet thick ice as predicted by various formulations (after Milano, 1975)
Fig. 4 Icebreaker Resistance vs. Speed Curves as Theoretically Prediction by Milano and as Obtained by HSVA-Model Tests

Fig. 5 Effective Strength $R/h^2$ vs. Flexural Strength of Model Ice with Different Salinities
Fig. 6  Fracture Portion of Resistance vs. $\sigma_o h^2$ for two Different Friction Coefficients

Fig. 7  Fracture Portion of Resistance vs. Velocity

Width of plate = 25.0 cm
Velocity = 8.0 cm/s
Inclination angle = 30.0°
Strength $\sigma_o = 0.8$ kPa
Elasticity $E = 2000$ kPa
Ice thickness = 2.7 cm
Velocity = 6.4 cm/s
Inclination angle = 30.0°
Friction coefficient $\mu$ = 0.15
Strength $\sigma_0$ = 0.8 kp/cm²
Elasticity $E$ ~ 2000 kp/cm²

Fig. 8 Fracture Portion of Resistance vs. Width of Plate

Fig. 9 Normalized Icebreaking Portion of Resistance vs. Tangent of Angle of Inclination + Roughness for two Friction Coefficients
Fig. 10 Normalized Resistance for Submerging and Accelerating the broken ice vs. Square of Froude Number

width of plate $b = 40$ cm (constant)
INTRODUCTION

Sound management is a prerequisite in the industrial enterprises of our time. The manager should have a good insight in the various factors which determine the cost and the magnitude of the production, and should be able to identify the limiting factors so that adequate measures can be taken timely enough to prevent stagnation in the development of the industry concerned. The manager should have enough persuasion and a sufficient grip of the finance of the enterprise to take action when and where such is necessary. A good foresight is required, for especially in our time the economical and social parameters are liable to change almost overnight.

In many industries there is a growing concern about threatening depletion of its primary resources, but the fishing industry has the advantage that its natural resources rejuvenate themselves. Scientific investigations have revealed that fish stocks living in fertile waters show an annual increment in weight of some 15%, an increment simply due to growth of the fish. If one considers the fish stock as a natural capital then this 15% is the interest made. If one should harvest only this interest and leaves the capital itself untouched, one could expect to go on fishing for a long series of years without observing signs of depletion or overfishing. Therefore it seems wise, in behalf of supplying the market with animal protein harvested from the aquatic environment, to manage the fishery properly and to avoid any form of wasteful exploitation. In most of the western countries facing the sea scientific information is available on growth and reproduction in the species of fish of commercial importance and on the losses brought about by both natural factors such as predation, diseases and parasites, and the fishing industry. On the other hand there is sufficient information available on the size of the fleet, on the power of its units and on the gear used to draw sound conclusions on the fishing effort. Further there is as a rule reasonably reliable statistical information on the catches landed and on the proceeds made. One is also informed about the money invested in the fleet, about the number of workers in the fishing industry, and about the running expenses for fuel, netting and wages. In combining the scientific information available with the economic and social aspects of the fishing industry it must be possible to find a basis for a sound fishery management. The fact that more than one country operates in one and the same water may complicate the matter, but one would at least expect proper management, leading to the so-called optimum sustainable yield - expressed in tons of fish - or if one...
prefers so, to the optimum sustainable profit - expressed in monetary units - in the fisheries in territorial waters and in rivers and lakes within the national boundaries. Whether such is the case can be concluded from the following examples, all dealing with fisheries in the North-East Atlantic area, fisheries in streams and lakes, in estuaries and coastal waters, and at the high sea.

THE SALMON OF THE RIVER RHINE

Once the river Rhine teemed with salmon. In the years following 1869, for which reliable statistics are available, one caught in its downriver section some 50 000 to 60 000 salmons per year, almost all of them big fishes, weighing some 10 kg each. Only some 14% of the catches consisted of males participating for the first time in the run, weighing some 2 kg each. The Kralingse Veer near Rotterdam was then the big market for salmon. One used huge nets to catch the migrating salmon, operating either from some type of raft or from a fixed installation on the river bank. Originally the nets were drawn in by hand, but in the shore installations one could make use of a horse-mill for this heavy work. In due course one decided to make use of steam power to draw in the nets. This made it possible to use still larger nets, blocking off the river hermetically. Since three of such large nets were operated in turns, but few of the migrating salmon got a chance to escape. The catches increased sharply by the introduction of steam power and in the years following 1880 one could often bring more than 100 000 salmons to the market. But this could not last. Too few salmons could reach the spawning grounds in the upriver tributaries. First the percentage of small salmons went up, next the total catches dropped alarmingly.

In the year 1885 Germany, The Netherlands and Switzerland concluded the so-called Salmon Treaty, which led to interruption of fishing for salmon during the week-ends and during a close season late in summer, but the expected and forecast return of large catches of salmon from the river Rhine stayed away. In fact, the measures taken came too late and moreover overfishing was followed by the construction of more and more weirs and barrages for shipping and hydro-electric stations, which made it impossible for the salmon to reach its spawning grounds in the tributaries. Here and there one constructed fish passes, but those were evidently not of a design appreciated by the salmon, and those fishes which did manage to bypass the barrages found their habitat deteriorated: instead of water running swiftly over gravel beds, the salmon found above the barrages almost stagnant water with a muddy bottom, totally unsuitable as spawning ground. Then came the rapid industrial pollution of the water of the river Rhine which gave the last few salmons caught a nauseating phenol flavour.

Was there no proper management at all? Evidently not. It is incredible that one did not foresee and forestall the disastrous results of hermetically blocking off of the river by a series of nets hauled with steam power. Towards the end of the nineteenth century one decided to release hatchery reared salmon in an effort to revive the once so prosperous stock, but though quite a bit of money was spent this way, one did not reach this goal. Overfishing, barrages and water pollution combined had deteriorated the situation for the salmon so drastically that a revival of the stock had become impossible. Of course, one could think of international measures to check the serious industrial pollution of the water of the river Rhine, now virtually an open sewer. This would be a costly measure, beneficial for other
interests too, but who would suggest to take away the weirs and barrages to restore the salmon's spawning grounds in the upriver areas? It seems for economic reasons much wiser to rely for purchase of fresh salmon on the recent development in Norway, where one rears salmon to consumption size in netting pens of various construction placed in well-sheltered fjords.

FROM ZUYDER SEA TO LAKE IJSSEL

In the year 1932 the communication between the North Sea and the Zuyder Sea became cut off by the completion of a huge enclosing dam. Henceforth it would be impossible for the stock of spring-spawning herring to reach its low-salinity spawning grounds in the southern part of the old Zuyder Sea, and for the anchovy, the money maker among the Zuyder Sea fishes, to find the warmer waters prevailing there in fine summers. The entire pattern of the fishery would change. The river IJssel, a branch of the Rhine, discharged so much fresh water that it took only a few years to transfer the old brackish Zuyder Sea into a lake containing low-salinity water, practically fresh enough to drink. At the same time it brought ample nutrients from the hinterland to which were added the discharges of sewers from the cities around the old Zuyder Sea, Amsterdam included. As could be expected from this pattern the water of Lake IJssel soon revealed a rich plankton and bottom fauna on which fishes adapted to low salinities could thrive. It could be expected that Cyprinids such as the bream (Abramis brama) would thrive there, together with the ruff (Acerina cernua), a small member of the perch family, and that the smelt (Osmerus eperlanus), being very euryhalinic, would survive here, be it as stock of rather smallish specimens, but all those would hardly be of any direct value for the fishery. It would be of considerable interest if the common perch (Perca fluviatilis) and the pike-perch (Lucioperca lucioperca) would thrive in the new Lake IJssel, for those fishes are welcome on the market. For the perch one felt quite sure that its stock would soon increase, but for the pike-perch, originally a non-indigenous species, some doubt seemed justified. Above all one hoped for the development of a rich stock of eels, the most valuable of the fresh water fishes occurring in the waters of The Netherlands. The elvers, born in the Sargasso Sea, had to make their entry from the north and would find the enclosing dam in their way. How should they take this obstacle? It is true, that two groups of locks had been constructed in the enclosing dam which would facilitate their passage if they could find them, but the engineers in charge insisted on keeping the saline waters from the open sea away from Lake IJssel. First one tried to let the elvers pass the locks in the same way as the ships for which they were constructed, but on the basis of scientific investigations one developed in due course an ingenious system of nocturnal manipulations with the doors of the locks which led to large-scale passage of elvers into Lake IJssel without any intrusion of salt water. Would the elvers stay in Lake IJssel or would they make their way into the hinterland? Fortunately many juvenile eels decided to stay in Lake IJssel, richer in food than most of the waters further upstream.

The transition from Zuyder Sea to Lake IJssel would offer an excellent opportunity to change the pattern of the fishery in such a way that one could speak of a rational exploitation of the natural resources. A bill had been passed which facilitated financial compensation for those of the old Zuyder Sea fishermen, who would lose their living. Therefore there was a good opportunity to set up an entirely new pattern for the fishery in Lake IJssel. If the eel was to become the leading species, it would be wise to catch them
as silver eels, preparing for migration to their spawning grounds, for then they reach their highest market value. This is the system so effectively used in the Lagoon of Comacchio in Italy. For a variety of reasons the planning turned out differently. Catching eels exclusively as silver eels would not only reduce the fishing season to a few months a year, whereas there was a good demand for eels throughout the summer season, but would also be exclusively beneficial for those fishermen who operated in the migration routes of the eels, depriving fishermen elsewhere around the Zuyder Sea from catching eels. Moreover all this happened during the ill-famed depression years and from the side of the Ministry one insisted rather to keep a large number of small fishermen in the fishery than to operate with a smaller number of stronger units. It was even suggested to make use of sailing vessels only, and to abstain from use of engine power for reasons of economy.

What did happen? The eels found enough food in Lake IJssel. Part of them were caught in the immature state with a variety of gear, among which the traditional fine-meshed eel-trawl, used during the dark hours of the night. The eels which could escape from the various gears of the fishermen metamorphosed and tried to make their way to the ocean along certain migration routes. At that time only fyke nets with long leaders could be used to catch the swiftly travelling silver eels. Scientific research revealed that there existed a certain relation between the magnitude of the immigration of elvers in a given year and the success of the silver eel fishery some seven years later. This made it even possible to forecast the results of the silver eel fishery. To prevent that the immature eels would be caught at too small a size, a minimum legal size of 28 cm was introduced, and to prevent too high a fishing intensity for immature eels no engines stronger than 25 hp were allowed in the fishing boats operating in Lake IJssel. The advice to stick to sailing vessels only was not accepted by the fishermen, for they soon found out that engines would allow them to operate in almost any type of weather and would lead to better economic results.

It was no easy matter to enforce the minimum legal size for immature eels, but the 25 hp rule seemed to work quite well. Around the year 1960 the eel fishery in Lake IJssel did not seem to flourish as well as before and it was especially the fishery for silver eels which led to much poorer results than forecast on the basis of the immigration of elvers. Scientific research carried out by the Netherlands Institute for Fishery Investigations revealed the cause of the decline. The lack of silver eels had to be ascribed to an increased fishery with the eel trawls, this not because more ships participated in this fishery, but rather because clever fishermen had learned to abuse the 25 hp rule in several ways. A faster boat appeared to lead to greater catches and better proceeds. Other fishermen saw this happen and soon followed the example, though dangerous and illegal. It soon turned out that some of the boats worked in reality with a horse power of 60 to 80 instead of 25. No small wonder that the silver eel catches had declined so drastically! The scientific report made also clear that the intensive trawl fishery for immature eels made it virtually impossible for the stocks of perch and pike-perch to develop, since too many juvenile specimens found an untimely death in the fine-meshed nets.

The report led to a serious clash in the Ministry and even to dismissal of its Director of Fisheries. A special Commission was set up in which both professional fishermen and sport-fishermen were duly represented. It could only corroborate the allegations of the biologists that the fishing intensity
had gone up alarmingly since the 25 hp limit became grossly abused and the 28 cm minimum legal size for eels was not always observed. The professional fishermen even admitted that they often fished for eels in the forbidden coastal zone, the spawning and nursery grounds for perch and pike-perch, though duly delimited by lights, and that the installation of radio-telephones on their vessels made it possible to keep each other informed on the whereabouts of the patrolling police-vessels. No small wonder that many pike-perch nests were destroyed and that the trawls made a terrible slaughter of the juvenile perch and pike-perch. Something had to be done to restore the former productivity of Lake IJssel. Reduction of the fishing intensity, especially of the trawl fishery seemed imperative, but one hesitated to take such a drastic measure and considered for the time being a step by step reduction of this fishery.

It was a political move, set going by the Organization of the Sport Fishery, which led to a breakthrough. Based on a passage of the report of the Commission, indicating that measures to protect the young pike-perch would be especially effective when a rich year-class would be forthcoming, they used the information that the juveniles born in the year 1969 would be numerous to take action. This move led to a complete prohibition of the nocturnal trawl fishery with fine-meshed nets, beginning with the 1970 season. Fierce protest from the side of the fishermen, the eel smokers and dealers, who claimed to lose their living, did not lead to alteration of this Ministerial decision.

What has been the result? It took a few years of adaptation to the new situation. The fishermen soon learned to catch immature eels with other gear than trawl nets: hooks and line, in use since long, baited wooden eel boxes and series of special fyke-nets set on the bottom. After some years had passed the silver eel catches showed a spectacular recovery which soon led to great prosperity among the fishermen operating with fyke-nets on the migration routes. Full protection of juvenile specimens of perch and pike-perch led to a spectacular increase in the catches of these species, welcome on the market in several countries. An explosive reproduction of the ruff, feared by the fishermen, stayed away. The whole fishery of Lake IJssel prospers again, even decidedly better than ever before, and the fishermen soon forgot their fierce protestation against the drastic changes in the fishery pattern. An unexpected result was a much faster growth of the immature eels, which now reach the stage of silver eels in fewer years than before. It is surmised that staying away of the disturbance produced by the trawl nets must have led to more peaceful conditions on the bottom of the lake and thereby to better growth of the eels, but for the time being this explanation is only a working hypothesis. Lake IJssel brings now higher proceeds than ever before, the only worry left being the influx of heavy metals, phenols and chlorinated hydrocarbons via the river IJssel, but the faster the growth of the fish, the shorter the time available for accumulation of those obnoxious pollutants in their flesh.

Many things happened since the year 1932 in the Lake IJssel fishery, and finally all turned out well, but is there any reason to claim that this favourable outcome has as basis a wise fishery management?
The brown shrimp (Crangon crangon) abounds in the coastal waters of the eastern shore of the North Sea, from Strait Dover till Jutland. It is rather smallish, up to 6 cm long, but of an exquisite flavour. A fishery for brown shrimps developed in due course along the usual empirical lines, not guided by scientific advice or official regulations, and once established the fishermen claimed that it was their lawful right to make a living that way. First one fished from the beaches, in which horses were often used on the Belgian section of the coast, later boats came into use starting with sailing vessels, then boats equipped with engines. Those operated a long time with trawls kept open by doors, but in due course the fishermen found the beam-trawl more efficient. It is only logical when fishing for creatures as small as brown shrimps that one uses fine-meshed nets. With those a by-catch cannot be avoided, but it took a long time before one learned more about the practical importance of the discards, of the impact of the destruction of creatures other than shrimps on the fishery elsewhere. Sets of shaking sieves were in use on the shrimpers to produce a marketable product free of alien objects and larger fishes were taken out by hand before one proceeded to boil the shrimps on board. It was not until a biologist of the Netherlands Institute for Fishery Investigations undertook a more detailed study on the brown shrimp and its fishery that one realized how many young flatfishes, especially plaice and sole, could be encountered in the catches at certain times of the year, especially in the Wadden Sea and in the Zeeland estuaries, the most important nursery grounds for North Sea flatfish. Of course, the fishermen must have seen this, but did not worry about it, since they fished exclusively for shrimps, whereas other fishermen, often from other sections of the country were after flatfish. The flatfishes in the by-catches were small, but often extremely numerous. Sufficient information was available on growth and natural mortality of these small flatfishes to allow calculation of the deleterious effect the brown shrimp fishery exerts on the flatfish fishery elsewhere in the North Sea. It turned out that the damage done to the recruitment of sole and plaice, expressed in money, exceeded by far the proceeds of the shrimp fishery.

For a fishery administrator such information is bad and unwelcome news, for one cannot just prohibit a fishery which offers already for centuries a living to ever so many fishermen. Should one have known the causal relation between the gathering of brown shrimps and of flatfish prior to the development of a professional shrimp fishery, then it would have been wise to decide to protect the flatfish in an effort to exploit the natural resources as rationally as possible, but it is for a fisheries administration almost impossible to cut short an already existing fishery with a long tradition. This would not only lead to fierce protest from the side of those who make a living that way, but would also upset the public opinion and would lead to strong opposition in Parliament, despite the economic logic of such a decision.

What happened in this case? One could not simply ignore the scientific evidence, especially not since the sole fishery had become the major money-maker in the Dutch fishing industry. Biologists and specialists in fishery techniques concentrated their efforts on this problem and found in due course two ways to reduce the losses suffered by the young flatfishes drastically. The first was introduction and adaptation of a so-called sieve-net, first developed in France, which led under certain conditions to large-scale escape
of flatfishes without any reduction in the shrimp catches. But more important still was the development of an ingenious apparatus to be installed aboard which could sieve the catch in water in such a way that both young flatfishes and juvenile shrimps return undamaged, alive and kicking, to their natural environment. The Dutch Government decided to promote the introduction of this apparatus on the shrimpers of the Dutch fleet and subvented its purchase for a certain percentage. It could, however, not compel all shrimp fishers to buy this special equipment and to use it, but since the fishermen observed that it is also a labour-saving device, there is a general tendency to introduce it on those ships which lend themselves to it. This way the deleterious effect of the shrimp fishery on the flatfish fishery elsewhere in the North Sea can be reduced to such an extent that both fisheries can reasonably co-exist.

THE DUTCH SOLE FISHERY

Of all the fishes caught in the North Sea the sole (Solea solea) brings by far the highest price per kg. Contrary to the events in most other species of appreciable commercial value the sole catches made in our time are definitely higher than those made earlier in the century. One would suppose that it are the larger and faster ships and the more efficient gear which could explain the greater sole landings in post-war years, but there is scientific evidence, based on egg-counts in the plankton, which indicates that the stock of North Sea sole must really have been of a far lesser magnitude earlier in the century.

The Dutch fishing industry, specialized in export of fresh table fish, concentrated its efforts more and more on catching sole. Specialized units were built for this purpose. The best catches could be made shortly after sun-set when the sole begins to forage, and again just before sunrise when the sole gets hungry again, but since one learned to use whole series of tickler chains, which chase the sole out of the sand any time of the day, the sole catches are no longer limited to the dark hours of the night.

More and more units were built specially for the sole fishery and in the mid-nineteenfifties the day came that the financial proceeds of the sole fishery surpassed those of the herring fishery, which had predominated for ages in the Dutch fishing industry. The North Sea sole fishery became more and more a Dutch business, and now 80 to 85% of the sole caught is taken by Dutch vessels. This would make it possible to develop a sound management for this fishery. International agreement would hardly be necessary, for all measures taken would operate practically exclusively in behalf of the Dutch fishery.

What did happen? Scientific investigations had made clear that reproduction shows sharp fluctuations in the sole, sharper than in many other fish species. Though the number of eggs produced during the spawning season will not show major deviations from the average, the success of reproduction varies largely. Poor years, in due course leading to at most a modest recruitment, alternate with good years producing excellent catches a few years later. Unfortunately really rich yearclasses seem to occur only about once in ten years. Very great numbers of juvenile soles were, for instance, produced in the years 1947, 1958, and particularly 1963, whereas 1969 gave a reasonably good birth rate. About 2½ years later the first "recruits" make their appearance in the catches. The question which factors are to be held
responsible for success or failure in reproduction is still far from being solved, but even if one knew this one would still have to wait and see what nature has in mind.

If a really good yearclass has grown up to recruitment size the fishermen will notice this by a sharp increment in the percentage of juvenile soles in the catches. Since the market for soles is excellent in many European countries big money will be made when the catches are good. Therefore a rich yearclass incites the fishermen to enlarge their fleet, which shows consequently neat peaks roughly 4½ years after the birth of each rich year-class. The 1973-1974 peak in the number of fishery units is, however, higher than could be expected on the basis of the size of the 1969 yearclass only, but easy terms for getting money from the banks worked out that way. The money made is not only invested in more ships, but also in bigger and stronger ships for this same type of fishery. A bigger ship with a stronger engine makes better catches, though it also requires greater investment and higher costs for fuel and gear. Fishermen are above all keen on great catches and therefore tend to build ships which are bigger and stronger than those of their colleagues. This would not matter if one reduced at the same time the number of units in the sole fishery, but neither the fishing industry itself, nor the Government has the power to achieve this. The final result is a fleet too big and too strong for the sole there is to catch. Surely, the stock of soles does not keep pace with the size of the fleet. The fishermen saw the catches made per hour fishing decline alarmingly and tried to compensate this by building still bigger and stronger ships. For an individual fisherman this may seem to help, but the total catches will not go up, on the contrary. Now the banks which offered money on too easy terms begin to realize that brand-new ships specialized for the sole fishery are not such a good guarantee for their money when there is not enough fish to be caught to cover the running expenses.

Scientific evidence shows that bigger catches could be made by a further increase in the mesh size of the trawl nets, though this would initially lead to a reduction in the catches, but also that the Dutch cutter fleet specialized for the sole fishery is much too strong. With a fleet about one third of the present size one could catch the same total tonnage of sole as today. But the question arises who is going to give way and how one can prohibit someone to build still another ship for the sole fishery. A smaller fleet landing the same annual catches as today would greatly increase the remuneration of each individual ship, but how could one reach this goal on basis of the present set of fishery laws and regulations? The scientists know that even a fleet as large as today will not exterminate the North Sea sole and that a depleted stock may in due course recover when the fishing intensity is finally reduced because one loses too much money operating in the present way. But they are not too sure that nature will readily produce a good stock of soles once the fishing intensity would be drastically reduced. There is such a thing as the "stock-recruitment relation" which means that recovery of a stock will take a long time once the total number of eggs produced during the spawning season has dropped below a critical level. The biologists studying the North Sea sole are afraid that this critical level does already operate, and therefore dare no longer prophesy that a good year-class will be there at least once in ten years time.

What management would be conceivable to safeguard the future of the Dutch sole fishery in which so much money has been invested and for which there is
not enough fish?

THE NORTH SEA HERRING

For ages the herring fishery predominated in the North Sea. Several countries participated in it, the usual gear being the driftnet set out at sunset. The mesh size was such that only adult herring, rising to higher water layers in search of food in the dark hours of the night, would get entangled by its gill covers. In the period between World War I and World War II one observed symptoms of serious over-fishing in demersal species such as flatfishes and gadoids in the North Sea, but not in the herring fishery. Samples of herring caught in the driftnets appeared to consist of several yearclasses, sometimes up to ten. Thus the existing fluctuation in the strength of the individual yearclasses was hardly reflected in the size of the catches. When one decided in 1946 to set up an international regulation of the North Sea fishery to prevent over-fishing, pelagic fishes such as herring and mackerel were considered to be so plentiful that they needed no protection at all. No legal minimum size was considered necessary for the herring.

Driftnetting revived after World War II, but in addition one began more and more to use fine-meshed trawlnets to catch the herring while it sojourns close to the bottom during the daylight hours. This became possible by use of boats with stronger engines which could move the net with a speed faster than that of herring trying to escape by swimming. Gradually the trawls caught more and more herring and finally driftnets became obsolete in the Dutch fishery. This development was furthered by the introduction of electronic fish-finding equipment which made it possible to locate the herring schools with considerable precision. Since herring schools are sometimes observed in midwater one developed a pelagic trawl to chase the herring wherever it occurred.

This development led to a noteworthy increment of the fishing intensity for herring, and thereby to a reduction of the number of yearclasses found in the catches. The intensive fishery made it virtually impossible for the herring to reach an age of 10 to 15 years, as it could with some good luck in earlier times. In reality the catches made in the last few years consist virtually exclusively of the yearclass which is just recruiting, herring of three years old, whereas herring of earlier yearclasses represent only a small minority in the catches. The stock which spawns in The Channel late in the year and congregates along the east coast of England in late summer and early autumn, suffered more from intensive trawling than any other stock of North Sea herring and lost thereby much of its former importance.

But perhaps the prolific North Sea herring could have coped with this development if it had not been attacked from two sides by an industrial fishery. First the so-called Bløden fishery, taking juvenile herrings in the German Bight to be reduced to fish meal in Danish and German plants, and next the powerful attack by the newly developed Norwegian fish meal industry in the nineteen-sixties. Technical improvement made it then possible to haul huge purse seines with the aid of a power block and thus entire schools of herring, traced by electronic fish finding equipment, could be caught. The so-called Atlanto-Scandian herring, the northernmost stock, consisting of large-sized individuals, migrates in winter to some sections of the Norwegian coast to spawn. Norwegian fishermen used to land appreciable quantities of this herring of which the greatest share was reduced to fish.
meal, the market for direct human consumption being very limited in Norway. Now the introduction of the power block made it possible to expand the fishery on this herring stock, thought to be inexhaustible. The Norwegian Government backed the expansion of the fish meal industry and soon huge quantities of herring were landed. But the stock was not inexhaustible at all, and dwindled to insignificance within a few years towards 1970. Too much money was already invested in ships and fish meal plants to stop the ill-planned Norwegian industrial fishery. Therefore one went with all force after the mackerel and after the North Sea herring proper. It was especially this latter development which brought the North Sea stock of herring down to an ebb so low as one had never seen before. The International Council for the Exploration of the Sea warned the North-East Atlantic Fisheries Commission that too heavy a toll was levied of the North Sea herring and that only drastic measures could safeguard the future of the herring fishery. The Council noted that the 1973 yearclass was of reasonable magnitude, able to secure a good offspring, but that later yearclasses were poor or even extremely poor. Therefore the Council advised to abstain from fishing on the 1973 yearclass in an effort to offer the North Sea herring an opportunity to revive its stocks. The North-East Atlantic Fisheries Commission discussed this advice seriously, but the fishing industry refused to follow it because one simply needed the herring now. All one did at this very late moment was to introduce a minimum legal size for herring, which meant that the Danish fish meal industry would no longer be allowed to fish purposely for juvenile herring in the North Sea. It could, however, fish for sprat, and among the sprat one often finds juvenile herrings as a by-catch.

A quota system was introduced by the North-East Atlantic Fisheries Commission for all species of commercial importance. Also for herring, but the countries which traditionally fish for herring destined for direct human consumption see with dismay that the break-down of the overall herring quorum over the various nations leads to by far the greatest share for Norway, based on the magnitude of its catches in recent years. Many countries, among which The Netherlands, claim that high priority should be given to fishing for direct human consumption, but on what basis could one compel a given country to eat the fish it catches instead of feeding it to pigs and chickens after reducing it to fish meal?

The future seems very gloomy for the North Sea herring fishery. The advice of the scientists to give the 1973 yearclass a chance to produce its offspring was flung to the winds and hence this yearclass was wiped out by the 1976 fishery. Later yearclasses are so poor that one can hardly expect a revival of the stocks, even when they are completely left in peace. The danger is then that sprat takes over from the herring in the North Sea, in the same way the anchovy took the place of the over-fished sardine in the coastal waters of California. Stocks of demersal fishes such as flatfishes and gadoids will not run the risk to be fished down to such a low ebb as schooling pelagic fishes such as herring and mackerel. With modern fish finding equipment and modern gear one could trace and catch the very last school of these pelagic fishes, whereas the demersal fishes, which do not concentrate to such an extent, will be left alone when the catch per unit of effort drops below a given level.

Though all countries fishing for herring in the North Sea are seriously concerned about the present state of the stocks, it seems hardly possible to reach agreement leading to a sound management. There is no sound legal basis
to stop the industrial fishery and there are no international funds to buy it out. Every ton of juvenile herring taken as a by-catch reduces in fact the future stock of adult herring by four tons - the natural mortality taken into account - but only a strict control could enforce obedience to limiting the by-catch of the industrial fishery to the tonnage agreed. Countries traditionally fishing for herring destined for direct human consumption are very hesitant to accept regulations restricting their catches, as long as the real cause of all the trouble in the herring world, the industrial fishery, has not disappeared and as long as the quota are not based on the former catches for human consumption only.

CONCLUSIONS

The examples discussed can only lead to the conclusion that the type of management observed in industrial enterprises ashore does not find its equivalent in the fishing industry, though all the ingredients for a sound management seem to be available. In reality the present situation is rather chaotic in the North-East Atlantic area, despite the efforts made by the Permanent Commission for the Overfishing of the North Sea, followed by the North-East Atlantic Fisheries Commission, operating in a larger area. The proclamation of 200 mile zone by countries or groups of countries has thus far not led to a real improvement of the situation, to a closer approach of the desired rational exploitation of the natural resources in the sea.

What may be the main reason that it is so very difficult to persuade the fishing industry to strive towards a sound management? Most probably the general mentality of those engaged in it, which differs considerably from that reigning in the industrial enterprises ashore. In broad terms one could say that the fishery is not thinking in terms of consolidating its enterprises on long term, but is rather interested in quick profit. Fishermen are no investors, but adventurers challenged by the presence of fish in the sea, in the lakes and in the streams, taking the appropriate actions to catch at short notice as much of it as they possibly can. Good management, on the other hand, would require foresight, planning of the exploitation of the natural resources at longer term. The scientists in the working groups of the International Council for the Exploration of the Sea assess the fluctuations in the stocks of fish of commercial importance and warn the North-East Atlantic Fisheries Commission when overfishing threatens or is already evident. In that case they know that the best remedy would be to stop fishing on the stock concerned to offer it a chance to revive, but realize that such will be hard to accept by the fishing industry which has invested so much money in its fleet and wishes to supply its market without interruption. Therefore they advise as next best measure a less drastical reduction in the fishing intensity, which may also lead to revival of the stock, but then of course on longer term. The crucial point is that the fishing industry is interested in what happens on short term, but cannot accept a really drastic reduction of its activities, whereas a slow revival would require so many years that one feels hardly any interest in so remote an improvement. The fishing industry is very well aware of the advice of the fishery scientists, but just hopes that they may have been too pessimistic in their assessments. It has even been said that the quota computed by the scientists of the Council have purposely been set too low since one realized that the various international fishery industries would not be satisfied with the quota suggested and would insist on adding an appreciable tonnage to it! One will understand to how much frustration such unrealistic and
ridiculous assumptions will lead in the world of the scientists.

Still, there is no need for despair. The fertility of the aquatic environment and above all of the North Sea is hardly affected by the fishery. In spite of all the activities of the fishing industry the North Sea can yield year after year some two to three million tons of fish. If the fishing industry continues to overfish the commercially most valuable species one will observe an almost automatic increment in the stocks of species of lesser commercial value such as sand-eels (Ammodytes spec.), sprat (Clupea sprattus) and Norway pout (Gadus esmarkii). Such low-grade species, rightly called the opportunists among the fishes, can be used by the industrial fishery. The total tonnage landed from a fertile sea like the North Sea will not decline as a consequence of too intensive fishing, but the proceeds will drop considerably if the fishing industry is not really willing to accept a sound management of the natural resources. One has the choice: to strive towards good management to approach as closely as possible the optimum sustainable profit, or to lose the high-grade fish and to exploit the low-grade species for the fish meal industry. It is up to the international fishing industry to make this choice, for coexistence of the traditional fishery for human consumption with a large-scale industrial fishery in the very same area seems hardly possible, unless one manages to develop fishing techniques which ensure a really "clean" catch, devoid of by-catches. One should realize, however, that when the choice is for an industrial fishery that considerably less animal protein will come from the aquatic environment to man's tables than in case one decides to fish for direct human consumption.

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THE CASE OF THE BROWN SHRIMP


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Fig. 1. Salmon landings from the downriver sections of Rhine and Meuse, 1865-1935.

Total landings of salmon in the Netherlands (Rhine and Meuse)
Fig. 2. The sieve-net in action. Shrimps, frightened by the ground-rope, jump high and pass through the rather wide meshes of the false upper, to land finally in the upper cod-end. Small flatfishes and inert objects come in the lower cod-end, which can be left open.
Fig. 3. Modern equipment on board of a Dutch shrimper. In the centre the rotating sieve, flanked by the two tanks filled with sea water in which the catches of port and starboard nets are dumped to be fed automatically to the sieve. In the foreground the cooker and a second sieve to free the boiled shrimps of the last alien objects.
Fig. 4. The strength of the Dutch flatfish fleet under influence of good yearclasses of sole.
Fig. 5. a) The total landings of sole from the North Sea 1918-1976.

b) Developments in the North Sea sole fishery. The catch per unit of effort in kg per 100 hours fishing of a standard ship under influence of the increasing strength of the fleet.
Fig. 6. The dramatic events in the stocks of North Sea herring under influence of a drastically increased fishing intensity.
Fig. 7. Cartoon by J.F. de Veen on the impact of the Norwegian fish meal industry on the North Sea herring stocks. (First published in "De Visserijwereld").
INTRODUCTION

The scientific approach to nearshore currents was initiated rather recently. D.W. Johnson (1919), in his classic treatise, considered the longshore current and undertow as wave-induced currents. He tried to treat coastal sediment movement and coastal processes by regarding these currents as media of sediment transport. In the same book he pointed out the importance of mass transport phenomena derived by the finite amplitude wave theory in understanding the generation of the longshore current and undertow. The existence of a strong offshoreward current has been well-known to fishermen through their daily experiences, as well as to lifeguards and skilled swimmers. In 1941, Shepard, Emery, and LaFond published a paper entitled "Rip Currents", in which they defined rip currents as offshoreward currents which return the mass of sea water transported shoreward by the function of wave motion. They stressed that rip currents, which are concentrated within the surface layer, differ from the hypothetically accepted undertow in their flow pattern. Rip currents frequently exceed 1 m/sec in velocity, and occasionally extend more than 300 m offshore. Thus, the function of rip currents in carrying away fine suspended sediment particles from the surf zone to deeper water regions seems to be quite important in the overall pattern of sediment movement in the nearshore area. In 1950, Shepard and Inman carried out extensive field observations in the nearshore area in front of the Scripps Institution of Oceanography, University of California. From their results, they proposed the concept of a nearshore current system or nearshore circulation pattern. In advance of Shepard and Inman's field observations, Putnam, Munk, and Traylor (1949) proposed formulae to evaluate the mean velocity of longshore currents. Their results encouraged researchers to undertake further studies to clarify the longshore sediment transport mechanism and to estimate the longshore transport rate. However, it was very difficult to grasp the flow characteristics of nearshore currents at that time. For these reasons, the main interests of researchers were concentrated on the longshore current itself during the 1950s. In 1961, Longuet-Higgins and Stewart presented the radiation stress concept and applied it to analyze various water wave problems such as wave-current and wave-wave interactions. Since the end of the 1960s, the application of the radiation stress concept has been extended to such phenomena as wave set-down,
The principal difficulties in the field observations of nearshore currents result from the fact that the phenomena of interest associated with the current system extend over a broad area in and near the surf zone. From that perspective, Shepard and Inman (1950) executed the pioneering work. Figure 1 is a well-known schematic diagram of the nearshore current system drawn originally by Shepard and Inman (1950) and revised by Inman and Bagnold (1963). In order to investigate the entire pattern of this current system, it is desirable to measure simultaneously the wave characteristics such as the wave height, wave period, incoming wave direction, and current field with the bottom topography inside and outside the surf zone. For that purpose, aerial photographic surveying seems to be the most suitable way to perform systematic observations in the nearshore area. Therefore, the following two systems have been developed and applied to field observations in order to take successive pictures from the air at specified intervals.

**Synchronized Helicopter System (SIHELS)**

Figure 2 is a schematic diagram of the synchronized helicopter system. The descriptive outline of this system is given in Table 1, from which it is recognized that the area covered by a single photograph is fairly large in comparison with the other system (Balloon Camera System). The accuracy of the pictures thus taken is very high and satisfactory in the quantitative sense for the present purposes, but a great disadvantage is the extremely high operating costs.

**Balloon Camera System (BACS)**

The first observations by means of a balloon camera system were made by Sonu (1969, 1972). The writer's group also intended to apply the same system. Figure 3 is a schematic diagram of the system initially employed. The balloon is used to suspend a capsule housing a motor-driven camera, and has an inner volume of $33m^3$ filled with helium gas. The system has been successively developed into a stereo balloon.
camera system as shown in Fig.4. A detailed description of the balloon camera system is given in Table 2. At present, camera altitudes up to 700m have been achieved to cover a broader area. The fact that the wind speed should be less than 7 to 8m/sec is one disadvantage of the present system.

Table 1 Outline of the synchronized helicopter system.

<table>
<thead>
<tr>
<th>Helicopter</th>
<th>Kawasaki-Hughes KH-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Camera</td>
<td>Zeiss RMK 15/23</td>
</tr>
<tr>
<td>Film</td>
<td>Kodak Aerocolor-N</td>
</tr>
<tr>
<td>Photo Scale</td>
<td>1/3000</td>
</tr>
<tr>
<td>Altitude</td>
<td>450 m</td>
</tr>
<tr>
<td>Coverage</td>
<td>690 m x 410 m (stereo)</td>
</tr>
<tr>
<td></td>
<td>690 m x 970 m (mono)</td>
</tr>
</tbody>
</table>

Table 2 Outline of the BACS (Balloon Camera System) and the Stereo-BACS.

<table>
<thead>
<tr>
<th>BACS</th>
<th>STEREO-BACS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Balloon</td>
<td>Ovoid type gas balloon, volume 33m³</td>
</tr>
<tr>
<td>Camera</td>
<td>Hasselblad 500 EL/M 70 mm motor-driven</td>
</tr>
<tr>
<td>Lens</td>
<td>Zeiss Distagon 40 mm</td>
</tr>
<tr>
<td>Film</td>
<td>Kodak Ektachrome MS 5256 EMS 475 70 mm x 100 ft ASA 64</td>
</tr>
<tr>
<td>Shutter</td>
<td>Synchronic radio control</td>
</tr>
<tr>
<td>Altitude</td>
<td>Up to 700 m</td>
</tr>
<tr>
<td>Air Base Length</td>
<td>33% of altitude</td>
</tr>
<tr>
<td>Photograph Scale</td>
<td>1/7000</td>
</tr>
<tr>
<td>Plotting Machine</td>
<td>Wild Autograph A7</td>
</tr>
<tr>
<td>Map Scale</td>
<td>1/500 (variable with altitude)</td>
</tr>
<tr>
<td>Contour Interval</td>
<td>0.2 m (variable with altitude)</td>
</tr>
</tbody>
</table>
RESULTS OF FIELD OBSERVATIONS AND DISCUSSION

Nearshore Current Pattern

The nearshore current consists of the mass transport induced by wave action, longshore current and rip current, and the system makes a circulation pattern as shown in Fig.1. The region isolated by two adjacent rip currents forms a unit cell. The nearshore current system can therefore be considered as consisting of multiple cells. The above situation holds under the condition that the waves arrive almost perpendicular to the shoreline. When the incident wave angle increases, the closed cell disappears and forms a meandering current or a longshore current, as illustrated in Fig.5.

In order to ascertain the overall nearshore current system pattern along a particular coast, we employ the simple method of observing the current speed and direction by tracing the movement of floats from alongshore stations a specified distance apart. Figure 6 is one example of the observed results, from which the location of rip currents can be easily determined. Based on these preliminary investigation data, the site for the field observation and the photographic coverage are selected. The research group at the University of Tokyo has done extensive field observations on various coasts during the last ten years, as illustrated in Fig.7.

Rip Current Spacing

The scale of a rip current is represented by the rip current spacing $Y_r$, the breaker zone width $X_b$, and the extension of the rip current from the breakerline to the offshore end of rip head $X_r$ (see Fig.1). The data obtained for $Y_r$ and $X_b$ are plotted in Fig.8, together with other available data. The data for El Moreno and Silver Strand beaches are reported by Inman, Tait, Komar and Nordstrom (1968), those on Scripps beach by Shepard and Inman (1950), those on seagrove beach by Sonu (1972), and those on Virginia beach by Harris (1969). The ranges of various factors covered in these data are as follows; $\tan \beta$ (beach slope) = 1/7 (El Moreno) ~ 1/85 (Kujukuri), $H_b$ (breaker height) = 0.2m (El Moreno) ~ 4.1m (Scripps), $T$ (wave period) = 2.8 sec (El Moreno) ~ 14.2 sec (Scripps), $X_r$ = 2.4m (El Moreno) ~ 300m (Scripps), and $Y_r$ = 6.4m (El Moreno) ~ 800m (Scripps). According to these results, it is seen that the data for the ratio of $Y_r$ to $X_b$ are scattered within the range of 1.5 to 8.0, but concentrate densely near the line $Y_r/X_b = 3$. This fact agrees well with the result of McKenzie (1958).

Generation Mechanism of Rip Currents

One of the most interesting problems in this field is the generation mechanism of rip currents. There are several hypothesis on this subject. Bowen and Inman (1969) assumed a longshore wave height perturbation which gives the radiation stress component spatial distribution, hence the driving forces. The question is why such a wave height variation appears. They considered the influence of edge waves, especially of standing edge waves, as the main cause. Sonu (1972) and Noda (1972), on the other hand, stressed the importance of the bottom topography in the nearshore area, which causes the convergence and divergence of wave energy due to the refraction of incoming waves. This opinion is supported by the fairly good agreement between numerical calculation under appropriate assumptions and the actual data obtained in the field. However, another question arises regarding the mechanism by which such rhythmic topography is generated on the sea bottom.
Hino (1973) treated the rip current generation mechanism from a completely different standpoint from those above. His basic idea is that if waves are arriving perpendicular to a straight and parallel beach, wave set-down and set-up should be uniform along the shoreline. Such a uniform wave set-up would be unstable to an infinitesimal disturbance, just as a slender rod compressed axially buckles when a critical compressive stress is exceeded. The important conclusion is that the optimum spacing for rip currents and beach cusps is about four times the surf zone width.

Sasaki (1974) proposed an infragravity hypothesis and presented a diagram in which he summarized various field data on rip spacing. He combined the previous theories such as Bowen and Inman's edge wave theory, and Hino's hydrodynamic instability theory with his proposed infragravity hypothesis. The diagram is given in Fig.9, where $I_r$ is the Irribarren number for surf similarity defined by the following equation (Battjes, 1974):

$$I_r = \frac{\tan \beta}{\sqrt{H_o/L_o}}$$  \hspace{1cm} (1)

Here, $\tan \beta$ is the bottom slope in the nearshore region, and $H_o/L_o$ the wave steepness in deep water.

There are several other treatments of the rip current generation such as those by Mizuguchi (1976) and Iwata (1976) who independently discussed the present problem as an eigenvalue problem.

**Three Domains of the Nearshore Current System**

Battjes (1974) classified breaker types using $I_r$ as followings:

* surging or collapsing: $I_r > 3.3$
* plunging: $3.3 > I_r$
* spilling: $0.5 > I_r$

In the same way, Sasaki (1974) classified the nearshore current system into the infragravity, instability, and edge wave domains, an outline of which is illustrated in Fig.9. A detailed description of these three domains is given in Table 3.

In each domain, the ratio $Y_r/X_b$ is given by

<table>
<thead>
<tr>
<th>Domain</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infragravity domain</td>
<td>$Y_r/X_b = \frac{157}{4} I_r^2$ $(0.23 &gt; I_r)$</td>
</tr>
<tr>
<td>Instability domain</td>
<td>$Y_r/X_b = \pi$ $(1 &gt; I_r &gt; 0.23)$</td>
</tr>
<tr>
<td>Edge wave domain</td>
<td>$Y_r/X_b = \left{ \begin{array}{l} \pi/2 \ 2\pi/3 \ \vdots \end{array} \right.$ $(I_r &gt; 1)$</td>
</tr>
</tbody>
</table>

That is to say, the nondimensional rip current spacing $Y_r/X_b$ is determined as a function of the incoming wave characteristics and the beach slope.
Table 3 Three domains of the nearshore current system (after Sasaki, 1977).

<table>
<thead>
<tr>
<th>Domain</th>
<th>Infragravity</th>
<th>Instability</th>
<th>Edge wave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range of $I_r$</td>
<td>$0.23 &gt; I_r$</td>
<td>$1 &gt; I_r &gt; 0.23$</td>
<td>$I_r &gt; 1$</td>
</tr>
<tr>
<td>Breaker type</td>
<td>Spilling</td>
<td>Spilling/plunging</td>
<td>Plunging/surging/collapsing</td>
</tr>
<tr>
<td>Surf zone</td>
<td>Always exist</td>
<td>Exists or does not exist</td>
<td>Does not always exist</td>
</tr>
<tr>
<td>Number of waves in the surf zone</td>
<td>More than 3 waves</td>
<td>1~3 waves</td>
<td>Less than 1 wave</td>
</tr>
<tr>
<td>Reflection coefficient $r$</td>
<td>$r &lt; 10^{-2}$</td>
<td>$r \sim 10^{-2}$</td>
<td>$r &gt; 10^{-1}$</td>
</tr>
<tr>
<td>Incident wave characteristics</td>
<td>Wind waves &amp; swell</td>
<td>Swell</td>
<td>Arbitrary</td>
</tr>
<tr>
<td>Microtopography</td>
<td>Longshore bar</td>
<td>Crescentic bar</td>
<td>Beach cusp</td>
</tr>
<tr>
<td>Classification by Guza and Inman (1975)</td>
<td>Dissipative system</td>
<td>Reflective system</td>
<td></td>
</tr>
<tr>
<td>Remarks</td>
<td>Incident wave with large wave steepness</td>
<td>Incident waves with small wave steepness</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Gentle bottom slope $\tan \beta &lt; 1/50$</td>
<td>Steep bottom slope $\tan \beta &gt; 1/10$</td>
<td></td>
</tr>
</tbody>
</table>

Longshore Current Velocity

The longshore current velocity distribution across the surf zone has been of interest for many years, and some data reported in Japan in the 1950s are given in Fig. 10 (Shimano, Hom-ma, Horikawa and Sakou, 1957). In 1969, Bowen applied the radiation stress concept to determine the longshore current velocity distribution under simplified conditions. In 1970, Longuet-Higgins presented another solution of the same problem, in which he introduced a parameter defined by

$$P = \frac{\pi N \tan \beta}{\gamma c_f}$$

(4)
where \( N \) is a dimensionless constant, \( \tan \beta \) the bottom slope, \( \gamma = H/h \), \( H \) the wave height, \( h \) the water depth, and \( c_f \) a constant frictional coefficient. It is thus seen that the parameter \( P \) is a nondimensional parameter representing the relative importance of horizontal mixing. Watanabe (1973) took the relation \( c_f = 0.15 \tan \beta \) obtained by Longuet-Higgins (1972) and compared the available data for the mean longshore current velocity and the result given by Longuet-Higgins (1970). Figure 11 shows the outcome of his computation and indicates fairly large values of \( P \) compared with the values 0.1 to 0.4 suggested by Longuet-Higgins. Following the above procedure, Sasaki (1974) analyzed a number of data obtained in the laboratory as well as in the field, and plotted the value of \( P \) as a function of \( R = H \cdot V/v \), where \( H_b \) is the breaker height, \( V \) the mean longshore current velocity, and \( \nu \) the kinematic viscosity (Fig.12). The scattering of the data is fairly large, but there is a clear tendency.

Recently, Komar (1975) recalculated the longshore current velocity distribution by including the effect of wave set-up and introduced the following relation,

\[
\frac{\tan \beta \cdot \cos \alpha_b}{c_f} \approx \text{constant} \tag{5}
\]

where \( \alpha_b \) is the incident wave angle at the breaking point. Table 4 gives the values of \( P \) evaluated by Komar (1975). That is to say, the Komar values of \( P \) are of the order of \( 10^{-1} \), while the Watanabe and Sasaki values of \( P \) are of the order of 1. The above discrepancy may be due to the reasons outlined below.

The first reason to be mentioned is the method used in estimating the frictional coefficient \( c_f \). Komar introduced the effect of \( \cos \alpha_b \), but Watanabe and Sasaki did not include it. As for the laboratory data, the term \( \cos \alpha_b \) may have a certain influence on the value \( \tan \beta / c_f \), but the evaluated values of \( P \) are rather consistent with the Longuet-Higgins result. On the other hand, for the field data, \( \alpha_b \) is normally very small, thus \( \cos \alpha_b \approx 1 \). Nevertheless, \( P \) takes values on the order 1, differing from the laboratory data. The second reason is that the theoretical treatment by Longuet-Higgins is limited to the case of a uniformly sloping beach; therefore there should be a restriction in applying the result to the field observation data because of the existence of bars and steps. The third reason is the scale effect on breaker characteristics between the laboratory and the field (Führlöster, 1970). The relative intensity of turbulence due to wave breaking in the field seems to be large compared with that in laboratory. The fourth reason is that the velocity of the alongshore current is not purely the longshore current velocity, but the complicated nearshore current velocity. This means that the theoretical formula of Longuet-Higgins cannot be applied to the field in the strict sense.

<table>
<thead>
<tr>
<th>( \tan \beta )</th>
<th>( N )</th>
<th>( c_f )</th>
<th>( \tan \beta / c_f )</th>
<th>( P = \pi N \tan \beta / \gamma c_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.050</td>
<td>0.005</td>
<td>0.0083</td>
<td>6.02</td>
<td>0.118</td>
</tr>
<tr>
<td>0.100</td>
<td>0.001</td>
<td>0.0183</td>
<td>5.46</td>
<td>0.022</td>
</tr>
<tr>
<td>0.052</td>
<td>0.003</td>
<td>0.0179</td>
<td>5.59</td>
<td>0.066</td>
</tr>
<tr>
<td>0.005</td>
<td>0.019</td>
<td>0.0169</td>
<td>5.92</td>
<td>0.116</td>
</tr>
<tr>
<td>0.010</td>
<td>0.0137</td>
<td>7.30</td>
<td>0.287</td>
<td></td>
</tr>
<tr>
<td>0.150</td>
<td>0.005</td>
<td>0.0250</td>
<td>6.00</td>
<td>0.255</td>
</tr>
</tbody>
</table>

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Simulation of the Nearshore Current Field

Nearshore currents are wave-induced currents, and the characteristics of incoming waves are needed in order to predict them. However, we have very limited knowledge about incoming waves, especially the long period waves which play the important role in determining the rip current spacing. Therefore, at present, we can only treat the nearshore current phenomena as a steady one induced by approximately regular waves. In this section the treatment by Sasaki (1975) will be introduced.

The basic equations are

\begin{align}
\frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} &= -g \frac{\partial \zeta}{\partial x} + M_x - F_x \\
\frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} &= -g \frac{\partial \zeta}{\partial y} + M_y - F_y \\
\frac{\partial}{\partial x} [u(\zeta + h)] + \frac{\partial}{\partial y} [(\zeta + h)] &= 0
\end{align}

where \( F_x, F_y \) are the frictional terms, \( M_x, M_y \) the forcing terms expressed by radiation stresses, and other quantities are defined in Fig.12. Because the nonlinear terms on the left-hand side of Eqs. (6) and (7) are negligibly small (Sasaki, Horikawa, and Hotta, 1976), the basic equations (6), (7) simplified by

\begin{align}
\frac{\partial \zeta}{\partial x} &= M_x - F_x \\
\frac{\partial \zeta}{\partial y} &= M_y - F_y
\end{align}

The forcing terms \( M_x \) and \( M_y \) are given by

\begin{align}
M_x &= -\frac{1}{\rho(\zeta + h)} \left( \frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right) \\
M_y &= -\frac{1}{\rho(\zeta + h)} \left( \frac{\partial S_{yx}}{\partial x} + \frac{\partial S_{yy}}{\partial y} \right)
\end{align}

The radiation stress tensor is expressed by

\begin{align}
S &= \begin{pmatrix} S_{xx} & S_{xy} \\ S_{yx} & S_{yy} \end{pmatrix} \\
S &= \mathcal{E} \begin{pmatrix} n(1 + \cos^2 \alpha) - \frac{1}{2} & \frac{1}{2} \sin 2\alpha \\ \frac{1}{2} \sin 2\alpha & n(1 + \sin^2 \alpha) - \frac{1}{2} \end{pmatrix}
\end{align}

where

\begin{align}
\mathcal{E} &= \frac{1}{8} \rho g H^2 \\
n &= \frac{c_g}{c} \quad (15)
\end{align}

and \( \rho \) is the fluid density, \( c_g \) the group velocity, \( c \) the wave celerity, \( \alpha \) the incident wave angle, \( H \) the wave height and \( g \) the gravitational acceleration.

The frictional terms are

\begin{align}
F_x &= \frac{f_w Hu}{(\zeta + h) T \sinh (2\pi h/L)} (16)
\end{align}
\[ F_y = \frac{f_w H v}{(\tau + h) T \sinh (2\pi h/L)} \] 

where \( f_w \) is the wave friction factor defined by Jonsson (1966) and is a function of the roughness length \( k \) and the horizontal amplitude of water particle movement along the bottom boundary, \( a_b \). According to Jonsson (1976), \( f_w \) is given by

\[
f_w = 0.30, \text{ for } a_b/k < 1.57
\]

\[
\frac{1}{4f_w} + \log_{10} \frac{1}{4f_w} = -0.08 + \log_{10} \frac{a_b}{k}, \text{ for } a_b/k > 1.57
\]

In order to proceed with the numerical calculation, we will introduce the transport stream function \( \psi \) defined by

\[
\frac{\partial \psi}{\partial y} = -uh
\]

\[
\frac{\partial \psi}{\partial x} = vh
\]

where \( \zeta << h \) is assumed.

Eliminating \( \zeta \) from Eqs. (9) and (10) and substituting Eqs. (19) into the obtained one, we can find the following equation (Noda, 1972):

\[
\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} + \frac{1}{F} \frac{\partial F}{\partial y} \frac{\partial \psi}{\partial y} + \frac{1}{F} \frac{\partial F}{\partial x} \frac{\partial \psi}{\partial x} = g \left\{ \frac{1}{h} \left( \frac{\partial \overline{S}_{xx}}{\partial x} + \frac{\partial \overline{S}_{xy}}{\partial y} \right) - \frac{\partial}{\partial x} \left\{ \frac{1}{h} \left( \frac{\partial \overline{S}_{xy}}{\partial x} + \frac{\partial \overline{S}_{yy}}{\partial y} \right) \right\} \right\}
\]

where

\[
F = \frac{f_w H}{h^2 T \sinh (2\pi h/L)}
\]

\[
\overline{S}_{xx} = H^2 \left( \frac{3}{16} \cos^2 \alpha + \frac{1}{16} \sin^2 \alpha \right)
\]

\[
\overline{S}_{yy} = H^2 \left( \frac{8}{16} \sin^2 \alpha + \frac{1}{16} \cos^2 \alpha \right)
\]

\[
\overline{S}_{xy} = \overline{S}_{yx} = \frac{H^2}{16} \sin 2\alpha
\]

In order to solve Eq. (20), the value of the roughness length \( k \) must be determined. Sasaki used trial and error to obtain the optimum value of \( k \) required for fitting the calculated current field with the observed one, and found it to be 5cm to 10cm for the Ajigaura sandy beach. This value of \( k \) has the same order of magnitude as those derived by Jonsson et al., (1974).
As an example of a numerical computation, we will take the case of BACS-731214 on the Ajigaura coast. The breakers had a height of 1.2m to 1.3m and period of 8 sec. The wind direction was SSW to WSW with a speed of 2.5m/sec. Figure 13 gives the computed area, the size of which is 400m alongshore and 600m offshore. The mesh point interval was selected to be Δx=10m. Figure 14 shows the bottom contours used as an input for the numerical calculation. Figure 15 displays the wave height distributions and the breakerline location obtained numerically. Through these computations we can recognize that the above result agrees well with the picture given in the lower part of the figure.

Figure 16 shows the computed transport stream lines, from which the flow pattern is clearly seen. The obtained flow pattern agrees fairly well with the observed one. However, it is recognized that the rip current direction seems to be strongly influenced by the functions of wind and tidal current. Figure 17 gives the comparison between the predicted current velocities and the observed ones. From this result we realize that if the roughness length is selected appropriately, the computed current velocity agrees with the actual one.

As a result of our experience, the lateral frictional term should be included in numerical computations to produce more realistic features of the nearshore current system, but wave set-up does not seem to have a strong influence on the nearshore current pattern.

**Long Period Waves in the Nearshore Area**

Since 1973, we have devoted great effort to measure the long period waves appearing in the nearshore area. In this section a portion of the recent results will be introduced. A field observation was made August 29, 1976 on the Ajigaura beach, Ibaragi Prefecture. On this coast there is a 200m long observation pier pertaining to the Public Works Research Institute, Ministry of Construction, Japan.

Figure 18 is a picture taken by STEREO BACS and shows the measuring pole sites. The water surface fluctuation at eleven sites was observed simultaneously for 40 min at one second intervals by using five 16mm memomotion cameras. Figure 19 is a hydrographic chart of the site. The breaker height and wave period were about 1.2m and 13 sec, and the width of the breaker zone was about 100m. The wind speed and direction were about 3m/sec and ESE respectively. The mean bottom slope up to a water depth of 2m was about 1/60.

Figure 20 shows the power spectra at sites H and K. Site H is in the vicinity of the breaking point, therefore its spectrum can be taken to represent the incident waves. The predominant wave periods are 7, 9, and 20 sec. Site K is in the swash zone, and the wave spectrum for wave periods less than 20 sec. decreases uniformly, but that for 40 sec is amplified. Figure 21 is a spatial variation of the energy spectrum density from offshore to onshore. From this diagram it is clearly seen that the frequency components higher than 0.2 Hz (T=5sec) observed outside the surf zone are damped very rapidly inside the surf zone, and those for 0.05 Hz (T=20 sec) to 0.2 Hz (T=5 sec) disappear in the swash zone, while long period waves of frequency less than 0.05 Hz (T=20 sec) cannot be seen outside the surf zone. Based on the above facts, we understand that long period waves are amplified in the surf zone by receiving wave energy from the short period waves. It should be remarked that site H was in the rip current, therefore a long period oscillation could be observed.

Ball (1967) obtained the cut-off frequencies of edge waves on the bottom profile expressed by
\[
\frac{h}{h_\infty} = 1 - e^{-\alpha x}
\]  \hspace{1cm} (22)

where \( h_\infty \) and \( \alpha \) are constant. These frequencies are

\[
\nu = \frac{\alpha \sqrt{gh}}{2\pi} \\sqrt{n(n+1)}
\]  \hspace{1cm} (23)

where \( n \) is an offshore modal number and \( g \) the acceleration due to gravity. In Fig. 22, our data are plotted together with Huntley's data (1976).

Fujinawa and Okada (1976) presented the following relationship between the surf beat wave height \( H_{1/3}^s \) and the gravity wave height \( H_{1/10}^s \) based on their data obtained for typhoon-generated waves with maximum wave height \( H_{1/3}^f \approx 3m \) and period \( T_{1/3} \approx 15 \text{ sec} \) and that of Goda (1975),

\[
H_{1/3}^s = 0.23 \frac{H_{1/10}^s}{h}
\]  \hspace{1cm} (24)

where \( h \) is the water depth. Table 5 is the calculated surf beat wave height \( H_{1/3}^s \) under various conditions of gravity wave height and water depth. That is to say, at the water depth \( h=1m \), \( H_{1/3}^s \) reaches to 0.65m for \( H_{1/10}^s=2m \), and 1.2m for \( H_{1/10}^s=3m \). From this fact, long period waves appear to be amplified in the surf zone, and exert a strong influence on the nearshore dynamic system.

**Table 5** Calculated surf beat wave height (after Sasaki, 1977).

<table>
<thead>
<tr>
<th>( H_{1/10}^s ) (m)</th>
<th>( H_{1/3}^s ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h=1m )</td>
<td>( h=2m )</td>
</tr>
<tr>
<td>1</td>
<td>0.23</td>
</tr>
<tr>
<td>2</td>
<td>0.65</td>
</tr>
<tr>
<td>3</td>
<td>1.2</td>
</tr>
<tr>
<td>4</td>
<td>1.8</td>
</tr>
<tr>
<td>5</td>
<td>2.6</td>
</tr>
</tbody>
</table>

At present, the directional wave spectrum of incident waves is still unclear; therefore the prediction of the predominant infragravity wave period is not possible.

**APPLICATION TO ENGINEERING PROBLEMS**

As can be inferred from the previous sections, the understanding of nearshore dynamics has expanded very rapidly and widely during the last ten years. Therefore these results are gradually being applied to various engineering problems. In the following, examples of such engineering applications will be briefly described.
Marine Recreation

The most typical marine recreational activity is sea bathing, and the amount of area required for sea bathing purposes is steadily increasing. Regarding the places where swimming is not allowed, the generation of strong rip currents is commonly observed. From that point of view, nearshore current observations are essential for the location and establishment of new bathing places. If we possessed sufficient knowledge, we would be able to control the generation and characteristics of these rip currents.

Heated Water and Sewage Discharged into the Nearshore Zone

Heated water from steam or nuclear power stations is normally discharged to the surf zone in Japan, and similarly for treated sewage. Therefore, from the environmental perspective, the mixing and diffusion processes of the heated or polluted water in the surf zone become of great interest to coastal engineers. Reflecting the above circumstances, we are performing field observations on the flow pattern and water temperature distribution in the vicinity of the outlet at the Fukushima No.1 nuclear Power Station of the Tokyo Electric Company, Ltd.

Figure 23 shows one detailed picture of the surface water temperature distribution compiled from infrared images, from which the interaction of the discharged plume and the nearshore circulation can be clearly seen. Such an interaction appears in the mixing process of heated water with surrounding water and in the shoreline topography.

Sedimentation

In the main part of the above discussions, we assumed that the bottom topographies are given. But in reality the sediment particles in the nearshore area are actively driven by currents and wave action. Therefore the bottom topography must be a time-dependent function, and the currents must not be stationary.

At present we are able to calculate the wave-induced current field fairly satisfactorily, but we are unable to predict quantitatively the bottom topography change. The main reason is that the precise relation for the rate of sediment transport is not known. Indeed, various structures have been erected along the coast for numerous purposes, and have altered the natural beaches artificially. Due to this fact, the long range prediction of coastal changes caused by a coastal structure is demanded by the public. Therefore, the solution of the above problem is today one of the most important tasks for coastal engineers.

SUMMARY AND CONCLUSIONS

Five years have passed since the review of recent progress in the study of longshore currents by Longuet-Higgins. In the present paper the writer has reviewed recent work on nearshore currents performed mainly by the Coastal Engineering Research Group at the University of Tokyo, and discussed briefly their engineering significance.

Firstly, field observation techniques were described capable of measuring the nearshore current velocity field spatial distribution simultaneously with the wave field.
Secondly, the pattern of nearshore currents, spacing of rip currents, generation mechanism of rip currents, three domains of the nearshore current system, and the longshore current velocity were discussed. During the past five years, great progress has been made in the study of rip current spacing based on field observations; however, the mechanism controlling rip current spacing is still unclear in part, and more complete field data are needed. As for the treatment of the longshore current velocity by Longuet-Higgins, several restrictions for its application in the field were pointed out.

Thirdly, nearshore current numerical models for engineering use were described. Present modeling assumes the phenomena be steady, and lateral mixing is neglected. To obtain more precise agreement with the current pattern, lateral mixing effects should be estimated and included in the model.

Lastly, three examples of engineering applications, as related to marine recreation, environmental problems, and sedimentation were briefly discussed. For the prediction of the nearshore current field, complex problems such as the wave-wave interaction, wave-current interaction and interaction between waves, currents and bottom topography should be clarified, particularly in connection with sediment problems.

ACKNOWLEDGEMENTS

First of all, the writer would like to express his sincere thanks to the Organizing Committee of POAC 77 in Newfoundland, Canada for presenting him with an excellent opportunity to give a talk at the Conference. The writer is also indebted to Dr. T. Sasaki and Dr. N.C. Kraus at the Nearshore Environment Research Center, Tokyo, Japan for their assistance in preparing the manuscript of this paper.

REFERENCES


Fig. 1 Schematic diagram of the nearshore current system, [after Shepard and Inman, 1950]

Y_r, X_r, and X_b are the rip current spacing, width of the surf zone, and offshore extensions of the rip current from the breaker line, respectively.
Fig. 2 Schematic diagram of the synchronized helicopter system (SIHELS). [Horikawa and Sasaki, 1972]

Fig. 3 Schematic diagram of the balloon camera system (BACS). [Horikawa and Sasaki, 1972]

Fig. 4 Schematic diagram of the stereo-balloon camera system (STEREO-BACS). [Sasaki, Horikawa and Hotta, 1976]
SYMMETRICAL CELLULAR

ASYMMETRICAL CELLULAR (MEANDER)

ALONGSHORE SYSTEM

Fig. 5 Nearshore current pattern. [Harris, 1969]

RIPS-730215

$H_b = 0.6 - 1.0 \text{ m}$

$T = 7 \text{ sec}$

Fig. 6 Observed rip current spacing (RIPS-730215). [Sasaki and Horikawa, 1975]
Fig. 7 Location map of filed observation sites. [Sasaki, 1974]

Fig. 8 Correlation between $Y_r$ and $X_b$.
[Sasaki, 1974, 1975]
Fig. 9 Three domains of nearshore current generation.  
[Sasaki, 1974, 1975]
Fig. 10 Offshore distribution of longshore current velocity measured in a laboratory basin; \( u \) and \( U \) are respectively the measured values and computed values using Putmum, Munk and Traylor's formula. [Shimano, Hom-ma, Horikawa and Sakou, 1957]
a) Estimation of $P$ by longshore current velocity data and Longuet-Higgins's (1970) formula. $c=0.15 \tan \beta$ is assumed. [after Watanabe, 1973]

b) Relation between $P$ and $R_e$. [Sasaki, 1974]

Fig. 11 Evaluation of the parameter $P$. 

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Fig. 12 Definition sketch for the nearshore current computation. [Noda, 1972]
Fig. 13 Computed area for the near-shore current. [Sasaki, 1974, 1975]

Offshore topography sounded on March, 1972 in meters.
Nearshore topography sounded on Dec 11th, 1973 in meters (not shown).

Fig. 14 Bottom contours in the computed area.
[Sasaki, 1974, 1975]
Fig. 15 Comparison between computed and observed surf zone wave height distributions. [Sasaki, 1977]
Fig. 16 Computed transport stream function, $\Psi$, (m³/sec). [Sasaki, 1974, 1975]

Fig. 17 Comparison between predicted and observed nearshore current velocities on a shoal. [Sasaki, 1974, 1975]
Fig. 18 A picture of an observed area (BACS-760829). Letters indicate the location of poles for the water surface fluctuation measurement. [Sasaki, Horikawa and Kubota, 1977]

Fig. 19 Hydrographic chart (INFR-760829). [Sasaki, Horikawa and Kubota, 1977]
Fig. 20 Power spectra of incident waves near the breaking point (H), and near the shoreline (K), [Sasaki, Horikawa and Kubota, 1977]

Fig. 22 Relation between peak frequency $\nu$ and offshore model number of cut-off edge waves. [Sasaki, Horikawa and Kubota, 1977]
Fig. 21 Onshore-Offshore transformation of wave energy density observed along the pier. [Sasaki, Horikawa and Kubota, 1977]
Fig. 23 Surface water temperature distribution in the vicinity of a nuclear power station, obtained from infrared photography. [Horikawa, Lin and Mizuguchi, 1977]
SYNOPSIS

At present, theoretical formulae, laboratory testing of small samples, model testing with artificial or natural ice, or mathematical models are not producing reliable results for a practical design as yet. Analysis of old and new structures exposed to ice thrust provides valuable information about actual ice forces, especially in cases where complete or partial failures occurred.

In this paper, stability and some failures of sixty-nine old and new lighthouses and lightpiers are analyzed. The results of this analysis are summarized in three tables.

INTRODUCTION

During the last decade great interest has been shown in establishing the actual ice forces acting on structures. Numerous laboratory tests on small ice samples have been carried out for a long time and much data has been obtained. A considerable amount of valuable research has been done about the properties of ice, but so far not too much correlation has been made with practical experience. Because of the great heterogeneity of ice in field conditions, the results obtained on small ice samples cannot be applied directly to designs. A relation between ice strength found in the laboratory and the effective ice strength against structures cannot be established as yet.

Laboratory modelling with artificial ice has been carried out for more than a decade in Canada, but here, as in other countries, these laboratory tests cannot reproduce all characteristics of natural ice and the complexity of ice-structure interaction in the field.

Numerous mathematical models have been proposed and carried out. However, without actual reference data representing field conditions, they do not solve the design problems, although they are valuable to indicate the trends of effects of the ice forces.

At present the prevailing opinion is that direct field measurements of the effective forces on structures are necessary for any correlation of theoretical and actual forces. For reliable conclusions, measurements over a long period are required, but, so far, only a few scattered measurements have been made. The
measurements of ice forces on structures is difficult and expensive (Danys, 1975, Offshore installations...), and it is not likely that enough of these will be made in the near future.

Analysis of the stability of structures which have been exposed to ice forces provides valuable information, especially if the complete failure of a structure has occurred. Of course, results of such analysis are only broad guidelines for a designer. Lightpiers, as other offshore structures, are individual structures and their satisfactory or unsatisfactory performance depends not only on the design ice force but also on many other design features as shape and local environmental conditions.

The winters in Canada are rather severe and it was acknowledged that ice thrust was one of the main forces to be resisted by a lightpier (Fig. 2, 3, 8 and 15). But for many years, very little was known about the magnitude of ice forces. Experience and judgement were the principal factors in the design and construction of such unique structures as lighthouses in the middle of a lake. Even now our knowledge of these forces is not complete.

Timber cribwork for the substructures of the lighthouses and other marine structures was used in Canada for many years (Danys, 1974). This flexible type of structure somehow withstands rather large ice forces without completely failing but deteriorates rather fast. Concrete and steel replaced timber cribwork about 15-20 years ago.

In this paper the stability of 69 lightpiers in the St. Lawrence Waterway between the Saguenay River and Sault Ste. Marie, Lake Superior, is analyzed (Fig. 1 and Tables 1, 2 and 3). Twelve of these lightpiers were built between 1902 and 1935 and fifty-six between 1953 and 1976.

For detailed analysis of the individual lightpiers all this waterway is divided into three reaches: 1. the Lower St. Lawrence River, from the mouth of the Saguenay River to Montreal, 2. the Upper St. Lawrence River, from Montreal to Lake Ontario, and 3. the Great Lakes.

In this paper only short references are made to the lightpiers which had been described previously in other publications.

EVALUATION OF THE STABILITY OF STRUCTURES

Often full information is lacking for the retroactive calculation of the stability of the failed structure. The absence of information about the foundations of old structures is usually the greatest problem. Even now, the evaluation of soft clays, for example, is rather approximate and high safety factors are used in design. A few decades ago the technique of investigating soil foundations and establishing their strength was not yet adequately developed.

Almost always there is no reliable information about the thickness of the ice which acted on the structure. Therefore, although it is often possible to calculate the value of the total ice force which caused deformation or collapse of the structure and to relate it to the unit width of the structure, i.e. kg/m or t/m (1000 lb/lin. ft), it is difficult to reliably express it for a unit area as kg/cm² (p.s.i.). Where no recording of ice thickness was available, the design value of the ice thickness was based on observations by the navigation and engineering staff and compared with the theoretical probable ice thicknesses (Carter, 1977).
Other factors which are difficult to assess are the water level at which the critical ice force was acting and modes of ice and foundation failures.

Investigations and testing of soil foundations for the lightpiers analyzed in this paper have been made mostly within the last fifteen years, and, generally, they were as complete as the up-to-date technique made possible.

For structures built during the last 15 years, full design data was available. However, for those structures built 40-70 years ago, design data was not available. The stability calculations of these lightpiers were based on the available old drawings, assuming that the structures had been built as is shown on these drawings.

In the summary (Tables 1, 2 and 3) of the apparent failure forces, the smallest ones probable have been given.

Calculation of ice forces on the structures basically followed the formulae adopted by the Marine Aids Division of the Canadian Coast Guard described by Neill (1976), Danys and Bercha (1976) and Danys, Bercha and Carter (1976). In the last two publications influence of slope angle and friction between ice and structure is analyzed.

LIGHTPIERS IN THE LOWER ST. LAWRENCE RIVER

Lighthouses near the Saguenay River

Two major lighthouses at White Island and Prince Shoal near Tadoussac, P.Q., are 160 and 175 km downstream from Quebec City near the mouth of the Saguenay River. White Island lighthouse, built in 1955 (Danys, 1975, Effect of ice) is founded on clayey till. Prince Shoal, built in 1963, is founded on firm sandy till. The substructures of both are conical in shape with slopes of 45° or close to it.

Prince Shoal lighthouse (Fig. 5) is larger, is built in deeper water and has a helicopter platform, and a very strong light, as well as other navigation aids. Lightkeepers stay on this lightpier all winter (Danys, 1965). The design ice forces and safety factors for Prince Shoal lightpier were high, higher than is assumed for the presently built lightpiers which are automated and without lightkeepers.

Ile aux Coudres Lightpiers

Old Lightpier - The old major lighthouse, about 80 km east from Quebec City, was built on the edge of a shoal joining the shore in 1930. The substructure was an octogonal pyramid with a very wide base (Fig. 6). It was built as a gravity structure on sandy till containing many cobbles and boulders. Its overall stability was great but because of the disintegration of concrete and the underscore of the foundation, as well as obsolescence, it was replaced by a new one, fully automated and remotely controlled in 1969 (Danys et al. 1975).

New Lightpier - The substructure of the new lightpier is conical (Fig. 7 and 8) with 45° sides. The design ice forces were larger than for the Brule Bank lightpier, namely 162 t/m (108,000 lbs/lin.ft) and the design effective thickness of the ice sheet acting on the structure was 91 cm (3 ft). There is no vibration of the substructure of this lightpier.
Brule Bank Lightpiers

Old Lightpiers - Two major lightpiers were built at Brule Bank, about 50 km downstream from Quebec City, in 1930. They were built on a "wandering" sand shoal, the upper layer of which is silty sand from 1.5 to 3.5 m thick. Depending on the size of the storm waves and on the direction of the wind, this shoal changes its shape as well as the thickness of the upper layer; thus, at low tide the lightpiers may be on the dry shoal or in 0.5-1.5 m of water.

Both lightpiers were founded on timber piles but the substructures had different shapes and sizes.

The front lightpier was similar to a bridge pier; it was elongated in the direction of the river stream (Fig. 9 and 10). Its substructure, surrounded by vertical steel sheetpiling, was vertical to the level of a high tide. This lightpier withstood ice forces for 35 years and had to be replaced because of gradual deterioration of steel sheetpiling and concrete at the high tide level. But the lightpier always vibrated extensively when hit by ice floes, especially when ice floes were hitting its broadside. This, apparently, was caused by the lateral movement of the tops of the timber piles.

The substructure of the lower lightpier had sloped sides and the base was larger. No vibration was felt on this lightpier. However, repairs had to be done to the damaged and disintegrating concrete as well as to protect the lightpier from undermining by waves and currents.

New Lightpiers - The circular substructures of both the new upper and lower lightpiers at Brule Bank (Fig. 11 and 12) are the same size. An analysis was made of the stability of the old front lightpier and it was concluded that it could withstand an ice force of 195 t/m (130,000 lbs/lin.ft) in the main direction and 90 t/m (60,000 lbs/lin.ft) in the direction of the weakest resistance (which was at a 30° angle with the main axis of the structure).

The selected design ice force for the new lightpiers was to be 112 t/m (75,000 lbs/lin.ft); considering the minimum safety factor of 2, the failure ice unit force would be 225 t/m (150,000 lbs/lin.ft). The effective ice thickness was assumed to be 91 cm (3 ft).

The side walls of the substructure are vertical to make it easier to dock a service boat. However, a slight vibration of the lightpier is felt when it is hit by ice floes. This is attributable to the fact that some 2.5 m of piles are in loose sand.

Lac St. Pierre

The navigation channel across the 32 km long and 13 km wide lake has seven bends or curves (Fig. 1). As early as 1816 a lightship was placed at Curve No. 2 and later two more were added at Curves No. 1 and 3.

Old Lightpiers at Curve No. 2 - In 1905 three lightpiers were built at Curve No. 2 for two ranges replacing a lightship; Fig. 13 shows the front lightpier of both ranges.

The front lightpier completed in 1905 was put out of service by ice at the end of the same winter in 1906. Under ice pressure the 15.25 m long crib (Fig. 13) tilted
3 m in the downstream direction. Then the superstructure and concrete of the sub-
structure, which was similar to that one shown in Fig. 16, were removed and a con-
crete cap was built to a level about 1.2 m above the low water level. A removable
steel tower with a superstructure was placed on the concrete pier in spring when
the ice was gone and removed before the beginning of winter. The same arrangement
was used for the front lightpiers at Curve No. 3, first built in 1910 and replaced
in 1935 (Fig. 17) (Danys, 1975, Lightpiers on very weak foundation).

Complete design and construction drawings are not available for the lightpiers
built at that time. The exact length of the timber piles for the old front lightpiers at Curve No. 2 (Fig. 13) and Curve No. 3 (Fig. 16) are not known and some
assumptions had to be made.

The two old rear lightpiers at Curve No. 2 were built on square timber cribworks,
12.2 x 12.2 m in size each, which were supported by 128 timber piles. Their dia-
meter at the butt was 30-36 cm and their length - 12 m (Danys, 1975, Lightpiers on
very weak foundation). These two lightpiers were built in water 1.8 m deep (at low
water level) and they were in operation more than 60 years. During the sixty years
the lightpiers tilted 75 cm and settled more than 90 cm. Because of their general
deterioration, they were replaced in 1969. In this paper the stability of these
two lightpiers as well as the stability of two front lightpiers which failed at
Curves No. 2 and 3 was calculated for the original design conditions.

Old Lightpiers at Curve No. 3 - Curve No. 3 is near the village Pointe du Lac; often
the lightpier at this location is called Pointe du Lac lightpier.

The first lightpier for a front light of the Pointe du Lac navigational range was
completed in 1907 (Fig. 16). Its general design was the same as the design of the
lightpiers at Curve No. 2 (Fig. 13). But it was built in water 6.4 m deep so that
the timber cribwork was larger, namely, 15.25 x 15.25 m, and the substructure,
including the cribwork, was higher. At the end of the same winter, in the spring of
1908, ice tilted the lightpier so badly that it had to be abandoned.

Another lightpier was built 550 m closer to the shore in a depth of 1.8 m at low
water. As was mentioned before, the substructure was made low so that ice could
float over the top of the substructure, exerting little or no ice thrust on the
lightpier (Danys, 1975, Effect of ice and waves).

This lightpier was in operation until 1935 when it was abandoned because the ice
thrust gradually displaced the top concrete slab of the substructure about 5.1 m.
Then a new one of a similar design (Fig. 17) was built just behind the old one and
was in operation until 1965 when the top concrete slab was pushed off the cribwork
for 11.6 m.

The edges of the tapered concrete slab broke off leaving a vertical face about 60 cm
high exposed to ice thrust. At low water level ice floes did not slide over the
concrete cap but exerted a horizontal force against the deteriorated edge. No great
ice force was required to displace the concrete top because it was not firmly
attached to the cribwork.

New Lightpiers - In the sixties the need for fixed navigation aids - lightpiers - in
Lac St. Pierre for icebreakers and other ships operating in winter or even at the
beginning and the end of winter became more obvious. A program of construction of
a number of the lightpiers in the lake was started in 1966, but studies of suitable
structures were being made for several years before because of the very difficult

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conditions in the lake. Besides large ice forces, the problem of very weak foundation had to be overcome (Danys, 1971, Lightpiers on friction piles ...).

Generally, the bottom of the lake is covered with very soft clay 45-90 m thick. A new type of lightpier had to be developed (Fig. 14, 15 and 18). In the design concept it has been considered that the lightpier may settle unevenly and move several inches laterally. The superstructure was designed to be light and adjustable for tilting (Danys, 1975, Lightpiers on very weak foundation).

First two lightpiers were built in 1966 at Curve No. 2 for the rear lights of the two ranges. Further, two lightpiers were built for the front lights at Curve No. 2 in 1969 (Fig. 14). In 1972 two lightpiers were built for the Western Range of Yamachiche Bend and one lightpier at Curve No. 1; in 1973 another lightpier was built at Curve No. 3 (Fig. 18).

For the new lightpiers in Lac St. Pierre the design compressive ice force has been assumed to be 14.1-17.6 kg/cm² (200-250 p.s.i.). For the first two lightpiers the effective ice thickness was assumed to be 60 cm because at that time the known measurements indicated the maximum ice thickness of 45-50 cm. However, measurements made in later years showed that the maximum ice thickness may reach 80 cm, and the design ice thicknesses for the later lightpiers were increased 90 cm.

Ile aux Vaches - A new lightpier was built in 1975 (Fig. 4) to replace an old front lightpier of a range because erosion by the strong currents started to undermine the old structure.

**LIGHTPIERS IN THE UPPER ST. LAWRENCE RIVER**

**Lake St. Francis and Lac St. Louis**

The glacial till deposits are in many areas erratically covered with very soft marine clays reaching thicknesses of approximately 35 m. The glacial tills are very firm foundation, approaching a hardpan in their properties. But the clays of the marine deposit are a very weak foundation requiring piles.

**Lake St. Francis Cribwork Structures** - In 1958 four small lightpiers of the type shown in Fig. 19 were built in 1, 2 and 3 m depth of water. The main part of the substructure was a timber cribwork filled with stones and capped with a concrete slab. Where a soft foundation was encountered, eight timber poles were driven 12 m deep or to refusal which was 5 and 8 m. Soon three of these lightpiers sustained various structural damages, such as tilting and displacement. A stability analysis showed that they could resist only very low ice thrust, i.e., 13.5-16.5 t/m (9,000-11,000 lb/lin.ft). The structures were modified by adding more piles and changing the flat slab to a smaller cylinder. The resistance to the ice thrust was increased to 30-46 t/m (20,000-31,000 lb/lin.ft) but later all of them had to be replaced by the conical concrete lightpiers designed for much larger ultimate ice thrust, i.e., 168 t/m.

**Conical Concrete Substructures** - In 1966 five lightpiers of the type shown in Fig. 20 were built in Lake St. Francis in locations where the depth of the soft marine clay was from 20 to 35 m deep. The main part of the substructure of the lightpier was designed as a cone as such a shape more effectively breaks the floating ice and permits the building of a structure of smaller dimensions and so reduces construction costs (Danys, 1971, Effect of cone-shaped structures). Further four lightpiers were built in 1973-74.
In 1967 in Lake St. Louis four lightpiers were built of the same type except that they were founded on the firm glacial till not requiring any piles. These thirteen piers have not suffered any damage or deformation. Fig. 15 shows ice breaking against one of these lightpiers of the conical shape which has been widely used for Canadian lightpiers in the last fifteen years.

St. Lawrence River International Rapids Section

Six of these concrete lightpiers were of the type shown in Fig. 21 built in 4.5 to 8.5 m depths of water on firm glacial till. Their longitudinal axes are located in the direction of the flow, and the upper part exposed to the ice pressure is made of minimum dimensions required for the placing of the small light towers. The stability of these piers in the direction of the current is much larger, roughly about four times as much, than in the lateral direction (Table 2). After 12 years, one, and later the other one, of these lightpiers were overturned by the lateral ice thrust. Under exceptional conditions the winds may propagate large ice floes against the long side of the lightpier which may introduce impact force exceeding the stability force. The calculated ultimate ice forces which these six piers can withstand are summarized in Table 2.

St. Lawrence River, Brockville Narrows - Concrete Piers

In 1959 six concrete lightpiers of the type shown in Fig. 22 were built on granite rock foundation. Their stability in the lateral direction is very low (Table 2) but so far only one pier LL337 has failed in 1976. Because of the natural restrictions, there is little possibility for ice floes to move perpendicularly to the broadside of the lightpiers.

LIGHTPIERS IN THE GREAT LAKES

Lake Erie

One of the first major Canadian offshore lighthouses was built 4 miles offshore of Point Pelee in Lake Erie in 1857 (Commissioners, 1858). It should be considered that it was a remarkable structure at the time for then prevailing state of the marine construction technology.

The engineer William Scott who designed and built the lighthouse substructure introduced some novel features in the design: near circular-octagonal shape in plan and sloped sides to reduce wave and ice forces. These elements are being used today for reduction of the acting forces. William Scott had confidence in his structure and wrote in his report that the structure "appeared to be as stable as rock itself". It was until a fire 43 years later completely destroyed the tower and superstructure which were constructed of timber. The new lighthouse was built in 1902 several miles away and called Pelee Passage lighthouse.

Pelee Passage - The central part of the substructure of this lighthouse had circular masonry walls and cribwork (Fig. 23). Later a third outer ring was added to protect the main body from ice and wave forces. This ring consisted of 16 cribs having a 45° slope on the outside for reduction of ice forces. Ice, waves and decay of timber have gradually destroyed the outer cribs and started to erode the cribwork of the middle ring, although repairs have been done several times. After 70 years of service the lighthouse was replaced in 1976 with a new one (Fig. 24) anchored into rock.
**South East Shoal** - A major lighthouse was built in 1926. It seems that the degree of stability is high but the foundation, loose sand, has to be continuously protected with a heavy rip-rap (Danys, 1975, Effect of Ice and Wave Forces).

**Detroit River**

Between 1953 and 1969 four lightpiers were built at Amherstburg and at the inlet of the river to Lake Erie (Fig. 25). Livingstone Channel Entrance lightpier LL677 was founded on broken rock and ice slightly tilted it. The foundation was encased with a reinforced concrete ring and no further damage took place. During the spring break-up all ice from Lake St. Clair is evacuated at once and it causes considerable ice thrust on the lightpiers.

**St. Clair River and Lake**

In 1962 a straight navigation channel, 8 km long, was excavated to by-pass the Southeast Bend in the St. Clair River. Nine lightpiers were built to mark the channel. Lightpier LL721 (Fig. 26) in the lake was designed as a concrete tower on a square submerged timber cribwork in 4 m deep water. The other eight lightpiers were located in very shallow water, 1 to 1.5 m deep. The foundation of the lightpiers were surrounded by rock berms designed to take ice thrust (Fig. 27). The rock fill at lightpiers LL724 and LL726 settled and was eroded by the moving ice leaving the foundation exposed to the floating ice. The lightpiers would resist only 22 t/m of ice thrust instead of 119 t/m as designed, and they failed.

The new lightpiers LL724 and LL726 have conical concrete substructures (Fig. 28) and were designed for an ice force of 89 t/m and an overall safety factor of 2.

**Lake Superior**

A major lighthouse at Gros Cap, near Sault Ste. Marie was built in 1953 as a gravity structure (Fig. 29). It has the shape of a bridge pier and the stability along the minor axis is much smaller than along the major axis. It appears that local conditions are such that the ice floes cannot exert large forces against the broadside of the lightpier.

**SUMMARY OF STABILITY CALCULATIONS**

In Tables 1, 2 and 3, the results of the stability calculations of 69 lightpiers are summarized.

It has been considered that a "failure" occurred if a structure or its component failed and the whole structure or its component had to be replaced. If the structure was tilted or deformed but still usable to support the navigational aids, it has been classified as a "damaged" structure.

For the "new" lightpiers, i.e., for the lightpiers built during the last 15 years, the design values have been multiplied by the least safety factor in order to obtain "failure" forces. In most cases, the stability of the foundation was the governing factor. But it could have been, for example, the shear or bending strength of piles, or foundation resistance against sliding or overturning which gave the smallest forces.
<table>
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<th>DESIGN PRESSURES</th>
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<th>CALCULATED FAILURE PRESSURES</th>
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**NOTES**
a - Failure of top slab only,  b - Smaller figure - pressure along minor axis, larger figure - pressure along major axis
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**NOTES**

- \( x-x \) pressure along major axis,
- \( y-y \) pressure along minor axis.
TABLE 3 Design and failure ice forces of lightpiers in the Great Lakes

<table>
<thead>
<tr>
<th>No</th>
<th>DESIGNATION AND STRUCTURE</th>
<th>YEAR BUILT</th>
<th>DESIGN PRESSURES</th>
<th>SUSTAINED DAMAGE OR FAILURE</th>
<th>CALCULATED FAILURE PRESSURES</th>
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<td>22 1</td>
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<td>1957</td>
<td>No</td>
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<td>131</td>
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<tr>
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<td>No</td>
<td>32 5</td>
<td>201</td>
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<tr>
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<td>17 6 134</td>
<td>84 5</td>
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<td>9 8 60</td>
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<td>7</td>
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<td>9 8 60</td>
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<tr>
<td>15</td>
<td>Gros Cap Piers</td>
<td>Longitudinal direction</td>
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<td></td>
<td></td>
<td>Lateral direction</td>
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+ Critical force - breaking wave
++ Pressures along minor axis

TABLE 4. Applied design ice impact forces+

<table>
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<th>t/m</th>
<th>lb/in²</th>
<th>1000 lb/ft</th>
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<tr>
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<td>200 - 250</td>
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<td></td>
<td></td>
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<td>9 8 - 11 6</td>
<td>140 - 165</td>
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<td></td>
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<tr>
<td>S N 76 - 1966, overall</td>
<td>3 5 - 17</td>
<td>50 - 242</td>
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<td></td>
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<tr>
<td>Zone I Middle and South Europe</td>
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<td>142 - 242</td>
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<tr>
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<td>50 - 150</td>
<td>33 - 100</td>
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<td>150 - 200</td>
<td>100 - 133</td>
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<tr>
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<td>Finland</td>
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<td>Lightpiers, before 1972 (maximum)</td>
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<td>100</td>
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<td>contemporary design (max)</td>
<td>300</td>
<td>200</td>
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+ Danys, 1975, Effect of ice and wave forces on the design of Canadian offshore lighthouses

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For the "old" lightpiers, built before 1935, a case of the least stability of the structure or its component was taken into consideration. In some cases an exact assessment of the forces was impossible and a probable average of the low values was assumed.

As the ultimate strength of soils, timber or even concrete, varies, the calculated values are also approximate. Ice thicknesses during the failure are not known. Therefore, the ice unit force per lineal meter of the structure is more accurate than that one expressed in kg/cm².

COMMENTS

From Table 1 it seems that the failure ice pressures were at least 12.3 kg/cm² (175 p.s.i.) in the Lower St. Lawrence River. The present design pressures of 17.6 kg/cm² (250 p.s.i.) in this region are larger than the observed failure ice pressures. Of course, in design the failure force depends on the assumed safety factor which may vary from 1 to 3 depending on the available codes or on the designer's judgement.

So far, the new lightpiers in the Lower St. Lawrence River have not been damaged by ice.

A lightpier at Yamachiche Bend in Lac St. Pierre was instrumented in 1973 to measure ice force and from the measurements during the winter 1973-74 the estimated compressive ice pressure was 13.4 kg/cm² (190 p.s.i.) or 82 t/m (55,000 lb/lin.ft).

The figures in the Table 1 may be compared with similar figures in Table 2 calculated for the Upper St. Lawrence River. An analysis of 32 lightpiers built between Montreal and Lake Ontario indicated that several failed structures could withstand only 5.6-7.7 kg/cm² (80-110 p.s.i.) pressure, but no structure failed which was designed for an ultimate or failure strength of 21-28 kg/cm² (300-400 p.s.i.) of ice pressure.

So far in Canada there is no set standards for ice design forces. The Marine Aids Division of the Canadian Coast Guard now is using 17.6 kg/cm² (250 p.s.i.) compressive pressure for design of the lightpiers where ice movement is expected to take place any time during the winter; 14.1 kg/cm² (200 p.s.i.) is used where ice moves only at the beginning and the end of winter.

During the past twenty years the design ice compressive stresses from 8.4-28.1 kg/cm² (120-400 p.s.i.) were used (Danys, 1975, Effect of Ice and Wave Forces).

ACKNOWLEDGEMENT

The permission granted by Mr. G.L. Smith, Chief of the Marine Aids Division of the Canadian Coast Guard, to present this paper is gratefully acknowledged.
REFERENCES


Fig. 1. General location plan

Fig. 2. Lac St. Pierre, Curve No. 2. Ice piling against rear west lightpier in 1908
Fig. 3. Lac St. Pierre, Yamachiche Bend.
Ice piling against rear lightpier

Fig. 4. Ile aux Vache front lightpier in the St. Lawrence River built in 1975
Fig. 5. Prince Shoal lighthouse in the St. Lawrence River built in 1963.

Fig. 6. Old Ile aux Coudres lightpier in the St. Lawrence River built in 1930

Fig. 7. New Ile aux Coudres lightpier in the St. Lawrence River built in 1970

Fig. 8. Aerial view of new Ile aux Coudres lightpier
Fig. 9. Old Upper Brule Bank lightpier in the St. Lawrence River built in 1930

Fig. 10. View of old Upper Brule Bank lightpier

Fig. 11. New Upper Brule Bank lightpier in the St. Lawrence River built in 1966

Fig. 12. Aerial view of new Upper Brule Bank lightpier
Fig. 13. Lac St. Pierre, Curve No. 2. Old front lightpier built in 1905, abandoned in 1906

Fig. 14. Lac St. Pierre, Curve No. 2.
New front lightpier built in 1968

Fig. 15. Lac St. Pierre, Curve No. 2. Ice against conical substructure of the new lightpier
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Fig. 18. Lac St. Pierre, Pointe du Lac. View of new front lightpier built in 1973
Fig. 19. Lake St. Francis. Substructure of Lightpier LL 124 built in 1958

Fig. 20. Lake St. Francis. Lightpier LL 99 at McKies Point built in 1966

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Fig. 25. Detroit River lightpier
LL 642 built in 1961

Fig. 26. St. Clair Lake lightpier LL 721
built in 1962
Fig. 27. St. Clair River lightpier LL 724, built in 1962 and abandoned in 1969

Fig. 28. New St. Clair River lightpier LL 724 built in 1969
Fig. 29. Gros Cap lighthouse in Lake Superior built in 1953
INTRODUCTION

We shall not venture by whom and when concrete was first applied in marine construction. It is well known that this excellent building material has been used for decades and centuries in piers, bridgefoundations, ships/barges etc. Here, as in many other sides of practical life experience has taught new ways for improving methods and equipment. This again has led to a wider field of use and more advanced and daring design.

With the extensive coastline and close affinity to marine activities it came quite natural for Norwegian construction industry to focus their interest on marine construction. Norway's geological and marine environment favours the use of concrete in our harbours, lighthouses, breakwaters etc. Completed projects and research as well have given Norwegian construction companies and institutions attached to the trade, vast amounts of valuable data on design, construction methods and concrete technology.

The North Sea oil exploration sparked the idea for a concrete gravity structure to support a drilling deck, and quite many concepts were born on the paper. A small number, 13 structures, have been built, and they all stand in the rough waters of the North Sea. Of these 13 structures 5 are of the Condeep type, a concrete structure designed and built by Norwegian Contractors. Furthermore, the company has participated in two other structures of Howard-Doris design. Norwegian Contractors is a joint venture of the 3 leading construction companies in Norway. This paper will deal with the construction of the Condeep platform.

THE CONDEEP CONCEPT

Basically, the structure consists of a base or caisson of 19 interconnected cylinders, three (or more) of which are extended upwards as towers to support the deck (fig. 1), which could be of steel or concrete, or a combination of the two. All cells have spherical domes at the bottom and top except for the tower cells.

As seen from the figure, the cylinder wall extend downwards as a concrete skirt. From the tip of the concrete skirt a steel skirt extends still further down. Depending on the soil conditions where the platform shall be installed, the outer 12 cells and some of the inner cells will be fitted out with a steel skirt. When being installed on location the steel skirt and part of the concrete skirt will
penetrate the sea bottom to a precalculated depth. The weight of the platform will count for most of the penetration while the final adjustments will be performed by evacuation of the water trapped in between the skirts. At a later stage the voids will be filled with a grout mixture.

Inside cell no. 1 there is a minicell containing all the ballasting and grouting facilities. If desired other cells may also be 'host' for a minicell. Normally cell and shaft no. 1 is intended for utility purposes while cell and shaft nos. 3 and 5 are used for drilling purposes.

The main dimensions of a typical Condeep platform are given in fig. 1. Naturally the structures are tailored for a specific location. The dimensions therefore depend on waterdepth, soil conditions, purpose of platform use (drilling, production, quarter, etc.).

CONSTRUCTION OF THE CONDEEP

As a guide-line for the Statfjord 'A' platform these volumes were required:

Total area of formwork: \( 250,000 \text{ m}^2 \)

Concrete: \( 87,000 \text{ m}^3 \)

Reinforcement: 12,000 tons (as a comparison: the Eiffel Tower required 7500 tons of steel)

Posttensioning steel cables: 2,600 tons

Max. vertical length of cable: 130 m

The construction and transport of such a huge, massive piece of concrete is complex. Apart from the practical difficulties in the different construction phases, the quality requirements are very demanding. When installed at the field the structure will be exposed to large forces from waves, wind, dynamic loads, climate etc. However, when under construction in sheltered waters, the forces acting on the uncompleted structure might be still larger under unfavourable conditions. For example when towing the bottom raft from the dock to the deep water site with the lower part of the cell walls only completed, the structure is very flexible and large bending moments could be introduced. Another critical phase with large forces acting occurs during deckmating when the entire structure except the upper 6-8 m of the shafts, will be submerged. At this stage the hydrostatic pressure could be as much as that corresponding to 170-180 m of water. Due to this the application of high tensile reinforcement, bars as well as cables, concrete quality of C 50 or better, strict control measurements etc. are essential. The congestion of reinforcement bars and cables, embedded ducts and pipes require a well selected material to produce a concrete of C 50 quality. Posttensionned cables which are used to a large extent and the grouting of cableducts stretching for a vertical distance of 100 m or more can impose serious problems.

The construction and installation are carried out in 5 main phases:

1. Construction of the base raft in a graving dock.

2. Completion of the concrete structure and mating with the steel deck at a deepwater site.
3. Tow-out to field.

4. Positioning and installation.

5. Grouting.

The tow-out, positioning and installation at the field will be covered in another paper presented by K. Werenskiold, also from Norwegian Contractors.

CONSTRUCTION OF THE BASE RAFT IN THE DOCK

The base raft of a Condeep includes the steel skirts, the concrete skirts, the lower domes and the lower part of the cell walls. The dead weight of such a concrete raft is approximately 45000 tons, and the draft would be about 12 m. By filling the voids between the steel skirts and lower domes with air, the raft can be given an extra uplift of approximately 25000 tons, thus reducing the draft to 8 m.

The dry docks planned and built by Norwegian Contractors in Stavanger can accommodate two larger platforms simultaneously. If required the two docks can be separated by a partitioning wall. Dock no. 1 has a depth of 10 m while no. 2 is more shallow with 8.40 m. A deeper dock would be preferable as this would give a wider range of platform designs, changes during construction, loading etc. However, a deeper dock would increase the risk of ground collapse, leakages, difficulties related to opening and closing of the dock, as well as the costs would increase considerably.

The Condeep design with 19 identical cells favours the use of a shutter system which can be moved from cell to cell in combination with steel skirt mounting. With the steel skirt of one cell completed the carpenters and steel fixers can move in, place the forms and reinforcement and then move to the next steel skirt prepared in the meantime. In this manner only 6 sets of shuttering would be required to complete the 19 concrete skirts.

The same applies for the lower spherical domes, although 9 sets of girders and forms have to be available to ensure smooth progress. Due to the heavy congestion of reinforcement and complicated bulkheads for the separate concrete pours, every second cell is left to be concreted in a second stage. To save time 2 or 3 cells are cast at a time. The very smooth progress is clearly mirrored in the short production time required to complete the bottom raft from el. + 0 to approximately el. + 15 (see figure 2). However, to achieve this all activities must be planned in greatest details to avoid bottlenecks and congestion of manpower.

When at least 4 lower domes are completed, erection of the slipform can commence. To build a slipform of this size with a total length of approximately 1200 m, requires know-how, skill and precision. For a simultaneous slipforming of the 19 cell walls Norwegian Contractors use a system developed by Ing. F. Selmer A/S, one of the joint venture companies. The complete formwork for 19 cells is built as one unit which is lifted by some 650 5 ton hydraulic operated jacks. The jacks are divided into two groups, each group served by one hydraulic pump. The two pumps could work automatically, but our experience is to operate, i.e. start, the pumps manually. Each separate jack again can be adjusted with a hydraulic handpump.

Concrete for the slipforming is brought from a stationary batchplant by truck-
mounted hoppers, lifted on to the slipform working deck by 4 Liebherr 130 cranes, and distributed to the 1200 m of form by use of wheel-barrows. It may look outdated to use wheel-barrows instead of more mechanized equipment like a concrete pump and hydraulic operated placer. However, we find the wheel-barrow very flexible, more reliable and less expensive than other equipment considered. Also concrete spill on reinforcement bars and working deck is kept to a minimum, thus reducing the risk of poor bond between reinforcement and concrete or lumps of already set concrete in the pour. The rate of progress for the slipform in the dock was about 180 cm per day. On average the jacks were activated 4-5 times an hour. Each stroke measured 20 m/m. The setting of the concrete was aimed to start 2/3 of the form depth, i.e. about 80 cm from the wet concrete surface. This was achieved by introducing different kind of additives to the wet mix.

Due to the thickness of walls, especial in the conjunctions, and the high concrete quality, the heat of hydration can be considerable. When cooling, this again might create cracks. The temperature peak follows as a wave after the slipform (see figure 4). To reduce some of the effects caused by the hydration heat, fresh water is continously sprayed on both sides of the cell walls. Using an optimal concrete proportioning and an even production, the cement content is kept to a minimum. By experience the maximum temperature in the concrete is raised by 1°C for each 10 kg of cement.

While in the dock only the first 6-7 metres necessary for float-out boyancy are slipformed. The rest must, of course, be left for the deepwater site.

In the dock extensive instrumentation systems for future use are installed or embeded in the concrete, including:

a) Navigation aids, such as echo sounders and sonars.

b) Ducts, pipes and manifolds for offshore grouting works.

c) Skirt pipes (evacuation systems).

d) Liquefaction systems.

e) Ballast systems.

f) Strain measuring devices.

g) Conductor penetration pipes.

To establish the air cushion needed for the float-out from the dock, the grouting pipesystem is utilized to convey air from the compressors to the voids beneath the lower domes.

As clearance between the dockbottom and the raft sustained by the air cushion, is limited, the float-out is performed on rising tide. When taking the concrete raft through the narrow dockgate water will rush in to replace the wet volume occupied by the raft. This may lead to unfavourable currents disturbing the navigation. For this reason tugs can only be used for pulling and braking, while wireropes connected through sheaveblocks to landbased winches will steer the raft out through the gate.
COMPLETION OF THE CONCRETE STRUCTURE AT THE DEEP WATER SITE

With the float-out from the dock completed, the raft is now towed to a deep water site for completion of the caisson and the shafts. The water depth at our site is approximate 250 m, and the site is capable of accommodating very large structures. In our experience a threepoint mooring gives best control of the platform excursion. If possible all three moorings should be fixed on shore, the tightening winch at least must be land based. To withstand the windforce from a 100 year storm we have installed 120 m/m ORQ chain, with a breaking strength of 1260 tons. Tightening capacity of the winch is about 450 tons.

With the bottom raft secured in the 3 chains of one mooring set, and water and electricity supply hooked up, the slipforming of the 19 cylindrical cell walls can continue. Two, or more, fully automatic batchingplants on concrete barges are moored alongside the structure. The four Liebherr 130 cranes with their bases cast into the bottom raft, will convey the concrete in buckets direct from the batchingplant to hoppers on deck. From here the concrete will again be distributed by wheel-barrows. While slipforming afloat the rate of climb will be close to 200 cm per day. This requires an average production of 1800 m\(^3\) of wet concrete per 24 hours. During the slipform operation different kinds of pipes, cables, embedment plates etc for installation to follow later, are cast in. For the Statfjord 'A' platform approximately 6000 different items had to be placed in the continuously moving shutter. To keep trace of when and where to place what computer outprints had to be prepared.

The duration for slipforming the 19 cell walls is approximately 25 days.

To facilitate operations while slipforming it is desirable to maintain a constant freeboard on the structure. By controlled pumping of seawater into the cells the evergrowing concrete structure will sink into the water, leaving a freeboard as desired. Such a pumping is also required to control the verticality of the structure while slipforming. The slipform itself is controlled by lasers.

With the cylindrical cell walls completed, but preferably before casting the upper spherical domes, the permanent ballast material must be brought into the cells. As the filling of ballast material into the cells may create unfavourable tensile stresses in the structure, strict measures must be taken to ensure a safe ballasting procedure. The Condeep platform is ballasted with approx. 100,000 tons of surplus material from concrete gravel deposits. The ballast material is distributed at the bottom of the 16 storage cells and covered with concrete. Other kind of ballast material like olivine and fine ground iron ore has been used in the Condeep. However, like in some other gravity structures concrete alone was not applied for ballasting purposes.

With the ballasting of one cell completed, closing of the upper cupolas can commence. As the upper domes have the same radius as the lower ones, the same system of steel trellises and wooden forms can be used. For the upper domes the system must be turned upside down, however. Also here a total of 9 shuttering sets is sufficient for a smooth progress.

The 3 cells without a cupola are the bases of the 3 conical shafts carrying the steel deck. These shafts can be slipformed in one or several steps, as required or desired. In either case a highly sophisticated slipform system is required. To slipform the conical shafts with tapered wallthickness a steelform was used, in
contrast to a wooden form employed for the vertical cell walls with uniform thickness. The actual form consists of overlapping steel plates supported by yokes and lifted by hydraulic jacks. The assembly of each shaft can be adjusted by means of a hydraulic jack system and manually operated screw control, thus permitting a gradual change in shaft diameter and wall thickness. When slip-forming the three shafts on the Statfjord 'A' platform 30 jacks were mounted on each shaft. Although the jacks used for the shaft assembly were of another type (for smooth climbing rods versus notched rods for the cell walls) the stroke was the same, i.e. 20 cm. The shaft radius and wall thickness was reduced continuously with an average of $\Delta R = 5.5$ cm/m and $\Delta t = 1$ cm/m. To eliminate rotation during slip-forming the 3 forms are interconnected in such a manner that each is permitted to advance or retard a certain vertical distance in relation to the others.

To keep the forms aligned to the true centerlines lasers are applied to control verticality and rotation.

Slipforming the Condeep shafts has been performed under severe winter conditions with winds of galeforce and stronger. Keeping the draft constant by deballasting, the ever increasing height of the shafts or freeboard, reaching a maximum of approximately 115 m above sea level, gradually reduce the efficiency of plant and manpower. Taking this into account together with the vast amount of embedment-plates, pipes, ducts, prestressing cables etc. being cast in, the slipforming system, equipment and installations used have proved to be very reliable and efficient.

A climbing rate of about 310 cm per day was average for the Statfjord 'A' shafts.

In the timespan between slipforming the shafts and deckmating, there are several operations, some rather complicated, to be carried out simultaneously and partly above each other. The safety aspect in this relation is one of the largest difficulties to handle.

The most important tasks to be performed before deckmating are: tensioning and grouting of cables, construction and mounting of the transition pieces on top of the shafts, mounting of guides and conductorpipes in shaft 3 and 5, mounting of steeldecks, pipes, pumps etc. in shaft no. 1.

**MATING WITH THE DECK**

With the shafts completed the concrete structure is now ready for mating with the steel deck. The steel deck will arrive at the deep water site floating on 2 barges spaced sufficiently far apart to let the 3 shafts enter between. For this purpose the concrete structure must be submerged by sluicing or pumping water into the cells. To compensate for the extreme hydrostatic pressure the 12 outer cells are filled with compressed air of approximate 4 ATA. Sluicing of water, pumping compressed air, control of tilt and draft are performed from a control center located in the utility shaft.

The ballasting continues until a few metres of the shafts only remain above water, and the deck is floated over the immersed shaft tops. Barges and deck will be locked into a position related to the shafts. By deballasting the concrete structure, the deck load will gradually be transferred from the barges to the 3 shafts. With that operation completed the barges are removed, and the platform
further deballasted to a suitable freeboard.

Considering the very large sizes and weights of the two pieces, the concrete substructure and the steel deck, the deck mating is a highly delicate operation. When immersing the caisson 170 m deep in the sea the 19 cylindrical cells will certainly be deformed. To relieve the structure of some of the strains thus induced, the 16 closed cells are filled with compressed air of 4 kg/cm². The deflection of the 3 shafts when submerged must be considered such that no stresses are induced into the deck when the caisson is deballasted.

CONCRETE PRODUCTION

Except for the concrete placed while in the dock phase, the total volume for all the platforms has been produced by batching plants mounted on barges moored along the floating structure. Each plant consist of two pan-type compulsory mixers of 0.5 m³ each, silos for cement and aggregates, hoppers and conveyors for receiving the aggregates which are delivered by boats carrying 300-500 tons. The cement is loaded from land based storage silos into 30 tons tanks mounted on barges, the barges are towed to the batchplant where the cement is blown into the silos.

For the Statfjord 'A' platform some typical production figures are as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete for slipforming of caisson, 2nd. stage</td>
<td>47.400 m³</td>
</tr>
<tr>
<td>Time needed for slipforming of caisson</td>
<td>28 days</td>
</tr>
<tr>
<td>Mean average daily production</td>
<td>1.680 m³</td>
</tr>
<tr>
<td>Maximum production per day</td>
<td>2.000 m³</td>
</tr>
<tr>
<td>Mean average rate of slipforming</td>
<td>180 cm</td>
</tr>
<tr>
<td>Mean average daily production of shutter</td>
<td>4.000 m²</td>
</tr>
</tbody>
</table>

Slipforming offshore structures of such magnitude requires production equipment which can be relied on. Shorter breakdowns can be controlled by adding retarders to the concrete, thus keeping it alive until equipment is repaired. A complete breakdown however, can be disastrous to the structure. For the Statfjord 'A' slipforming as well as for the other Condeep's produced, the total down time for the batching plant was approximate 15 hours during the 28 days of slipforming. The down time was mainly due to cleaning (every 4-5 hours) and maintenance.

CONCRETE QUALITY

It is generally accepted that a "good" concrete is well suited to resist the severe exposure conditions of a marine environment. A good concrete should have certain requirements related to the durability and strength.

For the first two Condeeps a concrete strength of C45 was required. The design for the Statfjord 'A' platform was based on a C50 concrete. The 28 days cube strength for the two qualities are 45 and 50 N/mm² respectively.

Adequate durability can only be ensured through careful selection of materials, proper attention to a number of design aspects and satisfactory workmanship. Some guidelines on these points can be found in most national Codes of Practice.
while special rules and recommendations are given in documentations given in recent years (1) (2).

Ordinary Portland Cement (PC 300) has been used throughout the structures. The cement content has varied between 400 to 450 kg/m³, the water/cement ratio approximate 0.42. Admixtures are essential aids to produce adurable and impermeable structure. Their basic functions are: to retard the setting time of the concrete, to improve workability, to entrain air (in the splash zone). The addition of 1-5 liters of BETOKEM LP per m³ will generally reduce the water requirement for a given workability by 5 to 10% and retard the setting time of the mix by 2 to 12 hours. When properly controlled this will lead to increased strength and absence of cold joints in large pours. Up to 24 hours retarding has been achieved without any reduction of quality.

The mix proportions have been selected on the basis of experience gained through previous projects (see figure 5) and a large number of supplementary tests. For the Condeeps one basic mix with minor variations only, has been maintained throughout. The necessary adjustments are related to changes in workability, retarding and air content.

For the Statfjord 'A' platform a total number of approximate 6000 cubes were tested. The mean average 28 days compressive strength on 10 cm cubes was approximate 55 N/mm². The results demonstrates a very uniform production with standard deviations of approximately 3 N/mm². Tests performed on cores drilled out of completed structures shows compressive strength 90 to 100% of the corresponding testcube stored under equal environments.

SUMMARY

The paper deals with different construction phases of a concrete gravity structure. Particular reference is made to the large scale slipform construction of storage cells and the shafts as well as proper selection of material for concrete mix, strict concreting procedures, inspection and quality control.

The experience gained from the concrete platforms already constructed demonstrates that large monolithic sea structures can be satisfactorily produced.

References:


1. - 2. month

2. Construction of the lower domes and slipforming of the lower part of the cell walls

7. - 8. month

3. The dry dock has been filled with water and the dock gate is removed. The construction is floating on an air cushion within the skirt walls

8. month

4. Outside the dock gate the air is released

9. - 16. month

5. The construction is anchored position

11. - 13. month

6. Filling of ballast sand and construction of the lower domes

14. - 16. month

7. Submerging the shafts

17. month

10. To field

15. 20. month

11. Submergence at the field

16. Penetration of shafts into the sea bed

12. The platform ready for operation

Figure 2

*EXC*
Ambient temperature 12 to 16 °C
BRENT B, CELL WALLS, ELEV. 23.4

Ambient temperature 10 to 20 °C
BERYL A, CELL WALLS, ELEV. 17, 6
③ INTERSECTION CELLS 1 & 2
⑥ OUTSIDE ARC, CELL 10

Fig. 4—TEMPERATURE CURVES CONDEEP

Fig. 5—HISTOGRAM 28-DAYS COMPRESSIVE STRENGTH
(10 cm cubes) CONDEEP BERYL A AND BRENT B
CONCRETE QUALITY AND PRODUCTION DATA
CONDEEP STATFJORD A.

SLIPFORMING OF CELL WALLS. CONCRETE GRADE C50

<table>
<thead>
<tr>
<th></th>
<th>MARCH</th>
<th>MAY</th>
<th>JUNE</th>
<th>1975</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL VOLUME OF CONCRETE</td>
<td>9200 m³</td>
<td></td>
<td>47400 m³</td>
<td></td>
</tr>
<tr>
<td>TIME</td>
<td>6.5 days</td>
<td></td>
<td>28 days</td>
<td></td>
</tr>
<tr>
<td>MEAN CONCRETE PRODUCTION</td>
<td>1415 m³/day</td>
<td>1690 m³/day</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MEAN SLIP RATE</td>
<td>150 cm/day</td>
<td>174 cm/day</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MEAN 28 DAY COMPR STRENGTH (10cm CUBES)</td>
<td>54.4 N/mm²</td>
<td>54.6 N/mm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ST. DEVIATION</td>
<td>2.9 N/mm²</td>
<td>3.1 N/mm²</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*PEAK PRODUCTION 660 m³ PER 8 HOUR SHIFT

Figure no. 6
ABSTRACT

A comparative study is presented for the effect of soil modelling of an offshore gravity monopod. The foundation conditions considered are: i) Rigid Base, and ii) Semi-Rigid Base. Representing the hydrodynamic effects in terms of the added water mass concept, the structure foundation system (Semi-Rigid Base) is analysed for three soil-models, i) linear soil, ii) 'equivalent linear' soil, and iii) Elasto-plastic soil (following the Drucker-Prager yield criterion). In all the analyses, the structure is assumed to be linear. The wave forces on the tower are computed from the Morison equation and at the base, from diffraction wave theory. For the 'equivalent linear' soil model, nonlinearity is taken into account by an approximate iterative procedure in which the stiffnesses are made strain-compatible at each soil element centroid. In the elasto-plastic model, an incremental finite element stiffness procedures is used. The results indicate the significant influence of soil behaviour on structural response, particularly displacements. Further research needs to be done for studying the sensitivity of structural response to variations of soil shear moduli, variable damping, and response to stochastic wave loading.

INTRODUCTION AND REVIEW OF LITERATURE

The advantages associated with i) heavy weight, ii) resistance to marine corrosion and fatigue, and iii) elimination of difficult and expensive piling operations for installation make the gravity concrete monopod an attractive alternative to a pile-supported steel braced structure.

Malhotra and Penzien (1970) and Berge and Penzien (1974) presented a method for analysing the fixed based offshore platform taking into account wave-structure interaction. The nonlinear equations governing the wave-excited response were solved by the method of equivalent linearization. Applications were described for simple two-dimensional and three-dimensional steel framed offshore structures. Later, the above method was extended by Penzien (1976) to analyse a structure-foundation system taking into account the coupling of the linear structure with frequency dependent foundation impedances. Bell, Hansteen, Larsen and Smith (1976) considered both deterministic and stochastic wave loadings in the calculation of the dynamic characteristics of a CONDEEP type gravity platform. The formulation of the problem was based on a lumped mass finite-elemented model for the structure while the foundation and the soil were represented by a linear elastic half-space model, which provides equivalent spring, and dashpot constants. Nataraja and Kirk (1972) carried out a spectral analysis of a typical multi-tower gravity platform to wave loading consid-
ering soil-structure interaction. Hasselman (1975) studied the probabilistic re-
response of an interactive soil structure system to seismic excitation. The analysis
was based on the linearized subsystem approach; the platform structure and the pile
foundation formed one subsystem and the soil, the other subsystem. The nonlinearity
of the soil, treated in terms of the secant shear modulus following the suggestion
of Hardin and Drnevich (1972), was represented in terms of hysteretic soil shear
stress-strain curves which vary with strain and depth. The spring constants used
were based on the lateral and vertical forces of the soil corresponding to the
strains developed at different levels; the response of the entire soil-structure
system was calculated in an iterative manner. The results indicated that dynamic
tuning between the soft soil and the platform structure gives a high level of re-
response and, for a very high input level, the soil column frequency may exceed the
structure period causing a significant dynamic attenuation.

Utt, Durning, Duthweiler and Engle (1976) analysed the recorded dynamic response of
a monopod platform in Cook Inlet, Alaska. The response was simulated with computer
models to study the reason for observed changes in the fundamental frequency. Based
on the observation of a change in platform frequency from 0.91 Hz in 1967 to 0.77 Hz
in 1972 owing to changes in the soil conditions of the foundation, it was concluded
that the platform's dynamic characteristics are insensitive to deck mass changes.
Haldar (1977) and Haldar, Swamidas, Reddy and Arockiasamy (1977) studied the dynamic
response to ice forces of a fixed offshore platform considering soil nonlinearity.
The method used was an equivalent linearization technique as proposed by Seed and
Idriss (1969). Two types of structure (one a pile supported framed tower and the
other, a gravity type monopod), were analysed and the response of the structures
computed in three different ways: i) Time-history analysis, ii) Response spectrum
method, and iii) Power spectral method. The soil nonlinearity was considered in
terms of both strain and displacement dependency with depth. Later, the method was
extended for analysing a prestressed concrete gravity platform subjected to a deter-
Reddy, Haldar, Arockiasamy and Swamidas (1977) presented a comparison of the model-
ing criteria for deterministic dynamic analysis of offshore tower foundation systems
subjected to ice forces. Arockiasamy, Reddy, Haldar, and Yen (1977) presented the
dynamic response analysis of a large diameter offshore prestressed concrete gravity
platform foundation system subjected to wave loading. Soil nonlinearity was consid-
ered in terms of an elasto-plastic model obeying the Drucker-Prager yield condition.

STATEMENT OF THE PROBLEM

The structure analysed for the present problem is similar to that one described in
Ref. 15 and shown in Fig. 1. The vertical part of the tower is modelled as a beam
element of hollow circular cross-section with varying diameter. The bottom concrete
caisson, which serves as foundation and also as a compartmented storage, is idealized
as an assemblage of equidistant ribs, and modelled as a thick slab for determination
of the bending, shear and torsional rigidities Timoshenko and Kriger (1959). The
foundation, and the soil medium are represented by, i) two-dimensional isoparametric
plane strain, and ii) variable-number-nodes thick shell and three dimensional isopar-
ametric elements. The thickness of the soil elements in the plane strain model is
varied parametrically until its fundamental frequency matches with that obtained from
the three-dimensional soil model. The finite element idealisation of the soil
structure system is shown in Fig. 2. The hydrodynamic nodal loading is taken into
consideration in terms of the concept of added water mass, which is equal to the sum
of the mass of the water displaced and the water mass inside the cylindrical tower.
The soil-structure system is analysed for two types of foundation conditions: i) Rigid base and ii) Semi-rigid base. The semi-rigid base is again treated in terms of linear and nonlinear soil behaviour. In all the analyses, the structure is assumed to be linear. The nonlinearity, which is introduced in the soil due to the cyclic loading, in the superstructure (waves), is considered with two kinds of modelling; i) Equivalent Linearization, and ii) Elasto-plastic based on the Drucker-Prager yield condition. These models will be discussed in detail later. The determination of the wave loading on the tower is based on Stoke's third order wave theory, and on the caisson by the McCamy-Fuchs (1954) linear diffraction theory using a special purpose programme. The three semi-rigid cases considered are:

Case A: Linear Soil. The frequencies, mode shapes and response are obtained from SAP IV (1974) for the structure-foundation system due to wave force time-history using an initial set of shear moduli for the soil elements (Fig. 3).

Case B: Equivalent Linear Soil. The shear strain amplitudes at the soil element centroids, and the corresponding new set of shear moduli are computed by another special purpose programme developed by the authors. The SAP IV analysis is repeated with the new set of shear moduli until the shear strain amplitudes in the soil elements converge (Equivalent Linear Model).

Case C: Elasto-plastic Soil. In order to overcome the limitation of the computer programme, NONSAP (1974) in the element library (beam element not included), a special technique was used to analyse the soil structure system for elastic-perfectly plastic model. The time-dependent axial forces and shears transmitted to the soil in Case A, at the sea bed level, are treated as inputs to the nonlinear soil medium as shown in Fig. 4. A computer programme is specially developed to implement this, and make the forcing function adaptable to NONSAP. The foundation soil medium is then analysed considering the Drucker-Prager yield condition (1952) for nonlinear soil characteristics. The revised soil shear moduli are estimated by the special purpose programme used for Case B, and used to modify the input axial forces and shears. This cyclic procedure, involving a 'melding' of SAP IV and NONSAP, is repeated till convergence is achieved. Finally, the revised soil shear moduli (converged) obtained from NONSAP analysis are given as input data to SAP IV programme to compute the structural response i.e. the nonlinear (elasto-plastic) response.

GENERATION OF WAVE FORCES

A summary is given for the generation of hydrodynamic forces on the structure in two parts: the first part concerns those forces which are a result of fluid acceleration and viscous and from drag (Morison Equation); the second part concerns those from the diffraction wave theory to obtain the resultant wave force in the bottom part of the tower (McCamy-Fuchs Equations).

Wave Forces on Towers

Water particle velocities and accelerations in the horizontal and vertical directions are calculated at different intervals of depth and position along a wave using Stoke's third order wave theory (1974). This theory is applicable to the example problem because the ratio of relative depth to wave length (d/L) is greater than 1/8. A 28-m wave (corresponding to centenary storm) with a 15-sec period and 100-m water depth is considered for computing the hydrodynamic forces on the structure. The wave length is initially assumed to be equal that for deep water, and the exact value is computed by an iterative procedure from the equation.
\[
L = \frac{9\pi^2}{2\pi} \tanh \left( \frac{2\pi d}{L} \right) \left[ 1 + \left( \frac{2\pi d}{L} \right)^2 \right] \frac{14 + 4 \cos h^2 \left( \frac{4\pi d}{L} \right)}{16 \sin h^4 \left( \frac{2\pi d}{L} \right)} \]  

where

\[
a = \left( -0.5 \sqrt{b_1} + \sqrt{0.25 b_1^2 + \frac{a_1}{27}} \right)^{1/3} + \left( -0.5 \sqrt{b_1} - \sqrt{0.25 b_1^2 + \frac{a_1}{27}} \right)^{1/3}
\]

in which

\[
a_1 = \frac{L^2}{2 \pi h} f_3^{-d}(L)
\]

\[
b_1 = - \frac{H L^2}{2 \pi^2 h f_3^{-d}(L)}
\]

and

\[H = \text{wave height}\]

The wave speed \(C\) is calculated from

\[C = \frac{L}{T}\]  

where

\[T = \text{wave period}\]

The horizontal water particle orbit velocities and accelerations are given by

\[u = C F_1 \cosh (K) \cos (G) + C F_2 \cosh (2K) \cos (2G) + C F_3 \cosh (3K) \cos (3G)\]  

and

\[a_x = \frac{2\pi C}{T} F_1 \cosh (K) \sin (G) + \frac{4\pi C}{T} F_2 \cosh (2K) \sin (2G) + \frac{6\pi C}{T} F_3 \cosh (3K) \sin (3G)\]

where

\[F_1 = \frac{2\pi a}{L} \frac{1}{\sinh \left( \frac{2\pi d}{L} \right)}\]

\[F_2 = \frac{3}{4} \left( \frac{2\pi a}{L} \right)^2 \frac{1}{\sinh^4 \left( \frac{2\pi d}{L} \right)}\]

\[F_3 = \frac{3}{64} \left( \frac{2\pi a}{L} \right)^3 \frac{11 - 2 \cosh \left( \frac{4\pi d}{L} \right)}{\sinh^7 \left( \frac{2\pi d}{L} \right)}\]
and
\[ G = 2\pi \left( \frac{x}{L} - \frac{t}{T} \right) \]

and
\[ K = \frac{2\pi}{L} (y + d) \]

in which
\[ y = \text{distance to water particle from the bottom}, \]
and
\[ d = \text{water depth} \]

The horizontal component of the wave force on the fixed tower is obtained from Morison's formula (19) as
\[ F = \frac{1}{2} \rho C_D D u |u| + \rho C_M \frac{\pi D^2}{4} \frac{du}{dt} \]

where
\[ D = \text{diameter of tower} \]
\[ C_D = \text{drag force coefficient} \]
\[ C_M = \text{inertial force coefficient} \]

Dividing the tower into a number of elements, the fluid velocity and accelerations are computed at element centroids and the wave force time histories are generated by a specially developed computer programme.

**Diffraction Wave Forces on the Caisson**

For the bottom of the tower, the caisson diameter is greater than 20 per cent of the wave length and the fluid pressure gradient will vary over the width of the caisson; hence diffraction wave theory is used to obtain the resultant wave force. The total force in the horizontal direction, at a depth, \( y \), of the surface, per unit length of caisson is given by MacCamy and Fuchs (17) as
\[ F_h(y) = \frac{\rho g H L}{\pi} \cosh \left[ \frac{2\pi(y + d)/L}{\cosh \frac{2\pi d}{L}} \right] \left[ f_A \cos \left( \frac{2\pi t}{T} - \alpha \right) \right] \]

where
\[ H = \text{wave height} \]
\[ \tan \alpha = \frac{J_1' \left( \frac{\pi D}{L} \right)}{Y_1' \left( \frac{\pi D}{L} \right)} \]

and
\[ f_A = \frac{1}{\sqrt{\left[ J_1' \left( \frac{\pi D}{L} \right) \right]^2 + \left[ Y_1' \left( \frac{\pi D}{L} \right) \right]^2}} \]

in which

155
The wave force time histories are computed using Eqn. 6 at the nodal points of the caisson by a computer program specially developed by the authors.

**SOIL STRESS-STRAIN BEHAVIOUR**

Soil stress-strain characteristics are nonlinear, and under constant cyclic loading follow hysteretic loops, such as those shown in Fig. 5. For loading (such as wave and earthquake) causing high strains, the secant shear modulus decreases and the damping (defined as a function of the area enclosed by the loop) increases, compared with loads associated with smaller strains. It is readily apparent that the secant modulus and damping are strain dependent.

An approximate relationship between the shear modulus and undrained shear strength (Fig. 6) for clays, established by Seed and Idriss (1970) for a wide range of strain amplitudes, is used for estimating the initial shear moduli of the soil elements and subsequent reduction factor. For the purpose of analysis, the maximum shear modulus, $G_{\text{max}}$, for most types of clay samples can be set equal to the undrained shear strength, $\sigma_u$, times the value of $(\frac{G}{S})_{\text{max}}$ at low strain levels. The strain dependence of the shear moduli and damping ratio for clays, given by Idriss (1975), is shown in Fig. 7. A reasonable estimate of the shear modulus of clay at any strain amplitude can be obtained by determining $G_{\text{max}}$ from the undrained shear strength of the soil sample, and applying the reduction factors shown in Fig. 7 and Table I to determine the values at other strain levels. The above data has been used in evaluating the response for both kinds of models.

**DYNAMIC EQUILIBRIUM EQUATIONS**

The dynamic equations of motion for a system of structural elements are expressed in the following matrix form as

$$
\begin{bmatrix}
M^* & 0 \\
0 & K^*
\end{bmatrix}
\begin{bmatrix}
U \\
\dot{U}
\end{bmatrix}
+ 
\begin{bmatrix}
C^* \\
0
\end{bmatrix}
\begin{bmatrix}
\dot{U} \\
U
\end{bmatrix}
+ 
\begin{bmatrix}
K^*
\end{bmatrix}
U = P(t)
$$

(7)

where

- $[M^*]$ = combined mass matrix for soil structure systems having only diagonal elements for lumped mass approximation,
- $[C^*]$ = combined damping matrix for the soil-structure system,
- $[K^*]$ = stiffness matrix for the soil-structure system being positive symmetric,

and

- $[P(t)]$ = load vector.

In the above equation, the mass matrix $[M^*]$ contains lumped structural masses, added water masses and participating soil masses. Similarly, the damping matrix $[C^*]$ contains the viscous equivalent of the structural, hydrodynamic and soil damping coefficients (specified in terms of modal damping) in the response calculation. The formulation of the stiffness matrix $[K^*]$ depends on the soil modelling.

For the rigid base and linear soil (Case A) model, the linear stiffness matrix is formulated in the usual manner as
\[
[K] = \int_{\text{Vol}} [B]^T [D] [B] \, dv
\]
(8)

where

- \([B]\) is the strain displacement matrix,
- \([D]\) is the constitutive matrix with linear stress vs. strain relationship.

In Case B (Equivalent Linear Model), the linear stiffness matrix is formed with initial soil shear moduli for the first cycle of the iteration, and then modified with the revised set of shear moduli as obtained from the strain compatible soil properties - Table I.

In Case C (Elastic-perfectly plastic Model), a nonlinear material constitutive matrix is formed which is dependent on the changing stress state. (Ref. 5). From the material constitutive matrix, the stiffness matrix may be formed in the usual manner as described in Eqn. 8.

**Case A: Rigid Base and Linear Soil**

The response is evaluated by the Normal Mode method. The shear moduli are not strain-dependent for the linear soil case.

**Case B: Equivalent Linear Method**

The normal mode method makes use of superposition which is only valid for linear systems. In the section on soil behaviour, it was pointed out that shear deformation occurring in soils due to dynamic loading introduces nonlinearity. Seed and Idriss (1969) included it by introducing an equivalent linear method in which an approximate solution is obtained by a linear analysis, provided the stiffness and damping used in each soil element are compatible with the effective shear-strain amplitudes developed in the element. Data on strain-compatible soil properties for clays and sands has been published in Ref. 25. Typical variations of shear modulus and damping with shear strain are shown in Fig. 7 and summarized in Table I.

In applying the above equivalent linear method, a set of shear moduli and damping values is estimated first for the soil elements. Based on these variable properties, the structure is analysed and the stress history is computed at each soil element centroid. Having computed the stress history, the effective shear-strain amplitudes for each element are estimated, assuming that the effective shear-strain = factor \(x\) \(|\nu|\) maximum, where \(\nu\) defines the shear strain, and the factor is a constant varying between 0.50 and 0.75.

The effective shear-strain amplitude computed at each element centroid is checked in Table I to ensure that the strain amplitude is compatible with the values of shear modulus and damping used in the response evaluation. If the soil properties are not compatible, modified values are estimated from Table I, and the process repeated until convergence is achieved. The values obtained from the last iteration define the nonlinear response.

**Case C: Elasto-plastic Analysis**

Yield Surface Equation - Using the Drucker-Prager yield criterion, the yield surface can be given as (5, 8, 21)
\[ F = 3 \alpha' \sigma_m + \overline{\sigma} - K = 0 \]  
(9)

where

\[ \alpha' = \frac{2 \sin \theta}{\sqrt{3} (3-\sin\theta)}, \]

\[ K = \frac{6c \cos \theta}{\sqrt{3} (3-\sin\theta)} \]

\[ \sigma_m = \frac{1}{3} (\sigma_x + \sigma_y + \sigma_z), \]

and

\[ \overline{\sigma} = \left[ \frac{1}{2} (s_x^2 + s_y^2 + s_z^2) + (T_{xy}^2 + T_{yz}^2 + T_{zx}^2) \right]^{1/2}, \]

in which

\[ s_x = (\sigma_x - \sigma_m), s_y = (\sigma_y - \sigma_m), s_z = (\sigma_z - \sigma_m), \]

and

\[ c \text{ and } \theta \text{ are the cohesion and angle of Coulomb friction.} \]

The normal vector, of the derivative of the yield function with respect to stresses, is given as

\[ \frac{\partial F}{\partial \{\sigma\}} = \{\alpha\} \]  
(10)

in which

\[ \{\sigma\}^T = \{\sigma_x, \sigma_y, \sigma_z, T_{xy}, T_{yz}, T_{zx}\} \]

Expressing Eqn. 10 as a function of the invariants \((\sigma_m, \overline{\sigma}, \phi)\)

\[ \frac{\partial F}{\partial \{\sigma\}} = C_1 \frac{\partial \sigma_m}{\partial \{\sigma\}} + C_2 \frac{\partial \overline{\sigma}}{\partial \{\sigma\}} + C_3 \frac{\partial J_3}{\partial \{\sigma\}} \]  
(11)

where

\[ J_3 = s_x s_y s_z + 2T_{xy} T_{yz} T_{zx} - s_x T_{xy}^2 - s_y T_{yz}^2 - s_z T_{zx}^2 \]

and

\[ \phi = \frac{1}{3} \sin^{-1} \left[ -\frac{3}{2} \frac{J_3}{\sigma_3^2} \right] \text{ with } -\frac{1}{6} \pi \leq \phi \leq \frac{1}{6} \pi \]

The yield surface is thus defined in terms of the constants \(C_1, C_2 \) and \(C_3\).

Iterative Procedure for Nonlinear Response - The nonlinear dynamic analysis is based upon the formulation of an approximate, incremental, assembled nonlinear effective stiffness matrix \([K]\) at any time step, which is used to estimate increments in the nodal displacements \(\{\Delta u\}\), for a given load increment \(\{\Delta R\}\), by solving the equation

\[ [K] \{\Delta u\} = \{\Delta R\} \]  
(12)

For any deformed configuration, the difference between the sum of the element equilibrating forces and the applied nodal force, at any node, is treated as an
"unbalanced" nodal force. The unbalanced nodal forces are reduced to small quantities by successive corrections to approach the dynamic equilibrium configuration. The displacement and stress histories, at the soil element nodes, are obtained from NONSAP analysis using the above yield criterion.

The effective shear-strain amplitudes at each element centroid are estimated and checked for strain compatibility with those reported by Refs. 24 and 25. The properties of the soil elements, which do not exhibit compatible values, are modified and the SAP IV analysis is repeated with the new set of shear moduli for the soil elements. The input vertical forces and shears to NONSAP are then modified and the cyclic procedure repeated; the response from the final iteration of NONSAP analysis is assumed to be the approximate nonlinear response.

RESULTS, DISCUSSION AND CONCLUSIONS

The displacement time-histories of displacement at a typical node on the structure, and stresses at the centroid of a typical soil element are shown in Figs. 8 and 9. The maximum stress, $\sigma = 15.75$ kN/m$^2$, occurs at 14 sec. for Case C. The corresponding values in Case A and Case B are only $11.00$ kN/m$^2$ and $13.5$ kN/m$^2$ respectively. The increase in displacement when nonlinearity is taken into account is significant; for Case B the displacement at the load point is almost four times that for the Rigid Base condition $(0.1040 \text{ m})$. The elastic-plastic regions in the soil foundation, at 14 sec, are shown in Fig. 10. The hatched nodes of the soil elements, 3, 4, 5, and 13 indicate the plastified regions according to the yield surface equation.

The example illustrated in Case B has 270 D-O-F; computation times (CPU) for the eigenvalue and response analyses were 96.51 sec and 10.70 sec. on an IBM 370-158 computer. It must be pointed out that although the convergence in the shear strains in the soil elements is faster in the nonlinear analysis, it requires more number of iterations for the computed shear strain amplitudes to converge within a reasonably small percentage.

The example illustrated in Case C has 44 D.O.F.; computation times (CPU) for the eigenvalue and step-by-step solution (calculation of effective load vectors, updating effective stiffness matrices and load vectors for nonlinearities, solution of equations, equilibrium iterations, etc.) were $17.76$ sec and $364.85$ sec on an IBM 370/158 computer. It must be pointed out that the finite element mesh chosen was relatively coarse in order to reduce the computation time and avoid large cumulative errors in the incremental nonlinear finite element analysis.

The time domain analysis based on 'melding' of SAP-IV and NONSAP, with special purpose programmes for each of the following:

i) Generation of wave forcing functions,

ii) Shear strain amplitudes at the soil element centroids for Case B, and

iii) Equivalent time dependent nodal vertical forces and shear acting on the soil foundation for Case C

permits the application of 'equivalent linear' and elasto-plastic modelling for the soil foundation of a gravity platform.

While the analyses have been illustrated for the structure-foundation response to wave excitation, they can be easily extended to ice force and earthquake excitation. The accuracy in the analysis can be improved as more data from field and laboratory tests on cyclic loading of marine soils and gravity platforms, becomes available. Parametric studies are needed for the sensitivity of the structural response to i) changes in soil shear modulus values and ii) variable (strain-dependent) damping.
ACKNOWLEDGEMENT

The authors would like to thank Dr. R.T. Dempster, Dean, and Dr. A.A. Bruneau, Vice-President of Professional Schools and Community Services, Memorial University of Newfoundland, St. John's, Newfoundland, for their continued interest & encouragement. The support of this investigation by National Research Council of Canada, Grant No. 8119, is gratefully acknowledged.

REFERENCES


### TABLE 1: Strain-Compatible Soil Properties
(Ref. 14)

<table>
<thead>
<tr>
<th>Effective Shear Strain</th>
<th>( \log (\gamma_{eff}) )</th>
<th>Shear Modulus Reduction Factor*</th>
<th>Fraction of Critical Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{eff} ) (%)</td>
<td></td>
<td>Clay</td>
<td>Sand</td>
</tr>
<tr>
<td>( \leq 1 \times 10^{-4} )</td>
<td>-4.0</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>( 3.16 \times 10^{-4} )</td>
<td>-3.5</td>
<td>0.913</td>
<td>0.984</td>
</tr>
<tr>
<td>( 1.00 \times 10^{-3} )</td>
<td>-3.0</td>
<td>0.761</td>
<td>0.934</td>
</tr>
<tr>
<td>( 3.16 \times 10^{-3} )</td>
<td>-2.5</td>
<td>0.565</td>
<td>0.826</td>
</tr>
<tr>
<td>( 1.00 \times 10^{-2} )</td>
<td>-2.0</td>
<td>0.400</td>
<td>0.656</td>
</tr>
<tr>
<td>( 3.16 \times 10^{-2} )</td>
<td>-1.5</td>
<td>0.261</td>
<td>0.443</td>
</tr>
<tr>
<td>( 1.00 \times 10^{-1} )</td>
<td>-1.0</td>
<td>0.152</td>
<td>0.246</td>
</tr>
<tr>
<td>0.316</td>
<td>-0.5</td>
<td>0.076</td>
<td>0.115</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0</td>
<td>0.037</td>
<td>0.049</td>
</tr>
<tr>
<td>3.16</td>
<td>0.5</td>
<td>0.013</td>
<td>0.049</td>
</tr>
<tr>
<td>( \geq 10.00 )</td>
<td>1.0</td>
<td>0.004</td>
<td>0.049</td>
</tr>
</tbody>
</table>

* This is the factor which has to be applied to the shear modulus at low shear strain amplitudes (here defined as \( 10^{-4} \) percent) to obtain the modulus at higher strain levels.
FIG. 1 OFFSHORE GRAVITY PRESTRESSED CONCRETE TOWER
FIG. 2a THREE DIMENSIONAL FINITE ELEMENT IDEALISATION OF SOIL-STRUCTURE SYSTEM
FIG. 2b TWO DIMENSIONAL FINITE ELEMENT IDEALISATION OF SOIL-STRUCTURE SYSTEM

FIG. 3 TWO DIMENSIONAL FINITE ELEMENT IDEALISATION OF SOIL-STRUCTURE SYSTEM
FIG. 4 TWO DIMENSIONAL FINITE ELEMENT IDEALISATION OF THE SOIL FOUNDATION (not to scale)
FIG. 5 ILLUSTRATION OF STRAIN DEPENDENCY OF MODULI AND DAMPING IN SOILS (Ref. 24)

FIG. 6 IN SITU SHEAR MODULI FOR SATURATED CLAYS (Ref. 25)

FIG. 7a EXAMPLE RELATIONSHIP—REDUCTION IN MODULUS—VS. SHEAR STRAIN (CLAYS)

FIG. 7b EXAMPLE RELATIONSHIP—DAMPING RATIO VS. SHEAR STRAIN (CLAYS) (Ref. 14)
FIG. 8 TIME-HISTORY OF DISPLACEMENT AT TOP (NODE 1) OF TOWER

FIG. 9 TIME-HISTORY OF STRESSES AT CENTROID OF THE SOIL ELEMENT 22 (Cf. Fig. 2b)
FIG. 10 ELASTO-PLASTIC REGIONS OF THE SOIL FOUNDATION AT 14 SEC.
ABSTRACT

This paper discusses a mobile, gravity platform for year-round petroleum operations in the Canadian Beaufort Sea. The "Monocone" is a coined name derived from "Monopod" which alludes to the platform's single structural column rising from the centre of a circular base, and "Cone" which refers to a movable conical collar selectively positioned on this column. The conical collar causes the ice to fail in flexure and at minimum loads by maintaining a constant water line diameter in different water depths. Ice and wave forces constitute the main loading conditions and these were developed theoretically and experimentally, the latter in ice model basins and in a wave tank.

The environmental conditions and structural and foundation design of this prestressed concrete and steel structure are discussed, as well as model tank tests on seakeeping, towing, and lowering and raising procedures. The conical collar, if sufficiently large would allow controlled, near-vertical setdown of the platform to bottom in any water depth. In this paper, a "Monocone" platform for a maximum design water depth of 41.0 m is discussed.

INTRODUCTION

The continental shelf of the Canadian Beaufort Sea extends some sixty miles offshore the Mackenzie Delta and the Tuktoyaktuk Peninsula. Considerable open water normally exists between June and September, the period decreasing with distance offshore. Areas may still be invaded unexpectedly, however, by multi-year ice floes, some quite large with thick pressure ridges. In summer, storm waves build up shoreward from the northwest but are attenuated by the decreasing water depth. In winter the region is almost entirely covered by ice which grows to about seven feet thick by May. Internal stresses caused by wind, current and the pressure exerted by the polar pack causes the ice sheet to fail and give rise to an intricate system of pressure ridges.

Bottom soils are typical Delta sediments which generally vary from poor strength cohesive soils in the west to cohesionless soils in the east, reflecting the depositional history of the Mackenzie Delta.
In such an environment, it has been shown (Jazrawi and Davies, 1975) that a Monopod with its single column acting as a vertical indenter could be designed to remain on location year-round in the near-shore areas by failing the ice by the brute force method of crushing. However, in the exceptionally severe ice conditions further offshore, ice crushing loads become prohibitive to the economic design of marine structures. The Monocone, Fig. 1, was developed to keep ice loads within manageable levels by failing the encroaching ice in flexure. The concept also affords a method of controlled set down in deeper water and renders structural costs much less sensitive to water depth.

THE CONCEPT

The Monocone, Fig. 1, employs a steel conical collar to deflect and fail encroaching ice, upwards or downwards, depending on how the collar is deployed. To maintain a constant waterline diameter in various water depths, the elevation of the conical collar can be varied by controlled floatation, then frictionally clamped onto the shaft at the desired level. The collar, furthermore, is in two mechanically fastened segments which can be unlatched to enable removal from the shaft for repair or inversion.

The platform floats on a circular concrete hull, which when set down on bottom, also serves as the supporting base. The hull contains ballast tanks to lower or raise the structure on location, storage tanks for fuel and water and a pump room containing vessel service equipment. The hull is protected against deep keels of ice ridges by a heavy, sloping perimeter wall and by a system of 0.6 m high topside radial ribs with sand infill. The underside slab is flat and roughened to improve friction.

Supported on a 15.2 m concrete column and 15.2 m above the maximum design water level, is a triple deck, rectangular steel superstructure. This houses the drilling system, dry bulk and pipe storage, major mechanical systems and power generation, and a three-storey crew quarters.

The structure shown in Fig. 1 is for a maximum water depth of 41.0 m. Its light ship draft of 10.0 m is low enough to ensure passage around Alaska from a construction site on the west coast to the Beaufort Sea. On location the steel collar would normally be deployed with a minimum of 4.5 m protruding above water, to ensure proper ice uplift and failure. In that position, the water line diameter would be 27.4 m. In 26.0 m of water or less, the collar would rest on the hull, exposing increasingly larger waterline diameters. In water shallower than 16.8 m, the hull needs to be set in a dredged pocket with some 3 m water clearance above the top of the hull to allow most ice formations to pass over and be failed by the collar, instead.

ICE DESIGN CRITERIA

Geometric characteristics of ice formations were studied from photogrammetric surveys of stereo air photos and corroborated by "ground-truth" surveys involving coring and side-scan sonar investigations of typical ice ridges.

Mechanical properties of ice such as flexural, crushing and adfreeze strengths and elastic modulus, which are known to be dependent on salinity, temperature and loading rate, were also determined for a wide range of specimen size and "size-effects"
established for both crushing and flexural ice failure (Croasdale, 1974; Kry, 1975).

Ice-cone interactions were investigated both theoretically and experimentally the latter in ice model basins. The failure mechanism of a fairly uniform and homogeneous ice sheet on a cone is fairly well known and has been satisfactorily modeled by mathematical techniques (Nevel, 1965; Meyerhoff, 1962). For ridges, finite element analysis was conducted. The latter could model the three-dimensional behaviour of the interaction, and incorporate distributions of ice properties above and below the waterline caused by variations in ice temperature and consolidation within the ridge.

The post-failure forces exerted during the ice clearance process in pushing the broken ice pieces up and around the cone against friction or adfreeze was determined analytically. Adfreeze is ice adhesion to other materials by freezing. Unlike failure loads of sheets and ridges, rideup forces increase with waterline cone diameter because of their dependence on cone surface area above water, Fig. 2. Also evident from Fig.2, adfreeze forces can become very large in shallow water, again because of the large cone diameter exposed. Therefore, adfreeze control is essential to the economic design of the Monocone.

Theoretical studies of ice-cone interaction were supported by physical model tests using as modelling media both rapidly-chilled saline ice as well as synthetic wax. Furthermore, to enhance confidence in the validity of these tests, comparative data were generated in a large outdoor test basin on Imperial's research grounds in Calgary, Alberta (Robbins and Metge, 1975), Fig. 3. Physical modelling of ridges predicted failure loads to within 15% of the theoretical estimates and a reasonable match was obtained between "small prototype" ridge tests in Imperial's basin and equivalent small scale tests.

WAVE DESIGN CRITERIA

A wave hindcast for various locations across the southern Beaufort Sea was developed to establish design wave conditions. This was used to simulate wave forces on a 1/90 scale model in a test tank at the Hydraulics Laboratory of the National Research Council in Ottawa. A wave tank was considered the best means of reliably estimating wave loads because of the Monocone's complex geometry and the serious limitations of theoretical techniques in shallow water. Maximum wave loads and moments scaled up to prototype conditions, are given below:

<table>
<thead>
<tr>
<th>WATER DEPTH (m)</th>
<th>MAX. HORIZ. FORCE (kN)</th>
<th>MAX. VERTICAL FORCE (kN)</th>
<th>MAX. OVERTURNING MOMENT (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>41.0</td>
<td>+64,500 (10.0m &amp; 9 sec. wave)</td>
<td>+191,300 (10.0m &amp; 9 sec. wave)</td>
<td>-1,180,000 (10.0m &amp; 8 sec. wave)</td>
</tr>
<tr>
<td>21.0</td>
<td>+49,000 (10.0m &amp; 9 sec. wave)</td>
<td>+380,300 (10.0m &amp; 9 sec. wave)</td>
<td>+2,930,000 (7.6m &amp; 8 sec. wave)</td>
</tr>
</tbody>
</table>
The above peak values are not in phase. Vertical forces are generally much more because the Monocone has a projected area in plan about six times larger than that in elevation. Maximum vertical forces are in uplift and are more pronounced when the Monocone is in "shallow" water because of the proximity of the base to the passing wave.

**SOILS DESIGN CRITERIA**

A mobile drilling platform must be able to operate in a reasonably wide range of foundation conditions. Key foundation design criteria are shown below:

<table>
<thead>
<tr>
<th>Ultimate Strength in Shear:</th>
<th>Cohesive Soils</th>
<th>21 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cohesionless Soils</td>
<td>(Normal Load) x tan 30°</td>
</tr>
<tr>
<td>Ultimate Strength in Bearing:</td>
<td>215 kPa</td>
<td></td>
</tr>
<tr>
<td>Safety Factor Against Sliding Failure:</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Safety Factor Against Bearing Failure:</td>
<td>2.0</td>
<td></td>
</tr>
</tbody>
</table>

These safety factors are considered acceptable because in exploration drilling the site would be temporary. Also prior to setting the structure down on location, extensive soils investigations would be carried out to determine site suitability.

**STRUCTURAL DESIGN CRITERIA**

The concrete portion of the structure was designed by the ultimate strength method using ACI-1971 code and CSA-A23.3 - 1973 standard. The load factors used were comparable to those by the Norwegian classification society, Det Norske Veritas, for ultimate limit state. Under operating conditions structural design considered both a normal service and an extreme service condition to cover the requirement for rebar and prestressed steel to control cracking in the concrete.

**OPERATING CRITERIA**

The Monocone was designed to operate year-round as a bottom-founded drilling platform in water depths ranging from its light ship draft to a specified design maximum. Since the ice for the most part is impassable beyond the near-shore areas by normal means, the structure is designed with adequate storage capacity for fuel and other drilling consumable to drill a deep 6,100 m hole without resupply. Personnel transport and emergency supplies would be provided by helicopter and air cushion vehicle whenever possible. Details of rig specifications and storage capacities were discussed previously (Jazrawi and Davies, 1975).
MATERIALS

Due to the severe arctic marine environment special care was required in specifying the materials which would be used for the construction of the prestressed concrete hull, and the steel collar and superstructure. Most of the concrete in the hull was required to attain a strength of 41 MPa; but in critical areas of the hull and shaft 55 MPa concrete was specified. In locations of the structure subjected to freeze-thaw cycle (as in the tidal zone) air-entrainment or the use of special sealant would be required.

No special requirements were anticipated for the reinforcing and prestressing steel to avoid brittle fracture due to stress oscillations in low temperature service because the reinforcing steel would be subjected to axial stresses only and the post-tensioning steel would always be in tension with little stress change. Furthermore, most of the structure would be below the water line in relatively warm sea water. The structural steel used in the conical collar and superstructure was specified to have adequate fracture toughness, and in critical areas the carbon equivalent (CE) and the through-thickness reduction of area (RA) of a tensile specimen was specified to ensure resistance against lamellar tearing.

STRUCTURAL SYSTEM

Only the structural systems for the prestressed concrete hull and the steel conical collar are described here because these have unusual features. The steel superstructure is typical of an offshore oil drilling platform.

Concrete Hull

The Monocone hull including the column is an unusually complex prestressed concrete structure. In size it rivals some of the largest offshore structures built for the North Sea but due to the severe Arctic marine environment the Monocone possesses a very different shape and form.

Compared to the Ekofisk Tank, for instance, the Monocone has the following characteristics:

<table>
<thead>
<tr>
<th></th>
<th>41 Monocone</th>
<th>Ekofisk Tank</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Vol. (m³)</td>
<td>29700</td>
<td>79900</td>
</tr>
<tr>
<td>Regular Reinforcement (t)</td>
<td>4720</td>
<td>6980</td>
</tr>
<tr>
<td>Prestress Reinforcement (t)</td>
<td>1270</td>
<td>2990</td>
</tr>
<tr>
<td>Foundation Area (m²)</td>
<td>8450</td>
<td>6650</td>
</tr>
<tr>
<td>Towing Draft (m)</td>
<td>10</td>
<td>63</td>
</tr>
</tbody>
</table>

The higher percentage of reinforcement in the Monocone compared to the Ekofisk Tank is due mainly to:

- A substantial column protruding from a relatively thin disk, causing high bending stresses.
Shallow draft feature required minimizing concrete weight, in turn reducing concrete sectional area with a corresponding increase in steel reinforcement.

Fig. 4 shows a plan and section of the hull. The concrete hull has a radial framing system composed of intersecting radial and circumferential walls and frames, subdividing the hull into 24 watertight compartments. The outer 12 compartments are used for ballast water. The remaining compartments are used for fuel oil, drilling water and fresh water.

The protective perimeter of the hull was designed to a local ice pressure of 8300 kN/m² on any 4.6 m² of surface area. The resulting heavy wall accounts for nearly 25% of the concrete and steel reinforcement contained in the concrete structure.

Both the top and bottom slabs are designed as a waffle slab in the outer half and as a solid slab over the inner fuel and storage tanks. The bottom slab accommodates six layers of post-tensioning as they pass on either side of the shaft. The central concrete column is cylindrical with a wall thickness of 1.5 m. Below the top slab of the hull, the shaft flares into a conical section for better stress distribution and to provide space for the pump room. An elevator shaft and piping and a 4.5m diameter drilling shaft large enough to accommodate a subsea blow out preventer are located within the concrete column.

To gain an understanding of the effect of different prestressing tendon layouts on the top and bottom slab of the hull, a finite element analysis was carried out on the slab isolated as a disc in which tendon forces were represented by point loads. Several tendon layouts were studied, including bypass tendons, direct tendons and circumferential tendons, Fig. 5. The analyses showed that these tendon layouts had equivalent effect on the radial and circumferential stresses in the outer regions of the slab. It was noted that bypass and circumferential tendons were essentially equivalent everywhere but there were significant differences in the vicinity of shaft between the bypass and the direct tendons. These results are shown in Fig. 5.

Based on the stress analyses and practical considerations, the following tendon layouts were developed.

- Vertical Tendons in Shaft - These tendons apply a precompression to the shaft walls to limit the tensile stress in the concrete under ice impact from any direction.

- Bypass Hull Tendons - These are located in the bottom slab in six layers. They stretch across the hull radially but bypass the drilling shaft by curving around it.

- Direct Hull Tendons - These are located radially in the top and bottom slabs and are anchored between the shaft and hull perimeter.

- Top Slab Circumferential Tendons - These are located near the shaft and arranged in concentric rings with each ring consisting of four individual tendon lengths spanning 90 degrees.

- Perimeter Wall Circumferential Tendons - These tendons are located in concentric rings in the perimeter wall and composed of twelve individual segments spanning 30 degrees.
Vertical Wall Tendons - These consist of high strength rods and are provided in all vertical walls to induce some precompression so as to reduce cracking and increase the shear capacity of the wall.

Conical Collar

A plan and section of the conical collar are shown in Fig. 6. The framing system of the conical collar is radial. Each half has six radial sectors. There is a triangular trim tank at the lower extremity of each half which allows the conical half to float with its mating surface vertical. The two halves of the conical collar would be joined together in the floating mode using two guide pins and twelve locking pins. After mating, locking is achieved by means of hydraulic wedges.

Hydraulic jacks capable of clamping the collar to the shaft are located in watertight chambers in the upper and lower sections of the collar. The remaining internal space is used as ballast compartments for ballasting the collar, which has a light ship draft of about 1.8 m, to the design draft of about 12.2 m.

The collar structural system is designed to transmit the ice loads to the shaft at the location of the jacks. Design of the skin and scantlings was to an ice load of 8300 kN/m² on any 4.6 m² of the entire skin area. A 12.5mm steel abrasion wrap is provided on the skin.

In addition to the trim ballast, joining and clamping systems, the conical collar also has a roller and braking system to allow controlled sliding of the column through the collar during set down and lift off. The collar also has a skin heating system using engine waste heat augmented by auxiliary heat to reduce or eliminate adfreeze.

FOUNDATION SYSTEM

The Monocone would be seated on 2 feet minimum sand blanket screeded level. Lateral loads on the Monocone would be resisted by friction between the roughened base of the Monocone and sand blanket, and between the underside of the sand blanket and the underlying seabed. The Monocone base area is sufficient to transmit the horizontal load in shear through the weak 21 kPa silty seabed. The 'Seating on Sand Blanket' concept made possible by the heavy weight of the structure eliminated the need for shear keys and their inherent disadvantages which were necessary for the lighter structure of the steel Monopod (Jazrawi and Davies, 1975). For the 41 WD Monocone the ultimate sliding resistance was about 173500 kN for both the sand blanket (Ø = 30 degrees) and the silty seabed 21 kPa.

NAVAL ARCHITECTURE

Desk and hydraulic model studies were carried out to assess the behaviour of the 41 WD Monocone and conical collar in up to 12 m seas.

The model studies were carried out by the Marine Dynamics and Ship laboratory of the National Research Council in Ottawa. Plexiglass models of the hull and collar external and internal details were constructed to a scale of 1 in 60.
The Monocone was tested for towing resistance and course stability in a 4.5m x 61 m towing tank at prototype speeds of 0 to 6 knots in calm water. The increase in resistance due to waves was determined for a range of regular wave heights (1.2 - 12m) and wave periods (7-16 seconds). The tests showed that the Monocone would hold course stability using a simple line and bridle arrangement. Towing resistance was found to be quite manageable. For 4 knot tow in 3 m seas with collar on board it was estimated that a total of 14000 tug horsepower would be required. This modest horsepower is attributed to the sloping sides and rounded bottom of the hull perimeter.

The hull exhibited large transverse stability as would be expected by a 45.7 m metacentric height and 72 degrees stability range. The towing and seakeeping tests demonstrated that 0.75 m freeboard on the Monocone hull below the scuppers did not jeopardize the considerable stability of the 107 m diameter Monocone hull. The hull's sloping perimeter and roof, however, caused the top of the hull to become easily awash by waves but this also did not significantly effect floating stability.

The course stability of the collar alone was judged to be satisfactory. For 4 knot tow with 3 m seas it was estimated that a total of 4600 HP would be required. Ballasting of half of the conical collar to maintain the mating face vertical was found to be possible as predicted by desk calculation.

The most important aspect of the naval architecture concerns the set down and lift off of the Monocone. In 41 m of water, the model tests verified that while a level set down was not possible with a 52 m diameter conical collar, tilt set down and lift off were possible. (Fig. 7). Two methods were considered. The first, ballast control with free sliding collar, did not completely eliminate free fall. The second, ballast control with controlled collar movement, allowed gradual lowering of the hull under complete control. Although level set down had not been achieved during model testing, desk calculations showed that a 67 m base diameter conical collar would allow level set down and lift off. Near level set down with barge assist was also tested. Preliminary desk calculations which took barge water plane into consideration had suggested that barge-assist set down may be feasible by adopting a 6 degree heel angle by differential ballasting and gradually lowering the hull with the upper end of the hull supported by two barges (Fig. 8). Tank tests, however, showed that on total submergence, the hull became unstable both transversely and longitudinally which caused the individual cable loads to shift dramatically and for the hull to flip over on the side opposite the cables.

Tests were performed to assess the possibility of Monocone relocation over relatively short distances by deballasting close to neutral buoyancy and dragging the hull at low heel angles with its lowest point sliding on the seabed. Such a towing procedure would eliminate the need to refloat the structure and may make it towable in ice conditions.

The tests revealed that the model had a tendency to yaw and cartwheel. This tendency may be reduced by providing some means of lateral seabed contact such as contact shoes. Furthermore, the tests showed that the horsepower requirements for hydrodynamic drag alone were much higher than surface tow. The results for the 9 degree heel angle were as follows:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Speed (knots)</th>
<th>Horsepower (HP)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drag tow (9° heel)</td>
<td>2</td>
<td>20,000</td>
</tr>
<tr>
<td>Surface tow</td>
<td>2</td>
<td>2,400</td>
</tr>
</tbody>
</table>
CONCLUSIONS

This paper has endeavoured to demonstrate the viability of the Monocone as a relo-
catable drilling structure in the shallow waters of the Beaufort Sea. The main
conclusions are:

1. Comprehensive ice and wave design criteria are available to enable the
design of conical bottom-founded structures in ice-infested arctic waters.

2. The Monocone is a versatile and efficient offshore structure capable of
being set down in deep water under complete control. It is a year-round
and self-contained concept capable of operating without being resupplied
for long periods.

ACKNOWLEDGEMENTS

The authors wish to thank Imperial Oil Ltd. for permission to publish this paper.
They are particularly indebted to Mr. A.D. Boyd of Swan Wooster Engineering for
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MONOCONE 41
MAXIMUM DESIGN WATER DEPTH = 41 m

ESTIMATED QUANTITIES

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE</td>
<td>29,700 m³</td>
</tr>
<tr>
<td>REINFORCING STEEL</td>
<td>4,720 t</td>
</tr>
<tr>
<td>POST TENSIONING</td>
<td>1,270 t</td>
</tr>
<tr>
<td>HULL &amp; SHAFT</td>
<td>76,200 t</td>
</tr>
<tr>
<td>SUPERSTRUCTURE &amp; EQUIPMENT</td>
<td>4,940 t</td>
</tr>
<tr>
<td>STEEL CONICAL COLLAR</td>
<td>5,990 t</td>
</tr>
<tr>
<td>LIGHTSHIP WEIGHT</td>
<td>87,100 t</td>
</tr>
<tr>
<td>LIGHTSHIP DRAFT</td>
<td>10 m</td>
</tr>
</tbody>
</table>

FIG. 1
COLLISION CAPACITY
Horizontal load from stopping ice island 200 m x 200 m x 18 m moving at 0.6 m/sec = 112 000 kN

HORIZONTAL ICE LOADS
45° CONE

FIG. 2

HORIZONTAL LOAD (KN)

SHEET ADFREEZE
CONTROLLED BY HEATING SYSTEM
RIDGE FAILURE AND SHEET RIDEUP
RIDGE FAILURE (14 m thick)
SHEET FAILURE (3 m thick)

CORRESPONDING CONE DIAMETER AT WATER LINE
WATER DEPTH

FIG. 3

SHEET RIDEUP ON MODEL CONE
HALF SECTION

FIG. 4
TENDON LAYOUT DEFINITION

FIG. 5
FIG. 7  SLIDING COLLAR SET-DOWN

FIG. 8  BARGE ASSIST SET-DOWN
RESPONSE OF AN OFFSHORE LPG STORAGE PLATFORM TO SIMULATED ICE AND WIND FORCES

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ABSTRACT

The paper describes the dynamic response analysis to simulated ice and wind forces of a prestressed concrete floating facility for recovery, liquifaction, storage and cargo handling of liquid petroleum gas (LPG). 'Three dimensional beam' and 'three dimensional variable nodes isoparametric thick shell' finite elements are the two structural idealisations used. The time histories of force records (ice plus wind) are generated by superposition of a constant mean (wind + ice) and the fluctuating parts, obtained from using a nonstationary random process, filtering a shot — noise through a second order filter with postulated properties. Representing the hydrodynamic effects in terms of the added water mass concept, the structural response is obtained. The need is indicated for i) more realistic force records (based on actual field measurements), ii) ice basin model testing, and iii) incorporation of frequency dependent added water mass.
INTRODUCTION AND REVIEW OF LITERATURE

Remote oil production locations, where pipelines are not technologically or economically feasible, either due to water depths or distance to the nearest shore facility, necessitate alternative systems for shipment of crude oil. One such alternative is the floating storage, which has been used with increasing frequency in mild-sea areas. Floating storage uses a vessel converted for permanent or semi-permanent mooring located in the drilling field.

Wiegel, Dilley and Williams (1958, 1969) conducted tests with a 1:49 scale model of a moored, submerged buoyant offshore oil storage tank, with neutral buoyancy mooring lines. Measurements of large mooring line forces indicated the need for considerable care in mooring. The most critical wave period was about 10 sec. Gregg, McClure, Williams and Norton (1969) presented the detailed design aspects of a remote-controlled, instrumented and unmanned floating oil storage vessel for use by the Dubai Petroleum Company. Catenary mooring was analysed for maximum steady loads with superimposed motion due to wind, waves and currents. Hooper (1971) carried out a comparative analysis of various competing systems, including island terminals, monomoors, telescoping support platforms, and a floating stable platform for storage and transfer of crude oil. Based on socio-political acceptability as well as economic feasibility, the stable platform emerged as the most desirable alternative. Feizy and McDonald (1972) described the successful application of Pazargard, the world's largest crude-oil storage barge, as a floating self-contained storage, desalting and crude exporting facility. Maddox (1972) conducted model tests with three types of single-point moorings; conventional catenary-moored SPM buoys, the newly developed single anchor-leg mooring and turret mooring, with directional irregular waves simulating limited fetch, open-ocean sea conditions. A procedure, based on energy correlation, has been developed for the design of open-ocean, all weather single-point moorings. Remery and Van Oortmerssen (1973) presented methods for the determination of mean forces induced by waves, wind and current on tankers, barges and other structures. Glanville Knight, Basile and Watanabe (1973) confirmed that large self-supporting oil storage and export problems. The design and construction of one million barrel floating storage barge serving the offshore oil fields in the Java Sea were described. Gerwick (1971, 1973) presented design criteria for the development of prestressed concrete ocean-going vessels, in view of greater tensile and compressive strengths, nonbrittle behaviour at low temperatures, economy, low maintenance cost, good impact resistance and a favourable mode of failure under accident conditions.

Bomze and Harris (1974) presented the method for computation of the three translatory motions - heave, surge and sway, and the three rotary motions - roll, pitch and yaw, of a ship moored in regular waves of given height and period approaching from an arbitrary direction. Patton and Johnson (1974) described the deepwater unloading facility off the Texas Gulf Coast, including the use of multiple, single point mooring systems, located around a central platform complex in approximately 100 ft. of water and connected to an onshore storage facility by multiple, large diameter submarine pipelines. Van Opstal, Hans, Salmons and Van Der Vlies (1974) described the computer programme, MOSAS, for determining the motion and stress behaviour of floating structures and semisubmersible units in irregular sea states. Yumori (1975) discussed the single point mooring concept for floating oil storage tankers. The design, fabrication, installation and operations of a Single Anchor Leg Mooring (SALM) system were presented by Kiely,
Pedersen and Gruy (1975). SALM is used to permanently moor a tanker which is receiving the product directly from the production platform. Terry (1975) recommended the floating offshore LNG liquefaction facility in oil fields, too far from shore or in too great a water depth, as a cost-effective alternative. Andersen (1975, 1976) discussed several major factors which favoured the floating facility for recovery, liquefaction, storage and cargo handling of the LPG; structural problems from earthquakes are avoided. Installation of a complete operating facility could be accomplished near sources of processing equipment and skilled manpower. Time and cost demands of a floating structure are less than those of a land-based installation.

Wichers (1976) presented the results of a study on the dynamic stability and the natural frequencies of the modes of vibration of the tanker in the horizontal plane, including the effect of the system parameters in steady current and wind. A procedure was presented to obtain practical values of added mass and damping for calculations of the natural frequencies and stability of the system. Harwig (1976) recorded single-point mooring forces and tanker motion data aboard the tanker THEOTOKOS at Ekofisk in the North Sea. The maximum forces in the bow hawser, connecting the THEOTOKOS to the single-point mooring buoy, were essentially a function of wave height. The tanker experienced long period yaw cycles, while moored, which exceeded those observed in model tests. The model test results tend to overestimate the bow hawser forces observed on the tanker and underestimate the pitch and roll motions observed. Muga and Freeman (1977) studied analytically the behaviour of a vessel moored to a single point using the impulse response functions and constant inertia coefficients in a time domain formulation of the governing equations of motion. The excitations considered were due to wind and/or current from any direction. The output, consisting of time-histories of vessel displacement (surge, sway and yaw) and hawser live loads, was compared with reduced scale model results. Benham, Fang and Petters (1977) developed a standardised set of shape coefficients for use by the marine industry to predict forces and moments due to wind and currents acting on moored very large crude carriers. Van Sluijs and Blok (1977) presented the results of a model study on the behaviour of anchor lines, used in the offshore mooring of a wide variety of floating structures. Special attention was given to the dynamic behaviour of the mooring system, resulting in higher tensions than those that would be obtained from calculations based upon the static catenary theory. The existence and significance of dynamic effects was confirmed by the results of forced oscillation tests with model anchor chains in calm water.

Naess and Borresen (1977) presented a theoretical study of the effects, which different methods of simulating an irregular seaway had on the prediction of the second order behaviour of moored offshore structures from model tests. The irregular seaways were simulated by two methods; one by superimposing a finite number of regular wave trains, the other with filtering white noise to obtain a time series with the correct spectral shape. Oortmerssen (1974, 1976, 1977) described the mathematical simulation of the behaviour of a moored ship in waves. The method was based on the equations of motion in the time domain according to Cummins, while the hydrodynamic loads on the ship were obtained by means of a three-dimensional source technique. Gruy and Kiely (1977) discussed the design, fabrication and installation of two identical single anchor leg mooring (SALM) terminals serving as crude oil export terminals. These were designed to moor safely and to load large tankers in severe environmental conditions. Model tests conducted on the fluid swivel/loading hose system were also discussed. Flory and Synodis (1975), and Flory and Poranski (1977) discussed important aspects.
of designing single point moorings including the various parameters, model tests, limiting loads and environments. Burns and D'Amorin (1977) described buoyant towers for the first phase development of Garoupa field, offshore Brazil. The process tanker was moored to the larger buoyant tower by means of a structural yoke.

**STATEMENT OF PROBLEM**

**Structural Configuration**

The example structure considered is a floating platform, similar to the Atlantic Richfield Company Liquid Petroleum Gas facility (ARCO-LPG), used in the Java Sea at the Arjuna field (Fig. 1). The platform hull, shown in Fig. 2, consists of three cylindrical barrel shells, whose shape most efficiently resists a hydrostatic pressure of 14.6 t/m², and also reduces the vertical span for side shell pressure (Andersen, 1977). Y-shaped sides and Y-shaped longitudinal bulkheads eliminate the necessity for transverse ribs and longitudinal stiffeners, and minimize the transverse bending moments in the deck. The hull is constructed of prestressed concrete, which offers the advantages of lower initial cost, corrosion-free performance in salt water, less maintenance, superior fire resistance, and better resistance to cyclic loading and fatigue. The development of design criteria took into account the loading for ships applicable to steel vessels of comparable dimensions, as required by the rules of American Bureau of Shipping. Pairs of circular steel LPG tanks rest end to end inside the cells on suitable tank saddles. A similar number of tanks, six in all, are saddle-mounted on the platform deck. The platform is anchored to a single buoy permanent mooring system, and is free to weathervane in the wind, current and drifting ice environment.

**Modelling**

The structure is modelled by i) three-dimensional beam elements and ii) variable-number-nodes thick shell and three-dimensional isoparametric elements (Fig. 3). The hydrodynamic effects of the sea water are represented by frequency-independent added masses. The effects of buoyancy on the LPG is idealised with the use of one-dimensional axial stiffness boundary elements representing the supporting action of the sea water. The linear elastic mooring system is idealized using beam elements.

**Loading (Ice and Wind Forces)**

In view of the non-availability of ice and wind force records for large floating structures, artificial force records are generated. The method makes use of the similarity between randomly oscillating force records obtained for ice and wind, and seismic records. In this investigation, the sum of the means of the ice and wind forces is determined, and superposed with the fluctuating parts generated using the method described by Swamidas, Reddy and Purcell (1976). The mean part of the ice force is determined by summing the shear stresses (the Coulomb failure criterion for pack ice proposed by Sodhi, 1977 and the relationship suggested by Assur, 1975) acting on the wedge shaped accumulation of ice in the direction of the flow. The mean part of wind force is computed using the wind for Labrador City, data specified by National Building Code of Canada (1975). The artificially generated time histories are used as the forcing functions in the time-domain response analysis (Bathe, Wilson and Peterson, 1974) of the floating LPG. The method uses a nonstationary random process obtained from filtering a shot
noise through a second order filter with postulated properties. Assur's (1975)
formula is used in estimating the mean ice force record. The overall
force per unit width, Fig. 4, of the idealized failure plane passing through the
single buoy mooring and the junction of the cross sectional and longitudinal faces
of the LPG is given by

\[ F = c_2 \frac{2\theta}{\pi} F_1 + (1 - \theta_1) F_2 \]  

(1)

where

\[ c_2 = \text{a coefficient less than 1}, \]
\[ \tan \phi = \text{friction coefficient of ice to ice}, \]
\[ \tan (\theta_1 + \phi) = \frac{6c}{\sqrt{2} \pi / 4} \frac{\sigma_c}{\sigma_f} \frac{h}{L} \]  

\[ \frac{c}{\sigma_f} = \text{ratio of crushing to flexural strength}, \]

and

\[ \frac{L}{\sigma} = L \frac{h}{3/4} \]

in which

\[ L = 4 \frac{E}{3k(1-\nu^2)} \]
\[ E, \nu = \text{Young's modulus and Poisson's ratio of ice}, \]
\[ F_1 = \text{average bending force}, \]
\[ \frac{2\theta}{\pi} \frac{\sigma_c h^2}{6L} \sqrt{E} e^{\pi / 4} \cos (\theta_1 + \phi) \]
\[ F_2 = \text{crushing force} = \frac{c}{c} \cdot \frac{\sigma h}{h} \]
\[ c = \text{propportionality constant assumed to be unity} \]

The shear force between the failure plane and the surrounding ice sheet is given by

\[ S = \sigma \frac{hL}{p} + F \tan \eta \frac{L}{p} \]  

(2)

where

\[ \sigma_s = \text{cohesive strength of broken ice masses} \]
\[ \eta = \text{angle of internal friction for broken ice} \]
\[ \eta = \text{masses, taken to be equal to 30° (Sodhi, 1977).} \]

The component of the force in the direction of the ice flow is given by \( S \cdot \cos \alpha \).
The mean part of the force is assumed to be produced by the impact of a moving
ice floe of constant thickness of 1.5m. The typical total (sum of the means
of the ice and wind forces, and the fluctuating force) record for a typical
node is shown in Fig. 5. Since wind induced vibrations require several minutes
to build up to their maximum amplitude, wind velocities, averaged over several
minutes, are used for the calculation of the mean force (26). The hourly wind
speeds that would have an annual probability of 1/100, are used in the estimation
of pressures for the Labrador Coast of Newfoundland.
ANALYSIS

Theory

The equations of motion for the structure, discretized as a finite element system, are written as

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{P(t)\} \quad (3)$$

where

\{"\} and \{\'} denotes double and single differentiation with respect to time,

\{u\} = nodal displacement vector,

\{M\},\{C\},\{K\} = mass (lumped structural and added water masses),

damping (viscous equivalent of the structural and hydrodynamic), stiffness matrices respectively,

and

\{P(t)\} = applied dynamic force (ice and wind) vector

These equations are decoupled and the modal solutions obtained by time-history analysis. Decoupling implies the assumption that the damping matrix \{C\} satisfies the orthogonality condition

$$\{\phi_m^T\}[C]\{\phi_n\} = 0 \quad (n \neq m) \quad (4)$$

where

\{\phi_m\} = eigenvector for the mth mode

The modal damping is expressed as a fraction of the critical value, and the damping matrix is assumed proportional to the mass matrix.

RESULTS AND DISCUSSION

The first two frequencies obtained for the 'beam model' are 0.53 Hz and 2.10 Hz compared to 0.47 Hz and 1.15 Hz for the 'shell model'. As the increase in accuracy, associated with three-dimensional modelling, is offset by the coarseness of the shell mesh, further studies are needed to adequately evaluate the 'beam' and 'shell' models for frequency analysis. For the 'Shell Model' the typical displacement time-history at a particular nodal point and the axial stress time-history for the mooring member are shown in Figs. 6 and 7. The analysis uses a damping value of 10% (critical damping for all modes), which constitutes the viscous equivalent of the hydrodynamic drag and structural damping (material). The deficiencies associated with the postulation of filter properties can be overcome by using actual wind and ice force time-history measurements. Ref. 34 used the field measurements of ice forces at Cook Inlet, Alaska. The actual ice failure patterns for a single point moored LPG need to be determined from ice model basin tests. The influence of frequency-dependent added water masses should be incorporated into the response analysis by using fluid finite elements.

ACKNOWLEDGEMENT

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REFERENCES


FIG. 1 CONCEPTUAL MODEL OF LPG PROCESSING AND STORAGE FACILITY ON SINGLE-BUOY MOORING IN THE JAVA SEA
FIG. 2 SCHEMATIC OF THE LPG
FIG. 3  FINITE ELEMENT IDEALIZATION (Cf Fig. 2)
FIG. 4 FORCES ON AN IDEALIZED FAILURE PLANE OF A MOVING ICE SHEET
FIG. 5 TYPICAL COMBINED ICE AND WIND FORCE RECORD FOR THE LPG
FIG. 6 TIME HISTORY OF DISPLACEMENT AT TOP CORNER OF REAR FACE (LEFT)

FIG. 7 TIME-HISTORY OF AXIAL FORCE IN MOORING MEMBER
ICE RUBBLE FIELDS IN THE VICINITY OF ARTIFICIAL ISLANDS

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ABSTRACT

Ice rubble fields increase the ice forces exerted on artificial islands in the late winter. The islands, built as exploratory drilling platforms in the shallow waters of the Southern Beaufort Sea, induce the formation of rubble piles in the early winter when there are extensive thin ice movements. Individual rubble piles up to 13 m high and rubble fields extending over distances of the order of 10 island diameters have been observed. As the winter progresses, the rubble consolidates and sufficient freezing occurs by late winter that the rubble field acts as a rigid annulus around the island. Observations confirm that the active zone of subsequent ice failure is generally at the periphery of the rubble field. Calculations indicate that although parts of the rubble field must be grounded, they cannot significantly increase the sliding resistance of the island-rubble combination. Therefore, the increased effective diameter leads to higher ice forces on the island when the thick, late winter ice crushes against the edge of the refrozen rubble field.

INTRODUCTION

Exploration for petroleum reserves in Arctic regions has provided incentive for increased understanding of ice interactions with offshore structures. Imperial Oil Limited has used artificial islands constructed by various techniques (Garratt and Kry, 1977 and de Jong et al, 1975) as drilling platforms in the shallow waters of the Beaufort Sea. The design of these islands must consider all of the environmental influences and in particular the loads imposed by winter ice (Croasdale and Marcellus, 1977). Although the ice is landfast for most of the winter in the shallow waters of the Beaufort Sea, sufficient movement can occur to laterally shift an improperly designed island. Proper design requires sufficient overburden to cause ice failure before the sliding resistance of the island is overcome.

An island is a wide structure compared to the ice sheet thickness. During ice movements, the failed pieces of the ice sheet cannot clear around an island as they can around relatively narrow structures. Therefore, during the early winter when the ice is thin and subject to large movements, the failed ice accumulates as a rubble field around the island.

The implications of the presence of this rubble field later on in the winter when the ice sheet reaches its greatest thickness and can impose the largest loads must be considered for the proper design or possible defense of an artificial island.
FORMATION

The formation of the rubble field around an artificial island depends on the weather history once freeze-up has commenced. The variations which can occur from year to year result in very different rubble fields as illustrated in Figures 1 and 2. These are aerial photos of the remains of the artificial island Netserk B-44 in February 1976 and November 1976 respectively. The island was built during the summer of 1974 in 5 m of water as a sandbag retained island with steeply sloping sides (1 in 3). The exploration well was drilled during the 1974-1975 winter and in subsequent summers severe erosion of the abandoned island occurred. The original waterline circumference is drawn on Figures 1 and 2.

Figure 1 shows that during the winter 1975-1976, there was a single large movement of a relatively thick ice sheet. The island acted as a rigid indentor when the ice sheet moved and formed a wake behind the island. The ice in line with the island during this motion formed most of the rubble on the northwest side of the island. This large movement occurred in late October 1975 and at that time, the ice sheet could be 0.3-0.5 m thick and the mean air temperature near -15°C.

To quantify the upper topography of the rubble, height profiles were determined along several lines from stereo-pair photographs taken in November 1975 and February 1976. Two of the survey lines, A-A' and B-B' are indicated in Figure 1 and the height profiles along these lines at the two times are plotted in Figure 3. They show that although there was no significant change in the rubble field after its initial formation, there were additional movements generating some new rubble, particularly to the south of the island. Figure 3 also shows that there was a general settlement of the main part of the rubble field with a net decrease of the peak sail height from 7 m to 6 m.

In the winter of 1976-1977, there was a much different formation history for the rubble (Figure 2). Large movements of the ice sheet did not occur once it became relatively thick. Rather it appears that repeated movements of very thin ice occurred with time intervals between some movements sufficient to allow consolidation of the rubble and cause subsequent motion of thin ice to extend the pile seaward. This concept of formation is supported by the presence of the ridge indicated by the arrows in Figure 2.

In the shallow waters where artificial islands are presently built, the rubble is soon grounded during its formation. The physical processes that occur prior to this grounding are exactly those associated with pressure ridge formation in thin ice (Weeks et al, 1971). Relative to the ice forces for which the island is designed, the forces to generate the rubble are quite small.

The occurrence of grounding should not lead to an increase in the forces required to build the rubble. Parmeter and Coon (1973) used an energy approach to estimate the force to build a given floating ridge, and they concluded that for low ridges, most of the work done by the force in building a ridge appears as gravitational potential energy in the ridge.

If a similar approach and assumption is used for a grounded ridge, with a given sail height $H$, then the force to build the ridge is found to be less than for a floating ridge with the same sail (Ralston, 1977).

If the geometries of the two cases are as shown in Figure 4, then according to Parmeter and Coon (1973) if the sail and keel heights of the floating ridge are
related such that

\[ \frac{H_k}{H_s} = \frac{\rho_i}{\rho_w - \rho_i}, \]

where \( \rho_i \) and \( \rho_w \) are the density of the ice and the water respectively, then the effective stress exerted by the ice sheet to generate the stored potential energy is given by

\[ \sigma_f = \frac{1}{2} \rho_i g H_s, \]

where \( g \) is the acceleration due to gravity.

Equation (1) implies that along the vertical symmetry plane of the ridge, the ice is locally in isostatic equilibrium. However, if the keel extends further from the center than the sail as indicated in Figure 4, there is a net buoyant force on the ice sheet. When this force becomes large enough, the ice sheet is failed at the edge of ridge which then grows in extent rather than height. This is the height limiting mechanism for floating ridges.

For the grounded ridge with the geometry and symbols defined in Figure 4, the potential energy per unit length of ridge \( E \), is given by

\[ E = (1 - \gamma) (\rho_w - \rho_i) g \left[ \frac{\rho_i H_s^3}{3(\rho_w - \rho_i) \tan \theta} + \frac{H_k H_s}{3 \tan \phi} + \frac{WH^2}{2} \right], \]

where \( \gamma \) is the ridge porosity which is assumed to be equal in the sail and keel during the ridge formation.

The volume of ice per unit length of the ridge, \( V \), is given by

\[ V = (1 - \gamma) \left[ \frac{H_s^2}{\tan \theta} + \frac{H_k^2}{\tan \phi} + \frac{WH}{\tan \phi} \right]. \]

This volume of ice per unit ridge length must equal the product of the ice sheet thickness and the total extent of the sheet movement \( U \) which generated the rubble.

Both the energy and volume per unit ridge length are functions of the sail height \( H_s \). The width \( W \) is also a function of \( H_s \) and with the assumption that as the ridge grows, the horizontal extents of the sail and keel increase at the same rate, then

\[ \frac{dW}{dH_s} = \frac{2}{\tan \theta}. \]

Thus the effective stress in the ice sheet applied to create the grounded ridge is given from Equations (3), (4) and (5) as

\[ \sigma_g = \frac{1}{t} \frac{dE}{dH_s} \cdot \frac{dH_s}{dU} \]

\[ = \sigma_f \left[ \left( \frac{\rho_w - \rho_i}{\rho_i} \right) \left( \frac{H_k}{H_s} \right)^2 + 1 \right] / \left( \frac{H_k}{H_s} + 1 \right). \]
Equation (6) states that \( \sigma_g < \sigma_f \) since once the ridge grounds

\[
\frac{H_{km}}{H_s} < \frac{\rho_l}{\rho_w - \rho_l}.
\]  \( \text{(7)} \)

Therefore, once the ridge grounds, the force required to continue increasing the gravitational potential energy for a given sail height is decreased.

This result is one aspect of the fact that most of the potential energy is stored in the keel of a ridge, and the keel of a grounded ridge is restricted to the water depth. The potential energy, \( E_k \), stored in the keel of a floating ridge (geometry as in Figure 4) is proportional to the energy, \( E_s \), stored in the sail,

\[
E_k = \left( \frac{\rho_w - \rho_l}{\rho_l} \right) \tan \theta \left( \frac{H_k}{H_s} \right)^3 E_s, \tag{8}
\]

assuming that the ridge porosity is equal in the sail and keel. For \( \theta = 24^\circ \), \( \phi = 33^\circ \) (Weeks et al., 1971) and a sail to keel ratio of 4.5 (Kovacs, 1970), Equation (8) states there is 7.5 times as much energy in the keel as in the sail. This disparity increases as \( H_k/H_s \) approaches the isostatic ratio of 8.4 and as freezing occurs in the submerged pore volume of the ridge.

The influence of grounding on the ridge building process is to remove the limit to the sail height which exists for floating ridges. For floating ridges (Parmeter and Coon, 1972), the limit height is achieved and lateral growth of the ridge begins once the ridge is sufficiently large that it imposes sufficient flexural loading on the ice sheet to fail the sheet at the edge of the ridge. However, a grounded ridge is supported by the sea floor and, therefore, the maximum load it can apply to the ice sheet is limited and not dependent on the sail height. In the landfast ice region in which Imperial has constructed islands, sail heights in the rubble around the islands have reached 13 m.

Alternate mechanisms provide height limitations for the grounded rubble. The ultimate limit is provided by the crushing strength of the ice sheet. However, before this is achieved, other mechanisms are likely activated. As an example, if a pile were growing by a thick sheet riding up the seaward edge of an already existing pile, eventually a height would be achieved when the frictional force could provide a sufficient load to trigger an instability between the ice blocks riding on the irregular slope of the underlying pile-up. Particular height limiting mechanisms will depend on particular rubble and ice movement conditions.

The grounding makes the rubble field a fixed object with respect to the small motions of the adjacent floating ice sheet. Therefore, small ice sheet movements such as vertical tidal oscillations or wind induced horizontal motions lead to an active crack on the outside periphery of the rubble field. This crack can be seen to extend around most of the rubble field perimeter in Figures 1 and 2.

The active crack represents a zone of weakness where ice failure will occur for extensive ice movements. The effective failure stress for the ice sheet at the periphery of the rubble field will depend on the failure mode in this active zone. Equations (2) and (6) provide only lower bounds for the stress and take no account of failure modes.
An alternate failure mechanism would be a flexural failure due to an eccentric loading on the vertical face of the active crack as it closes. All else being constant, the horizontal load per unit width which could be supported in this failure mode varies as the square of the ice sheet thickness. That is, the horizontal load could increase rapidly for increasing ice sheet thickness. For sufficiently thick ice sheets, this mechanism must be transformed into a crushing failure, since for a constant effective crushing stress, the horizontal load per unit width varies only as the ice sheet thickness. The near vertical faces on an active crack, kept open by tidal motions promote this transformation from flexural to crushing failure by increasing the likelihood of the ice sheet jamming against the rubble periphery. The possibility of the ice having to crush against the periphery of the rubble field once it achieves its maximum winter thickness of more than 2 m cannot be ignored for the purposes of island design. However, the crushing will likely be governed by local contact effects since the edges of the active crack will, in general, not match as it is closed by an arbitrary movement.

CONSOLIDATION

Consolidation of a rubble pile begins once interblock motion ceases. The principal mechanism is freezing of the seawater which fills the pore volume of the keel. Sintering and bonding as a result of brine drainage consolidate the blocks in the sail, but these processes are relatively slow especially in the cold of winter.

The initial pore volume, initial ice temperature, sail height, average air temperature and wind speed all influence the solidification process. The number of influencing factors accounts for the wide variability of consolidation noted by Weeks et al (1971) for free floating ridges.

The initial pore volume coupled with the initial ice temperature has a considerable influence on the amount of ice that can be formed immediately (Weeks and Kovacs, 1970). This ice forms at the contact points of the blocks as latent heat in the seawater warms the colder ice. An estimate of this effect is given by assuming a heat capacity C (of the order of 3 J·g⁻¹·K⁻¹) for the ice and an effective latent heat L (about 260 J·g⁻¹) (Schwerdtfeger, 1963). Then if the initial rubble porosity is γ, the fraction of a unit volume of sea water which could be frozen is approximately \((1-\gamma)C\Delta T/L\) where the mean ice temperature is \(\Delta T\) degrees below the freezing point. If the rubble is sufficiently solid or the mean ice temperature is sufficiently cold, then it would be possible for the ridge to completely solidify. This would require

\[
\gamma = (1 - \gamma) \frac{C\Delta T}{L}.
\]  

Equation (9) has been plotted in Figure 5 with the assumption that the initial temperature distribution in the ice sheet is linear and equal to the air temperature on the top surface. Porosities in the range 0.1-0.3 appear to be typical (Rigby and Hanson, 1976). Figure 5 suggests that it is unlikely that complete solidification would occur at once unless the blocks in the rubble were sufficiently small (thin ice) to be tightly packed and thus have a low porosity. Because the weight of the sail acts with a maximum compressive force at the waterline, there would be a tendency for a higher degree of consolidation nearer the top of the keel. Because grounding allows higher sails for thinner ice, the compaction effect should be more significant for grounded rubble than for free floating ridges.

After initial consolidation, heat transfer from the keel through the sail to the air will continue the solidification process. Grounding of the rubble will prevent
water currents from slowing its freezing. The sail will have a double-edged influence on the freezing of the keel. On the one hand, as an exposed mound of ice, heat transfer will be increased relative to the adjacent sheet ice, however, this same exposure causes the sail to act as a snow fence, accumulating snow and insulating the rubble field. Evidence of this action is seen in Figure 1.

The significance of consolidation is in the ability of the rubble field to transmit horizontal forces from the ice sheet to the island. In the worst case, these forces could be those necessary to crush the full thickness of the winter ice sheet. The stress to crush the ice sheet will be distributed throughout the consolidated thickness of the rubble pile. There will be a stress intensification approaching the island since the rubble has a wider extent than the island, but there will be a stress reduction whenever the rubble thickness exceeds the ice sheet thickness. If the rubble porosity is as much as 0.30, and the rubble is only 30 percent thicker than the ice sheet, there will be exactly the same solid ice area to transmit stress in a cross-section through the rubble or through the ice sheet. The rubble may not be able to transmit the same total load, however, due to stress concentrations in the vicinity of the inclusions. However, these stress concentrations will have significantly less effect as the porosity decreases. Furthermore, except for the most shallow waters, the grounded rubble keel is usually significantly thicker than the maximum ice sheet thickness. In the particular case of Netserk B-44, those keels which are grounded are more than twice as thick as the 2 m maximum ice sheet thickness. Therefore, for design purposes, one must assume that the rubble can transmit design horizontal forces from its periphery to the island.

Sliding Resistance

Grounded rubble offers resistance to a net horizontal force. The net weight of the sail which is not compensated by the buoyant force on the keel can generate a frictional force which must be exceeded to move the rubble. Failure of the rubble once this force is exceeded would be expected at the base of the keel since the normal compressive force due to the rubble weight decreases from a maximum at the water level to a minimum at the sea bed. Furthermore, the degree of consolidation of the ice blocks is expected to decrease towards the bottom of the keel. Failure in the seabed or in the keel will occur depending on which requires the smaller force. Assuming failure in the seabed is thus realistic or overpredicts the sliding resistance of the rubble.

The rubble is only grounded over a fraction of its total areal extent. The extent of grounding can be estimated by assuming that the surface topography reflects the keel location and that lateral transfer of vertical loads is negligible on the average. Generally, this assumption will underestimate the extent of the keel and hence, will overpredict the sliding resistance of the rubble. The grounded regions will then be those in which the sail height is greater than that which could be supported by the buoyant force on a keel just as deep as the water depth.

The height of sail \( h_{sb} \) which can be locally supported by the maximum possible keel, \( H_{km} \) is given by

\[
h_{sb} = \frac{(1 - \gamma_k)}{(1 - \gamma_s)} \frac{\rho_w - \rho_i}{\rho_i} H_{km}, \tag{10}
\]

where \( \gamma_k \) and \( \gamma_s \) are the average porosities of the keel and sail respectively which are not assumed equal after consolidation of the keel (\( \gamma_k < \gamma_s \)).
The ratio $H_{km}/h_{sb}$ from Equation (10) lies between the isostatic value of 8.4 and the mean keel to sail ratio of 4.5 for free floating ridges. For the particular case of Netserk B-44 in 5 m of water and taking a somewhat arbitrary value of 6 for the ratio $H_{km}/h_{sb}$, the height of sail supported by the maximum keel is approximately 0.8 m.

If $h_g$ is the local rubble height above sea level, then the grounded regions are those for which $h > h_g$. Taking the $j$th such region with a mean rubble height $h_{sj}$, the mean vertical stress $N_{pj}$ is given by

$$N_{pj} = (1 - \gamma_s) \rho_i (h_{sj} - h_{sb}).$$

(11)

Multiplying $N_{pj}$ by the region area $A_j$, and summing over all the grounded regions gives the total vertical force, $N_T$, of the rubble on the sea floor,

$$N_T = (1 - \gamma_s) \rho_i (h_s - h_{sb}) f A_R,$$

(12)

where the mean rubble height $h_s$ is the area weighted mean given by

$$\bar{h}_s = (\sum_j h_{sj} A_j) / f A_R,$$

(13)

and $f$ is the fraction of the total areal extent $A_R$ which is grounded.

The total sliding resistance $R_R$ of the rubble is proportional to $N_T$ and the total sliding resistance of the island $R_I$ is proportional to the weight of the island on an assumed failure plane. Assuming that the coefficient of friction in the seabed is equal to that in the island failure plane overpredicts the rubble sliding resistance since in general, island fill is a better material than seabed silt. With this assumption, the ratio of the rubble and island resistance can be written as

$$\frac{R_R}{R_I} = \left[ \frac{(1 - \gamma_s) \rho_i}{\rho_s} \right] \frac{\bar{h}_s - h_{sb}}{h'_{sf} + \rho'_s h'_{sf}/\rho_s} f \frac{A_R}{A_I},$$

(14)

where $\rho_s$ and $\rho'_s$ are the density and buoyant density of the island fill respectively, $h'_{sf}$ is the island freeboard, $h'_{sf}$ is the depth of the assumed island failure plane of area $A_I$.

Equation (14) shows there are four factors which influence the ratio of the sliding resistances of the rubble and the island. The first is the ratio of the average rubble density and the island fill density. Using a value of 2 Mg·m$^{-3}$ for island fill density and a ridge sail porosity of 0.20, the density ratio would be 0.37. The second factor is related to a ratio of rubble to island heights. Although this term clearly depends on the particular island and rubble combination, a value of order 1 would require very large average sail heights in relatively shallow water with a low island freeboard. From the height profiles determined for Netserk B-44 in February 1976, two of which are reproduced in Figure 3, a reasonable value for $h_s$ would be 3.6 m. For Netserk B-44, taking a freeboard of 4 m, an assumed failure plane 2 m below sea level and 0.8 m for $h_{sb}$, the height ratio factor is 0.56. The third factor, is the fraction of the rubble field area which is grounded. By combining Figure 1 and the measured rubble height profiles with the assumption that the sail conditions reflect the keel conditions leads to an estimate of 0.9 of the rubble being grounded. One expects that this factor will be less for more extensive rubble
fields as pictured in Figure 2. The fourth factor, the ratios of the areal extents of the rubble and the assumed island failure plane is normally the only factor which exceeds unity, and is likely the factor which varies most widely from year to year. Therefore, for the example of Netserk B-44 and to a greater degree of approximation for other island-rubble combinations, Equation (14) becomes

\[
\frac{R_R}{R_I} = 0.19 \frac{A_R}{A_I},
\]

Estimating the areas involved in Figure 1 suggests that the rubble might provide up to 1/3 of the total sliding resistance of the rubble-island combination. However, the rubble increases the effective diameter over which ice would have to fail by almost a factor of two. That is, the presence of the rubble would double the horizontal loads and only provide a 30% increase in the sliding resistance.

Since rubble resistance increases with area whereas the horizontal loads increase with the effective diameter eventually the rubble will benefit the island. That is, for a certain area and diameter of the rubble, the ratio of the horizontal load and the sliding resistance will be equal to that for the bare island loaded by the same ice sheet effective stress. Since the horizontal load is proportional to diameter for constant stress, this condition can be written as

\[
\frac{D_R}{R_R + R_I} = \frac{D_I}{R_I}.
\]

Assuming circular areas for the rubble and the island and substituting Equation (15) in Equation (16) leads to the condition

\[
\frac{D_R}{D_I} = 4.
\]

The rubble diameter must be four times the island diameter before the rubble is a benefit rather than a detriment. Equation (17) would be expected to be a lower bound, since generally the rubble sliding resistance was overpredicted.

For such large rubble fields, Figure 2 suggests that the circular assumption may be invalid. More sophisticated analyses would be appropriate to determine the influence of the rubble. In particular, consideration of failure of the rubble plate, and reevaluating the assumption of a rigid plate fixed to the island may be appropriate. In any event, Equation (17) clearly suggests that the rubble fields will increase possible ice loads on artificial islands.

CONCLUSIONS

Rubble fields increase the total horizontal loads experienced by artificial islands. The presence of a rubble field increases the effective cross-section against which a moving ice sheet must fail. At the same time, the additional sliding resistance of the grounded rubble does not significantly add to the sliding resistance of the islands.

There are no grounds to expect high forces during the formation of the rubble from thin ice. Energy considerations suggest that it takes less force to build a grounded rubble pile with the same sail as a floating pressure ridge. However, the grounding fixes the rubble to the island and promotes the formation of an active crack at
the periphery of the rubble. Once the ice sheet achieves its maximum 2 m thickness ice sheet crushing at the periphery of the rubble field must be considered to be the design ice failure mode.

Sufficient consolidation of the rubble is possible, due to initial interblock freezing and subsequent heat transfer to the air that ice forces can be transmitted to the island with the rubble acting as a rigid annulus. The degree of consolidation of the rubble field necessary to transmit ice forces to the island decreases as the rubble thickness increases.

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REFERENCES


Figure 1. Aerial photograph of the artificial island, Netserk B-44, February 4, 1976.

Figure 2. Aerial photograph of the artificial island Netserk B-44, November 20, 1976.
Figure 3. Height profiles along lines A-A' and B-B' shown in Figure 1 on November 11, 1975 (dashed) and February 4, 1976 (solid).
Figure 4. Cross-sectional profiles of idealized floating and grounded ridges.

Figure 5. Air temperatures at which it would be possible for rubble of porosity $\gamma$ to completely consolidate.
ABSTRACT

Engineers involved in the ocean industry come from the traditional disciplines of engineering such as civil, mechanical, electrical and chemical or from naval architecture and engineering programs. They are involved in the solution of engineering problems in the ocean environment such as offshore structures, deep submersibles, semi-submersibles, corrosion problems, sonar communication, marine instrumentation, coastal processes and the development of energy sources from the ocean.

How should ocean engineering be approached in the university? Should it be a graduate program aimed at engineers from the traditional disciplines or should it be approached as an undergraduate program either in a separate department or within a traditional department? In either case, it must meet the requirements of the ocean industry.

This paper reviews ocean engineering programs within the United States and their ability to satisfy the needs of the ocean industry. It presents the basic curriculum needed in both an undergraduate and graduate ocean engineering program. The relative advantages to the student and to the future employer of graduate versus undergraduate education are compared.

INTRODUCTION

Many academic institutions have active programs in ocean engineering at the graduate level, while other institutions offer special undergraduate options or minors in ocean engineering. Several colleges, seven to be exact, offer undergraduate degrees in ocean engineering.* However, there still seems to be a question as to how the education of ocean engineers should be approached in the academic community. Three alternatives which should be considered are graduate education in ocean engineering, undergraduate education in ocean engineering or undergraduate education in a traditional discipline of engineering with a few electives in ocean engineering.

In any case, the education needed by ocean engineers must meet the needs of industry

*This excludes the universities offering undergraduate degrees in Naval Architecture and Marine Engineering, Oceanography and either two or four year programs in either Ocean or Marine Technology.
and educate the student in areas of concern to ocean engineers. Defining the term ocean engineering is a good start in resolving this problem. Keil (1973) states: "Engineering in the ocean environment is the integrated application of the spectrum of engineering disciplines related to operations in the ocean environment." A less formal definition, but quoted most often, is given by Vine (1968) "A good ocean engineer is one who is constructively interested in problem solving at sea." In either definition we are dealing with engineering in the hostile ocean environment with economic constraints. The activities of concern to engineers working in the ocean environment are quite broad in scope. Table I by the Marine Board (1975) is a suggestive list of these activities with limited attention given to ship architecture and construction. These activities often overlap the traditional disciplines of engineering and involve interdisciplinary solutions.

How can education best be presented to educate engineers to meet the intent of the definitions stated earlier yet provide the basic skills necessary to solve technical problems in the areas of concern shown in Table I? The clear majority of ocean engineering programs are offered at the graduate level with at least thirty universities offering some type of ocean engineering program either organized within a traditional department or as an interdisciplinary program in a college of engineering. The students who apply for these programs usually come from the traditional engineering disciplines such as civil, mechanical, electrical, petroleum and chemical or from naval architecture and marine engineering. In most cases, students with backgrounds in the sciences are also well qualified to enter graduate ocean engineering programs.

Most engineers do not agree on the definition of ocean engineering and, as expected, most graduate programs do not agree on the specific content of ocean engineering programs. The variability of curriculum at different universities reflects this situation since, by definition, ocean engineering covers a broad spectrum of subjects. Core courses directed towards an understanding of the environment are usually required or strongly recommended. Herbich (1974) states that the core program should be divided into five basic technical areas, with the engineer gaining a knowledge of:

a. Ocean Environment and its Measurable Parameters
b. Behavior of Materials within the Ocean
c. Interaction of Humans within the Ocean Environment
d. Transfer and Communication Characteristic of the Ocean
e. Operational Instrumentation and other Hardware

In addition to core courses covering these areas, most universities require that the student spend a minimum of one week to 10 days at sea involved with some research project to obtain a first hand familiarity with the oceanic environment. It might be best to have each ocean engineering student place and retrieve an instrument package in the ocean as a degree requirement, thus allowing the student to appreciate Vine's definition of an ocean engineer. Upon completion of the core courses, specialization in selected areas of ocean engineering is usually encouraged, along with a thesis or special problem. The research program at the university most often controls the direction of the selected areas. Quite often the student who has entered ocean engineering from one of the traditional disciplines will conduct his or her research in a field closely aligned to the traditional area, and the student does not become an ocean engineer in the broad sense of the definition.

What major differences occur if the program of study is at the undergraduate level instead of at the graduate level? The undergraduate curriculum must provide the
basic education to our young engineers so that they are equipped to help advance
technology for industry. Yachnis (1968) has recommended that the following areas
of study be included in a typical undergraduate program.

a. Mathematical Methods in Applied Science
   1. Methods and Theory of Functions of Complex Variables
   2. Differential Equations
   3. Linear Boundary Value Problems in Partial Differential Equations
   4. Introduction to Tensor Analysis
   5. Introduction to Non-Linear Differential Equations

b. Ocean Environment
   1. Hydrodynamics
   2. Water Waves
   3. Principles of Physical and Biological Oceanography
   4. Materials Science in Hydrospace
   5. Diver Technology

c. Design of Systems in Hydrospace
   1. Floating and Fixed Platforms
   2. Submersibles
   3. Seafloor Engineering
   4. Deep Ocean Simulation Facilities Design

While this breadth of education, especially the mathematics, would be ideal, it is
probably beyond the reach of most undergraduate engineering programs. Education in
some of the advanced areas, such as design of systems in hydrospace, would not allow
the basic engineering requirements needed by ocean engineers to be given. It is
clear that a better balance of the basic engineering skills is necessary. The re­
quirements of the Engineering Council for Professional Development in mathematics,
basic sciences, engineering sciences, design, humanities, social sciences and other
technical electives will provide this balance.

The typical curriculum in ocean engineering has been examined for comparison at four
universities offering a Bachelor of Science degree in Ocean Engineering. The four
universities chosen are listed below, and it can be seen that each has a slightly
different organizational department.

<table>
<thead>
<tr>
<th>University</th>
<th>Date Initiated</th>
<th>Department</th>
</tr>
</thead>
<tbody>
<tr>
<td>Florida Atlantic University*</td>
<td>1965</td>
<td>Ocean Engineering</td>
</tr>
<tr>
<td>Florida Institute of Technology</td>
<td>1971</td>
<td>Oceanography and Ocean Engineering</td>
</tr>
<tr>
<td>Texas A &amp; M University</td>
<td>1972</td>
<td>Civil Engineering</td>
</tr>
<tr>
<td>Virginia Polytechnical Institute</td>
<td>1975</td>
<td>Aerospace and Ocean Engineering</td>
</tr>
</tbody>
</table>

Each of these universities require the undergraduate ocean engineer to complete
these fundamental curricula.

*An upper division university offering only the last two years of study and credit­
ed for initiating the first undergraduate program in ocean engineering.
Subject | Range of Quarter Hours
--- | ---
Calculus and Elementary Differential Equations | 22-29
General Chemistry and Physics | 18-24
English Composition and Literature | 9-12
Humanities and Social Sciences | 21-24
Engineering Mechanics (Fluid and Solid) | 20-27
Thermodynamics and Heat Transfer | 3-10
Engineering Materials | 3-7
Oceanography and Related Sciences | 8-18
Engineering Drawing and Computer Science | 9-16
Electric Circuits and Instrumentation | 5-11
Advanced Engineering Courses in Ocean, Civil, Aerospace (includes projects or independent study) | 24-51

Total required hours for Bachelor of Science Degree 193-199

The ocean engineering programs offered at each of these universities place their major emphasis on the basics of engineering. Studies in the ocean sciences are kept to a minimum, and the second emphasis in these programs is in terms of advanced engineering courses. It is at this level of the secondary emphasis that the curricula differs, with the subject area favored being that of the organizing department. For instance, Florida Atlantic University, Florida Institute of Technology, and Texas A & M University offer advanced civil engineering or specialized courses created for ocean engineers. Virginia Polytechnic Institute's emphasis is on aerospace engineering courses. One can conclude that these programs provide undergraduate engineers with the basic tools needed for an industrial or government career and also with the ability to pursue further education in either ocean engineering or in the traditional disciplines.

There is an additional approach which can be used to educate ocean engineers at the undergraduate level. Many of the engineering schools offer minors, options, or a required undergraduate course in ocean engineering. This will give the student the opportunity to be exposed to ocean engineering. The basics required in this type of survey course have been cited by Richards (1975) and consist of:

a. A basic practical scientific background needed by any engineer working in the ocean.
b. A survey of engineering technologies specific to the oceans.
c. An emphasis on the coastal zone and Continental Shelf.
d. A discussion of evolution of ocean industries.

This type of survey course has been developed by the author in the Civil Engineering Department at the University of Houston. Since many of the undergraduate civil engineers were being employed by Houston based ocean industries, it was felt that the student should be prepared to work on civil engineering problems in the ocean environment. The course developed at the University of Houston is a lecture laboratory course divided into five technical areas.

a. Ocean environment emphasizing submarine topography, navigation, physical properties of seawater, marine sediments and materials selection.
b. Wave Mechanics dealing with linear wave theory, wave forces on structures, and wave prediction.
c. Naval Architecture dealing with ship structures, stability and vessel motion.
d. Coastal processes, emphasizing beach processes and coastal protection.
e. Ocean Instrumentation, dealing with proper selection and design of instruments.

What are the advantages to industry and to the student from an undergraduate ocean engineering program versus a graduate program? The student may find that either program can be suited to his or her individual needs. Graduate study traditionally offers the greater flexibility to the student and allows for specialization. Undergraduate study allows the student to enter his chosen career field at an earlier time. The ocean industry often prefers the undergraduate ocean engineer because he or she is educated in applying engineering basics to the economic solution of problems in the ocean environment. If additional expertise is needed or desired in one technical area of ocean engineering, graduate study can be pursued at a later date. Students with undergraduate degrees in ocean engineering have been employed in the ocean industry or government or have gone on to graduate school in one of the traditional disciplines or in ocean engineering. Of the graduates from the ocean engineering program at the Florida Institute of Technology approximately 20% have gone on to graduate school, 30% have gone on to industry, 25% to government and 25% to whereabouts unknown.

Ocean engineering is a well recognized undergraduate curriculum. Herbich (1974) felt that ocean engineering programs are in a developmental state, and that the ocean engineering programs will expand at both the graduate and undergraduate level in response to the needs of industry, government and academia. Ocean engineering programs at the undergraduate level have matured and are well established at seven universities offering study in this area. Graduate programs are continuing to develop, quite often guided by the research needed at the universities. For instance, the graduate program in ocean engineering at the University of Massachusetts has become quite energy oriented with a focus on ocean thermal electric gradient and offshore winds. The ocean engineering programs at both the undergraduate and graduate level will continue to mature in order to produce the engineer needed by industry, government and the academic community who is interested in making things work at sea.
REFERENCES


### TABLE I

SELECTED OCEAN PHENOMENA AND ACTIVITIES OF CONCERN TO OCEAN ENGINEERS

<table>
<thead>
<tr>
<th>Minerals-Mining</th>
<th>Platforms</th>
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<tbody>
<tr>
<td>Sand and Gravel</td>
<td>Piers and Quays</td>
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<tr>
<td>Exploration Systems</td>
<td>Towers and Related Bottom-Sitting Platforms</td>
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<tr>
<td>Minerals</td>
<td>Artificial Islands</td>
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<td>Equipment</td>
<td>Tunnels</td>
</tr>
<tr>
<td>Environmental Impact Control</td>
<td>Buoys</td>
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<tr>
<td>Product Systems</td>
<td>Vessels</td>
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<td>Habitats</td>
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<td>Floating Cities or Seadromes</td>
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<td>Satellites</td>
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<td>Aircraft</td>
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<td>Diver Equipment</td>
<td>Balloons</td>
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<td>Diver Physiology</td>
<td>Submersibles</td>
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<td>Habitats</td>
<td>Waste Disposal</td>
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<td>Recreation</td>
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<td>Construction</td>
<td>Operations</td>
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<td>Pipelines</td>
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<td>Enhancement</td>
<td>Storage</td>
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<tr>
<td>(Groins, Sand Replenishment, etc.)</td>
<td>Platforms</td>
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<tr>
<td>Shore Protection</td>
<td>Subsea Completions</td>
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<tr>
<td>Outfall and Intake Siting</td>
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<td>Dredging Activities</td>
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<td>Environmental Impact Control</td>
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<td>Processing</td>
<td></td>
</tr>
<tr>
<td>Detection</td>
<td></td>
</tr>
<tr>
<td>Environmental Impact Stress</td>
<td></td>
</tr>
</tbody>
</table>
Ports, Harbors and Channels

Marinas
Navigation
Traffic Management
Dredging
Breakwaters
Pollution Control
Environmental Impact
Cargo Handling Systems

Environmental Engineering Data

Waves
Wastes
Currents
Ice
Tides
Bottom Topography
Chemistry
Temperature
Weather Modification
Maritime Weather

Environmental Sciences

Monitoring
Instruments and Sensors
Data Collection Systems
Laws and Regulations (affecting engineering standards)

Marine Recreation

Safety
Sport Diving
Boats and Service
Tourism
Swimming/Surfing
Sport Fishing

Surveying

Navigation
Sonar Applications
Photo and Television
Geodesy
Magnetics
Instrumentation

Communication, Telemetry and Power Transmission

Installation
Telephone and Telegraph Systems
Power Cables
Other Utilities
Sonar Systems
Frequency Allocation
Maintenance
Sea Floor Engineering

Sediment Mechanics
Geology and Geophysics
Equipment Design and Performance
Environmental Impact and Interaction
Topography

Naval Architecture

Ship Design and Analysis
Production Methods
Materials and Fabrication
Testing and Evaluation
This paper illustrates the use of a computerized simulation model which has been developed to calculate wave induced ship motions and mooring forces for vessels moored in exposed locations. Ship mooring facilities at one existing and one proposed oil terminal installation on the eastern seaboard of Canada have been analyzed with this computer model. The results of these studies have been compared with those of hydraulic scale model investigations previously completed by other agencies.

The computer program employed in these simulation analyses has been designed to assist in optimizing the design layout of offshore terminal mooring systems that are exposed to significant wave action. Therefore, the program incorporates the capability of analyzing vessel motions and mooring forces in a time domain solution for both subharmonic response to regular wave forces and slow drift response to non-linear wave group forces that are characteristic of irregular waves.

The results of the comparative mathematical and hydraulic model studies presented in this paper include a summary of simulation tests for alternative mooring arrangements, mooring line types, line pretension loads and varied sea state conditions. The maximum vessel motions and mooring forces resulting from regular waves and from slow drift oscillations generated by wave group formation in irregular sea states are evaluated and discussed.

INTRODUCTION

The advent of recent generations of oil tankers in the VLCC and ULCC class has necessitated the proposed location, design and construction of loading and off-loading terminals at exposed sites not previously considered because of adverse environmental conditions. Consequently, the design of these terminals and the assessment of their capability to provide safe mooring conditions have become increasingly difficult.

Computer simulation of ship motions and mooring forces can provide valuable information for use in the design and assessment of offshore terminals. Mathematical computer models provide a quick and inexpensive method of evaluating many vessels, dock orientations, mooring arrangements, sea state conditions and a
variety of mooring lines and fender types. Once mathematical models have been thoroughly verified, they can be employed to augment and reduce the requirement for expensive hydraulic modelling studies of potential offshore terminal facilities.

The purpose of this paper is to summarize recent computerized ship mooring analyses carried out by Public Works Canada for the existing tanker terminal at Come-By-Chance, Newfoundland, and for a potential tanker terminal at Tiner Point in New Brunswick. The results of these computer simulations are compared with data from hydraulic laboratory modelling studies of the same facilities.

The computer program entitled MOSA (Mooring Systems Analysis) which was used in these analyses (Seidl, 1976), provides the flexibility to evaluate vessel motions and restraining forces resulting from regular and irregular waves. Vessel motions and mooring reactions caused by slow drift oscillations that result from second order wave forces, and subharmonic oscillations initiated by first order wave forces can be modelled over a period of time. The alternative analyses that can be carried out are as follows:

1. Maximum vessel response in six degrees of freedom for regular waves of any height and period and a linearized mooring system;

2. A frequency domain solution for maximum statistical mooring line forces and vessel response in six degrees of freedom for irregular waves and a linearized mooring system;

3. A time domain solution of mooring forces and vessel response in six degrees of freedom for wave group drift forces and a non-linear mooring system;

4. A time domain solution of mooring line forces and vessel response in six degrees of freedom for regular or irregular waves and a non-linear mooring system. This solution employs a "convolution" technique to define frequency-dependent hydrodynamic forces.

METHOD OF APPROACH

A retrospective hydraulic modelling study of the Come-By-Chance oil terminal wharf in Newfoundland was carried out in 1975-76 by the Canadian Coast Guard, Transport Canada, as part of a recently concluded Canada-Japan joint offshore structures research program (Transport Canada, 1975; 1976). At about the same time, laboratory analyses of berthed vessels at a proposed tanker terminal near Tiner Point, New Brunswick, were undertaken by the Danish Hydraulic Institute (DHI) for Eastern Designers - Swan Wooster, consultants to the Provincial Government of New Brunswick (Danish Hydraulic Institute, 1976). In the Come-By-Chance study, selected supertankers were modelled for regular wave conditions with varied wave heights and periods as well as different mooring arrangements, vessel loadings, mooring line characteristics and pretensions. The DHI study of the Tiner Point location was similar in nature, with the main difference being that irregular sea action and severe tidal currents were evaluated in detail.

In 1977, Public Works Canada carried out comparative computer simulation studies of selected analyses drawn from these laboratory studies in order to assess the degree of correlation of the computer and hydraulic model results.
Come-By-Chance Laboratory Model

The hydraulic model tests carried out by the Canadian Coast Guard for the Come-By-Chance oil terminal wharf (Transport Canada, 1976) considered a 227,000 DWT tanker model positioned in a wave basin such that regular waves would impact on the moored vessel from a direction of 30 degrees off the starboard bow.

In the tests carried out in the laboratory and repeated with the computer model, wave heights of 1.8, 3.0 and 4.3 metres, with periods ranging between 5 and 15 seconds, were evaluated. The simulated mooring lines were three non-linear nylon hawsers for the head, breast and stern lines (Figure 1). Single steel spring lines were tested both with and without 10 metre nylon tails. Separate analyses were carried out for mooring line pretensions of 5, 7.5 and 10 tonnes. Current loads were not considered in these tests, and although the effect of wind was evaluated in the hydraulic model studies, analyses without wind were selected for the computer and hydraulic model comparisons.

Tiner Point Laboratory Model

The hydraulic model tests carried out by the Danish Hydraulic Institute for the once proposed oil tanker terminal at Tiner Point (Danish Hydraulic Institute, 1976), considered vessels of 100,000 and 26,900 DWT with head, quartering and beam seas.

Irregular seas with 1.2 metre significant wave height and spectral peak periods between 10 and 12 seconds, as well as less than 10 seconds, were evaluated. The simulated mooring arrangement (Figure 2) included non-linear nylon lines, all wire lines and wire lines with nylon tails. Analyses were undertaken for pretension loads of 13.5 tonnes per line for the 100,000 DWT vessel and 6.8 tonnes per line for the smaller 26,900 DWT ship. Wind and current loads were considered in these analyses.
Mathematical Model

The theory upon which the MOSA computer simulation model is based is governed by a set of six simultaneous second order differential equations for a moored vessel, which in simplified terms can be represented as:

\[ [m] \{\ddot{x}\} = -\{F_h\} - \{F_b\} - \{F_m\} - \{F_f\} + \{F_w\} + \{F_a\} + \{F_c\} \quad \text{................... (1)} \]

where

- \([m]\) = matrix of mass and inertia of the vessel (in air).
- \(\{\ddot{x}\}\) = vector of vessel displacement in six modes of motion, i.e. surge, sway, heave, roll, pitch, yaw.
- \(\{F_h\}\) = hydrodynamic forces and moments which are generated when the vessel is oscillating in calm water.
- \(\{F_b\}\) = hydrostatic or buoyancy forces and moments.
- \(\{F_m\}\) = elastic restoration due to the mooring system.
- \(\{F_f\}\) = fender friction forces and moments.
- \(\{F_w\}\) = wave excitation forces and moments.
- \(\{F_a\}\) = wind forces and moments.
- \(\{F_c\}\) = current forces and moments.

The hydrostatic forces, mooring restoration functions and fender friction forces are functions of vessel displacement in all six modes of motion.

The hydrodynamic force is a function of accelerations and velocities in the six degrees of freedom and is dependent on the frequency of vessel oscillation. In a linear system with steady state oscillation, this frequency would be equal to that of the wave excitation. In a non-linear system, however, the vessel oscillation may contain subharmonic frequencies differing from that of the excitation force. In irregular seas, a continuous band of frequencies may be present in the excitation force. Thus, a basic difficulty arises as to which frequency to use in defining the frequency dependent hydrodynamic coefficients.

A "convolution integral" solution to this problem was originally given (Cummins, 1962) and has since been applied in fields other than ship mooring studies. In moored ship problems this approach was demonstrated to yield useful results (Van Oortmerssen, 1976). A derivation of this technique is beyond the scope of this paper, but some of the expressions are given below:

The hydrodynamic force in the \(i\)-th mode, due to motion in the \(j\)-th mode, is given as:

\[-F_{h,ij} = \ddot{x}_j M_{ij} + \int_{-\infty}^{t} K_{ij}(t-\tau) \ddot{x}_j \, d\tau \quad \text{......................... (2)}\]

where

\[K_{ij}(\tau) = \frac{2}{\pi} \int_{0}^{\infty} b_{ij}(\sigma) \cos \sigma \tau \, d\sigma; \quad M_{ij} = \text{high frequency asymptotic value of the added mass}\]
= c 1-J
.. x.
J

Substituting and setting Fb
then be written as:
'" [ a .. x.
.• 1
L,

1-J

J

+

as a linear hydrostatic force, equation (1) can

t

c .. x. +
1-J J

f

_00

K .. (t - ,)

1-J

x.J (,) d ,J

+ F

. (xl' x •.. x6 )
2

moor,1-

F .(t) + F . f
.+F. d . t F
.
W,1current,1d rl t,l wln ,1

. . . . . . . . . . . . . . . . . . . . . . . .•

(3)

where
a ..

lJ

= m.o
..
1 lJ

+- M ..
lJ

This set of equations is then solved by a time stepping procedure, with line
tensions and fender forces evaluated at every time step.
In its most general form, the MOSA program uses frequency-independent hydrodynamic
forces, and solves the above set of six equations in a time stepping procedure,
considering non-linearities in the mooring system and all the component waves that
make up a sea state. First order wave forces are calculated by the well known
strip theory, as are the damping coefficients (b .. ) for which the K .. (,) functions
have been evaluated.
lJ
lJ
For slightly non-linear syste~s, as is the case with many mooring systems, a much
simpler and more economical approach consisting of linearizing the mooring restoration function is feasible. Then, the response to regular waves can be readily
evaluated in the frequency domain, whereby hydrodynamic coefficients are constant
for anyone wave frequency. The equations of motion then reduce to:

raJ

{~} +

[bJ

{i} + [c] {x}

= {Fw}

(4)

and the solution is given in terms of motion amplitudes and phase lags.
A third and final point warrants special attention, namely that of second order
wave drift forces and their effect on ship motions as discussed elsewhere (Seidl and
Lee, 1975; Van Oortmerssen, 1976). The theory exists to model slow drift vessel
oscillation, and advanced procedures are being developed to improve upon computer
simulation of this phenomena which must be considered due to its overriding
importance in many mooring conditions. Slow drift oscillations played only a minor
role in the two comparison cases presented in this paper and, therefore, will not
be discussed further at this time.
RESULTS OF COMPARATIVE TESTS
Come-By-Chance Laboratory and Computer Simulation Analyses
Of the range of conditions evaluated in the hydraulic modelling study of the ComeBy-Chance facility, eight specific test cases were selected for comparison by
computer simulation as summarized in Table 3. Separate computer analyses of these
eight cases were carried out for regular waves with heights of 1.8, 3.0 and 4.3
metres and periods of 5, 7-1/2, 10, 11-1/4, 12-1/2, 13-3/4 and 15 seconds. A
summary of these comparative tests, depicting maximum observed line tensions for
the hydraulic model and the computer simulations, are presented in Tables 4 and 5
respectively. The data for the 1.8 and 4.3 metre waves with periods of 5, 11-1/4,

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12-1/2 and 13-3/4 seconds and the 3.0 metre waves followed the same trends as can be observed in these tables and are not reproduced herein. Since fender compression forces and vessel motions were not recorded during the hydraulic modelling studies, these data have not been presented, although they are available from the computer analyses.

Table 3: Vessel Load, Mooring Line Pretensions and Mooring Line Characteristics for the Come-By-Chance Computer Simulation

<table>
<thead>
<tr>
<th>Vessel Load</th>
<th>Mooring Line Pretension</th>
<th>5 Tonnes</th>
<th>7.5 Tonnes</th>
<th>10 Tonnes</th>
<th>Type of Mooring Line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Load</td>
<td>Case 1A</td>
<td></td>
<td>Case 2A</td>
<td>Case 3A</td>
<td>Non-linear nylon head, breast and stern lines.</td>
</tr>
<tr>
<td>Ballast</td>
<td>Case 1B</td>
<td></td>
<td>Case 2B</td>
<td>Case 3B</td>
<td>Non-linear steel spring lines with 10 m nylon tails</td>
</tr>
<tr>
<td>Full Load</td>
<td>---</td>
<td>---</td>
<td>Case 4A</td>
<td></td>
<td>Non-linear nylon head, breast and stern lines.</td>
</tr>
<tr>
<td>Ballast</td>
<td>---</td>
<td>---</td>
<td>Case 4B</td>
<td></td>
<td>Steel spring lines (1% elongation at breaking strength), no nylon tails.</td>
</tr>
</tbody>
</table>

It is apparent when comparing the data of Tables 4 and 5 that the mooring line tensions derived by computer simulation and hydraulic modelling follow the same trends with varying conditions of pretension load, vessel loading and mooring line type. Maximum observed tensions in the head and stern lines are comparable for all wave periods, wave heights and vessel loading conditions, with slightly lower values being observed for the computer simulations. A similar agreement exists for the breast lines, with the exception of the data for 15 second, 4.3 metre waves and the vessel in ballast condition. Here the line tensions observed in the hydraulic model are significantly greater than the line forces derived by computer simulation. In the case of the spring lines, the ballast load condition resulted in much larger maximum line tensions occurring in the hydraulic model for 1.8 metre regular waves with periods greater than 7.5 seconds and 4.3 metre waves with periods above 5 seconds. Under full load conditions with 4.3 metre waves longer than 5 seconds, higher maximum spring line loads were also observed in the hydraulic model.

Examination of the time domain records from the computer simulations in these cases of disparate comparison identified several noteworthy points.

1. Under ballast load, regular waves excited long period subharmonic vessel oscillations in surge and sway which increased in amplitude with increased wave height and period. These long period vessel oscillations were in turn translated into wide variations in spring line tensions, and in the case of the 13 to 15 second regular waves, into long period variations in
breast line tensions. As a result, intermittent but very high line tensions were observed.

2. For the full load conditions, subharmonic oscillations did exist, but they were not as extreme as observed for the ballast load conditions.

3. For both load conditions, a real time period longer than the simulated time was required for the vessel to reach a position of stable dynamic equilibrium under the imposed steady state wave drift force. This condition was particularly noticeable for the larger 4.3 metre wave.

Table 4: Maximum Observed Tensions per Line in Metric Tonnes for the Come-By-Chance Hydraulic Model

<table>
<thead>
<tr>
<th>Mooring Line Pretension</th>
<th>Wave Period in Seconds</th>
<th>Head/Stern</th>
<th>Breast</th>
<th>Spring</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1.8 m Wave</td>
<td>4.3 m Wave</td>
<td>1.8 m Wave</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
</tr>
<tr>
<td>5 nt</td>
<td>7.5</td>
<td>7</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td>7.5 nt</td>
<td></td>
<td>9</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>10 nt</td>
<td></td>
<td>12</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>10 s</td>
<td></td>
<td>11</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>5 nt</td>
<td>10</td>
<td>7</td>
<td>9</td>
<td>8</td>
</tr>
<tr>
<td>7.5 nt</td>
<td></td>
<td>9</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>10 nt</td>
<td></td>
<td>11</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>10 s</td>
<td></td>
<td>11</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>5 nt</td>
<td>15</td>
<td>7</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>7.5 nt</td>
<td></td>
<td>9</td>
<td>12</td>
<td>9</td>
</tr>
<tr>
<td>10 nt</td>
<td></td>
<td>12</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td>10 s</td>
<td></td>
<td>12</td>
<td>16</td>
<td>12</td>
</tr>
</tbody>
</table>

nt - denotes 10 metre nylon tails on the single spring lines.
s - denotes steel spring lines (1% elongation at breaking strength) and no nylon tails.

NOTE: Model simulation for 50 to 100 minutes real time.
Table 5: Maximum Observed Tensions per Line in Metric Tonnes for Computer Simulation of the Come-By-Chance Facility

<table>
<thead>
<tr>
<th>Mooring Line Pretension</th>
<th>Wave Period in Seconds</th>
<th>Head/Stern</th>
<th>Breast</th>
<th>Spring</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.8 m Wave</td>
<td>4.3 m Wave</td>
<td>1.8 m Wave</td>
<td>4.3 m Wave</td>
</tr>
<tr>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
</tr>
<tr>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
</tr>
<tr>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
<td>Ballast</td>
<td>Full Load</td>
</tr>
</tbody>
</table>

- nt - denotes 10 metre nylon tails on the single spring lines.
- s - denotes steel spring lines (1% elongation at breaking strength), no nylon tails.

NOTE: Model simulation for 8 minutes real time.

As a result of the above observations, selected computer analyses that incorporated consideration of the steady state wave drift forces were repeated for real time simulation sequences in the range of 13 to 33 minutes. During these analyses, it was apparent that imposition of the wave drift forces caused the vessel to quickly reach a position of dynamic stability and that continued large amplitude sub-harmonic oscillations, particularly for the ballasted vessel, resulted in the occurrence of intermittent and large spring line tensions. Data selected from these simulations for the 227,000 DWT vessel in ballast condition are presented in Table 6 along with information excerpted from Tables 4 and 5.
Table 6: Maximum Observed Line Tensions per Line in Tonnes for the Model Vessel in Ballast Condition, Nylon Tails on the Spring Lines, and all Lines Pretensioned at 10 Tonnes

<table>
<thead>
<tr>
<th>Simulation Analyses</th>
<th>Tp (sec)</th>
<th>1.8 m Wave</th>
<th>4.3 m Wave</th>
<th>1.8 m Wave</th>
<th>4.3 m Wave</th>
<th>1.8 m Wave</th>
<th>4.3 m Wave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic Model (50 to 100 minutes)</td>
<td>7.5</td>
<td>11</td>
<td>13</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>13</td>
<td>16</td>
<td>15</td>
<td>17</td>
<td>22</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>15</td>
<td>20</td>
<td>23</td>
<td>36</td>
<td>50</td>
<td>&gt;70</td>
</tr>
<tr>
<td>Computer Model (8 minutes)</td>
<td>7.5</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
<td>11</td>
<td>11</td>
<td>12</td>
<td>14</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>14</td>
<td>18</td>
<td>17</td>
<td>26</td>
<td>31</td>
<td>61</td>
</tr>
<tr>
<td>Computer Model (longer simulation time and consideration of drift forces)</td>
<td>7.5</td>
<td>--</td>
<td>13</td>
<td>--</td>
<td>15</td>
<td>--</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>11</td>
<td>15</td>
<td>12</td>
<td>18</td>
<td>19</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>14</td>
<td>20</td>
<td>17</td>
<td>28</td>
<td>35</td>
<td>&gt;70</td>
</tr>
</tbody>
</table>

From these analyses it is apparent that the computer model must be run for a real time simulation period sufficiently long enough to allow subharmonic oscillations to become fully established, and secondly that steady state wave drift forces must be incorporated in the analyses to accurately model vessel motions and mooring forces. This is particularly important in the case of mooring restraint systems that are highly non-linear, such as those employed in the Come-By-Chance model tests described herein.

Tiner Point Laboratory and Computer Simulation Analyses

Out of the range of conditions evaluated in the hydraulic modelling study of the potential Tiner Point offshore terminal (Danish Hydraulic Institute, 1976), four sea states and one mooring line type were chosen for comparative computer analyses of a 100,000 DWT tanker in ballast condition. These sea states were quartering and beam seas of 1.2 metre significant wave height, both of which were evaluated for peak spectral periods of 7 and 11.5 seconds. Of the three mooring line types evaluated in the DHI hydraulic model studies, 140 mm circumference wire lines with 4.5 m nylon tails were modelled in the comparative computer simulation analyses. The two fender units modelled in both the hydraulic and computer simulations consisted of a pneumatic 4.5 φ x 9.0 L Yokohama backed by a 2 x 4 Seibu 800 H. Figure 3 depicts the restoration functions of this mooring system due to vessel displacements in surge, sway and yaw, respectively and indicates that the mooring system is quite linear for moderate displacements. This linearity of the mooring system is due to the use of very soft fenders and stiff mooring lines. The observation of the quasi-linearity of the mooring system suggested that vessel response and mooring reactions be calculated initially by the option of the MOSA program which solves the linearized equations of motion in the frequency domain.
Thus the response to regular waves, that is the complex response operators in all six degrees of freedom are calculated first. The ship motion spectra in all six modes of motion are then obtained by:

\[ S_{x_1x_1}(\sigma, \theta) = S_{\eta \eta}(\sigma, \theta) | H_{x_1}(j\sigma, \theta) |^2 \]  

The significant double amplitudes of ship motion in a sea state with principle wave heading \( \chi \) are obtained as:

\[ [X_1(\chi)]_{\text{sig}} = C |_{\sigma_{\text{min}}}^{\sigma_{\text{max}}} \int_{-\pi/2}^{\pi/2} S_{\eta \eta}(\sigma, \theta) | H_{x_1}(j\sigma, \chi + \theta) |^2 d\theta d\sigma \]

In the case of a one-dimensional wave spectrum, this expression reads:

\[ [X_1(\chi)]_{\text{sig}} = C |_{\sigma_{\text{min}}}^{\sigma_{\text{max}}} \int_{-\pi/2}^{\pi/2} S_{\eta \eta}(\sigma) | H_{x_1}(j\sigma) |^2 d\sigma \]

Since one-dimensional wave spectra were employed in the hydraulic model studies of the Tiner Point terminal, one-dimensional spectra were reproduced and employed in the comparative computer simulation study. It should be noted that to date, most hydraulic laboratories can only produce one-dimensional spectra in their irregular wave basins. In actual fact, natural sea states are more accurately represented by two-dimensional wave spectra. Since considerable differences in vessel motions and mooring reactions may result from two-dimensional as compared to one-dimensional spectra, two-dimensional spectra conditions were also examined in the computer analyses. Figure 4 depicts a sample of one and two-dimensional ship motion spectra in surge corresponding to one and two-dimensional wave spectra of the same significant height and peak period.
The results of the comparative computer studies shown in Figure 6 represent data derived from simulations of beam and quartering seas both with peak spectral periods of 7 and 11.5 seconds. In each case data on maximum mooring reactions are presented for simulation analyses of one-dimensional wave spectra with a frequency domain solution, two-dimensional spectra with a frequency domain solution, and one-dimensional spectra using a time domain solution which incorporates the convolution technique for calculating hydrodynamic forces.

In regard to the data for quartering seas, the agreement with the results of the hydraulic modelling study is quite good for maximum line tensions and a peak spectral period of 7.0 seconds. The results of the time domain analyses are generally lower than those of the other analyses due to the fact that the maximum values as plotted were obtained from time domain simulation runs in the order of eight minutes real time, whereas the frequency domain maxima were identified after 10,000 vessel oscillations. For the peak spectral period of 11.5 seconds, the frequency domain solutions resulted in the identification of maximum mooring line tensions that compare well with data defined by the hydraulic model, while maximum fender forces are overestimated. Here, due to the larger vessel motions, the non-linearity and asymmetry of the mooring system become important.
Figure 5: Maximum Observed Mooring Forces for the Tiner Point Hydraulic Model

Figure 6: Maximum Observed Mooring Forces for the Tiner Point Computer Simulation Model
In the case of beam seas, the mooring forces derived by computer simulation for the peak spectral period of 7.0 seconds are satisfactory, although the maximum fender forces and tensions in the breasting lines resulting from the one-dimensional spectrum analyses are unduly high since in a directional sea state all of the wave energy does not travel in the most severe abeam direction. For the peak spectral period of 11.5 seconds, both the frequency and time domain solutions indicate that large vessel motions and mooring forces will occur. Hence, asymmetry and non-linearity in the solution become very important, particularly in respect to fender forces, and the time domain solution and results thereby derived should be used for comparative purposes.

CONCLUSIONS

It can be concluded that mathematical modelling of the motions and mooring reactions of a moored vessel can provide results that are comparable with those of hydraulic modelling studies.

In the case of the Come-By-Chance wharf comparison, the observation that higher maximum line tensions occur for the selected vessel in ballast condition as compared to that of full load is verified by the hydraulic and computer models. The reason for this is the occurrence of large subharmonic vessel oscillations in the ballast case, a condition which does not occur for the fully loaded vessel. A word of caution is due here to both the computer and hydraulic modelling results of the Come-By-Chance facility. Although the terminal is not in a highly exposed location and is, therefore, not subjected to irregular waves of severe and highly variable nature, the fact remains that regular sinusoidal waves as modelled do not occur in nature. Hence the evaluation of vessel response to regular waves is only important in that one can then calculate statistically from these data, ship motion and mooring force spectra for irregular seas.

In the case of the Tiner Point terminal comparisons, it has been shown that the linearized frequency domain approach to estimating maximum mooring forces may lead to realistic predictions as long as vessel motions remain moderate. The importance of including the directionality of the wave spectrum must be stressed as it appears that consideration of a multi-directional spectrum may be far more important than the precise reproduction of the particular shape of a specific one-dimensional spectrum. The main advantage of the frequency-domain approach in the mathematical simulation of vessel motions and mooring forces is the simplicity of its application and minimal computer cost. This approach should, therefore, be used to get a first overall evaluation of a mooring system. However, for cases where large vessel displacements occur, time-domain simulations must be used to more accurately predict mooring reactions.

In the Tiner Point situation it is noteworthy that slow drift oscillation did not appear significant, even though wave drift forces proportional to the square of the wave envelope were included in the analyses. Likewise, subharmonic oscillations were completely absent, which was no doubt due to the linearity of the mooring system and the large damping in surge resulting from the high average fender forces.

In summary, it is postulated that computer simulation models can provide dependable predictions of maximum dynamic vessel motions and mooring reaction forces. As a result, they can be effectively used to augment hydraulic modelling as a tool to provide information on the design and utilization of proposed ship
mooring systems. However, computer and hydraulic models of moored vessels must remain as simplifications of real life situations. Therefore, further efforts to refine and verify these models and their underlying theory with data from instrumented prototype mooring systems such as now exists in Canada at the Come-By-Chance terminal should and will be undertaken.

ACKNOWLEDGEMENTS

The laboratory hydraulic modelling study for the Come-By-Chance wharf described in this paper was carried out by Transport Canada, Waterways Development Division. The laboratory study of Tiner Point was undertaken by the Danish Hydraulic Institute for Eastern Designers - Swan Wooster, consultants to the Provincial Government of New Brunswick. The authors wish to thank these agencies for their permission to allow us to quote their information.

REFERENCES


Transport Canada, Canadian Coast Guard, Waterways Development Division, "Come-By-Chance Model Study - Phase 1", Report No. LHL-646, Ottawa, July 1975.


AN EXPERIMENTAL PROGRAM ON MOORING FORCES OF LARGE VESSELS BERTHED AT OFFSHORE TERMINALS

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INTRODUCTION

There is an increasing need by regulatory bodies as well as the shipping industry for accurate means of determining the dynamics of large vessels berthed at terminals subjected to varying sets of physical and environmental conditions. While the ultimate means would ideally be a mathematical model incorporating established empirical and analytical methods to arrive at the answer, this approach remains academic until exhaustive studies are conducted on physical models. Recognizing this need, a comprehensive study program was initiated within the Hydraulics Research Centre of the Canadian Coast Guard. On a scale model duplicating conditions at the offshore terminal at Come-by-Chance, Newfoundland, VLCC models were used to study the dynamics of mooring-line forces generated as a result of combinations of operational and environmental factors. With the view of generalizing the scope of the studies, the program included parameters which would be beyond those expected at the site of Come-by-Chance. The initial program is now completed; however on the basis of findings from the study, some additional testing is expected.

THE SCALE MODEL

The model was built at an undistorted scale of 1/100 and operated according to the Froude law of similitude. Figure 1 gives the site and berth layout. The waves were produced by a wave generator at 30° to the face of the berth. The fenders' characteristics at the Come-by-Chance site were averaged and reproduced on the model by two points of contact.

Two VLCC models were used in the study, one of 227,000 DWT and the other 412,000 DWT. Their hulls were accurately molded of fiber glass. Both vessels were calibrated for two loading conditions, full load and ballast. In each case the centre of gravity, as well as the rolling, pitching, and heaving periods were calibrated to correspond to the figures given below:

<table>
<thead>
<tr>
<th></th>
<th>227,000 DWT</th>
<th>412,000 DWT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overall</td>
<td>326.8</td>
<td>377</td>
</tr>
<tr>
<td>Breadth</td>
<td>48.2</td>
<td>58.8</td>
</tr>
<tr>
<td>Loading</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Draft</td>
<td>10.1</td>
<td>11.8</td>
</tr>
<tr>
<td>Rolling period</td>
<td>10.6</td>
<td>11.4</td>
</tr>
<tr>
<td>Pitching period</td>
<td>10.4</td>
<td>12.1</td>
</tr>
<tr>
<td>Heaving period</td>
<td>11.1</td>
<td>13.0</td>
</tr>
</tbody>
</table>
The forces in the lines were measured by means of sensitive resistance-type transducers in the line-simulation system. Continuous recording of the force variation was obtained in all tests. Two line-simulation systems were used in these tests. In the first instance, the steel as well as the nylon rope-elongation characteristics were simulated by means of a coil spring, hence linear simulation. This method gave an accurate simulation of the steel-line characteristics, and an average approximation of the nylon-lines. In a second method (used for the E and D-series discussed later) the simulation of the nylon lines was changed so that the force/elongation relationship duplicated the parabolic pattern of the prototype ropes. This was achieved by designing a device using a counterweight and a plunger in the form of a triangular prism suspended in a constant level water tank. This method proved to be very practical to calibrate and to change the pretensions as the tests required.

THE TESTING PROGRAM

In a comprehensive testing program covering most practical situations that one can expect to occur in an offshore terminal, the number of tests resulting from the operational and environmental variables could easily multiply to impractical limits. The wave conditions (direction, height, length), mooring arrangements, mooring-line materials, mooring-line pretensions, vessel sizes, and vessel loading, are some of the important factors. To keep the number of tests within practical limits, judgement has to be used in selecting the factors to be analyzed. Also for the purpose of this paper only the most relevant situations will be discussed.

The program covers only one wave direction and a simplified mooring arrangement (Figure 2). In all tests wave heights of 1.83, 3.05 and 4.27 meters, each with 5.0, 7.5, 10.0, 12.5, and 15.0 second periods were used (excluding the 4.27 m/5.0 sec combination where the wave would be too steep). Mooring-line pretensioning of 5.0, 7.5, and 10.0 tons was tested in several series, after which 10.0 ton pretensioning was retained. Wind of approximately 65 km/hr from the same direction as the waves was simulated for some tests. Combinations of steel and nylon-lines were used. The steel-lines were calibrated to provide 1% extension at 68 tons, and the nylon-lines simulated the elongation properties of 68 mm DuPont Type 707 fiber.

Results from two test series are discussed. In the E-series the head, stern, and breasting lines consist each of bundles of three nylon ropes, and the spring lines consist of one steel rope. In the D-series, the lines are the same as in the E-series except that the spring steel-lines have 10-meter nylon tails added to them. In addition to these two series, some results from tests that were conducted with linear line simulation are discussed.

TEST RESULTS

Through the continuous registration of the forces in the six mooring lines a complete record of the variation of line tensions was obtained. For the purpose of this study, only the maximum forces (Fmax) were tabulated and plotted against a number of the other tested parameters. Therefore the forces discussed in this paper refer to the maximum that were measured. Where a line consisted of a bundle of three ropes (e.g. head/stern lines) the pretension specified is the pretension per rope. Figure 2 should be referred to in all discussions on the mooring arrangement and the locations of MD1, 2, 3, 4, and S1 and S2. (Results are for the 227,000 DWT vessel unless otherwise noted)
The analysis of the test results at various stages of the study program has led to numerous useful developments, deductions, and conclusions. It is felt that from the point of the users' benefits, the discussions on the results can be categorized in terms of those relating to practical applications and those relating to modeling techniques.

**Practical Application**

From the point of view of the practical user, although the test results are in most cases clearly demonstrating the point, it has not been possible to formulate generalities in terms of empirical formulae or otherwise. These points are more in the form of operational recommendations.

**Force Distribution in the System** - With the mooring arrangement used in these tests, the reactions in the spring lines proved to be the most sensitive to changes in the system. Figure 3 illustrates a typical situation. The forces in MD1, 2, 3, 4 remain more or less in the range of 30 to 40 tons, whereas the forces in S1 and S2 increase from about 16 tons to about 95 tons as the wave period (Tₚ) is increased from 5 to 15 seconds in the case of a wave height (Hₛ) of 3.05 m. At the lower wave height conditions, when the system is relatively calm, the forces in S1 and S2 are even less than those in MD1, 2, 3, 4 showing that there is only a slight increase from the original 10 ton pretension (Fₒ). These general tendencies are more pronounced when the wave heights are higher or the vessel is in ballast condition.

**Effect of Pretension** - Pretensioning of the mooring-lines has a definite effect on the reaction of the system. An example is given from the E-series with the vessel in full load (Figure 4). It is seen that in the nylon lines (MD1, 2, 3, 4) the effect of increasing the pretensioning from 5 tons per rope (15 tons per bundle) to 10 tons per rope (30 tons per bundle) is negligible. Even with the steel lines (S1 and S2), the tendency still holds true. On the contrary it was noted that the effect on the motions of the vessel is significant. Although motions were not recorded, visual observations and approximate measurements were sufficient to attribute adverse vessel motions to low pretensioning.

**Effect of Nylon-tails** - This effect was studied by duplicating the entire E-series tests (total of 112) in the D-series tests which differed only in that nylon-tails of ten meters were added to the spring lines. Figure 5 gives a typical comparison of the line forces in both series of tests with the vessel in full load condition and 10 ton pretensioning. The results show that with the addition of nylon-tails the forces in S1 and S2 are reduced greatly while the forces in MD1, 2, 3, and 4 remain practically the same as what they would be without the nylon-tails. Therefore, the net result is that the nylon-tails help to moderate the force distribution in the system.

**Effect of Vessel Loading** - A typical set of results is given in Figure 6 to compare the spring line forces generated under full load and ballast conditions. The forces are much higher when the vessel is in ballast. This is generally true for all other testing conditions.
Effect of Wave Height - Except in adverse situations, there is a consistent tendency for the forces to increase with the wave height (Figure 7). The results clearly show that the increase is not proportional to the increase in the wave height. Also exceptions seem to be numerous making a definite generalization impossible - the observations on the wave period especially apply to this parameter.

Wave Period and Other Factors - One of the most important factors that affect the ship's motions and the forces in the system is the wave period. From the results given in Figures 3 to 10 it is seen that the wave period in many situations could be the deciding factor. This is true especially when the wave period is close to one or several natural periods of the vessel motion. The vessel's periods of roll, pitch, and heave are in the range of 11 to 14 seconds. The results showed that the line forces increase as the wave period exceeds 10 seconds. Obviously this is when the vessel motions in the six degrees of freedom approach the state of resonance, a phenomenon too complex to have been treated within the scope of this work. Another factor that must affect the dynamics of the system is the fendering, which was also not studied in this program.

Modeling Techniques

This discussion is considered particularly useful because much remains to be investigated and developed in the field of vessel motion modeling. This is true as much for physical modeling as for mathematical modeling although in each approach the difficulties may be different. The present study has helped this cause in a number of ways, some of which are selected for discussion in the following sections.

Effect of Wind - At the outset of this study concerns were expressed that the program did not include tests with wind simulation. It was felt that the effect might be significant especially with the vessel in ballast condition due to the larger exposure. Both the E-series and the D-series were then tested with steady wind of about 65 km/hr approximately 30° with the face of the berth (the same direction as the waves). A typical set of results is given in Figure 8 - vessel in ballast condition, wave height of 3.05 m, and 10 ton pretensioning. The forces in the spring lines S1 and S2 were plotted for both series and the forces in these lines clearly prove that the effect of the wind on the system is insignificant in comparison with the forces generated by other parameters.

Simulation of Rope Characteristics - In the beginning of the program several series of tests were conducted with the elongation properties of the steel as well as the nylon ropes being simulated by coil springs. A spring has linear strain/elongation properties, giving in this manner a linear approximation rather than an exact duplication of the nylon rope elongation. This is the most common method used in similar studies elsewhere, although some have attempted other elaborate arrangements to obtain closer representation. The plunger system devised as a result of this problem proved to be very practical and yet highly accurate in duplicating force/elongation properties of materials. Therefore this system was used in the bulk of the tests. Figure 9 is given to illustrate the magnitude of difference in the results if the non-linear properties of the mooring lines were to be simulated instead of the linear approximation obtained by the use of coil springs. For all practical reasons, the results show that the concept of linearizing the load/elongation curves is an acceptable model simplification when
waves and the ship motions are themselves small enough to be linearized. In applications where waves are well within the non-linear range (such as breaking waves) or very large vessel motions are expected, the non-linear model would be preferred. Otherwise, because of other factors which have more pronounced effects on the dynamics of the system, non-linear mechanisms would be unnecessary from the point of view of improving the results.

Analytical Model - The Department of Public Works recently purchased from a private consultant an analytical model (MOSA/DPW) treating the dynamics of moored vessels (3). This model is still at the developmental stage, however Public Works have made some test-runs with a set of data prepared from the site and vessel parameters tested on the physical model. The data for the computer were prepared from the E-series with the vessel in ballast condition (except that in the analytical model the waves were at 22.5° to the longitudinal axis of the vessel instead of 30° used in the physical model). A comparison of results from the two models is given in Figure 10. The analytical (MOSA/DPW) results, with a few exceptions, seem to follow the same trends as those from the physical model, however the analytical results are mostly lower especially with higher wave periods. At a 15 second wave period the forces in S1 and S2 from the analytical model are about 50% of those measured on the physical model. This difference warrants extreme caution in the use of the analytical model (MOSA/DPW) at its present state.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are made on the basis of results from the testing programs discussed in this paper. They apply to the conditions tested in the study and might be extended to similar site, vessel, and sea conditions only according to individual experience and judgement.

A balanced force distribution in the system is very important. When the elongation properties of the mooring lines differ greatly, as is the situation with steel and nylon lines, the force distribution in the system could be severely unbalanced. Especially if the circumstances necessitate the use of steel ropes for the spring lines and nylon or similar synthetic ropes for all the other lines, the spring lines would absorb most of the system's energy. These lines could reach their breaking point while the others are far from their optimum capacity. Such a mooring arrangement should be avoided or used with caution under moderate conditions only.

Addition of nylon-tails on steel lines has definite advantages and should be encouraged whenever operationally possible. It was shown that nylon-tails greatly reduce the forces in the spring lines resulting in a more balanced force distribution in the mooring system.

Tests showed that increasing the pretensioning from 5 to 10 tons per rope reduces the vessel motions without increasing the line forces. To ensure that risk of damages to the manifold and other terminal facilities is minimized, it is recommended that the pretensioning is closely monitored and maintained at 10 tons/rope. When the sea is relatively calm this can be done with some accuracy if there are force measurement devices at the site. Otherwise, the skill and experience of individual operators would have to be relied upon.
Table 1. Maximum mooring-line forces obtained from the HRC physical model and DPW analytical models MOSA1 and MOSA2 in each case the vessel in ballast condition with 10-ton pretensioning and 10 meter nylon-tails on the steel lines (D-series).

**HRC - Physical Model**

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>MD1</th>
<th>MD2</th>
<th>S1</th>
<th>S2</th>
<th>MD3</th>
<th>MD4</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>35</td>
<td>-</td>
<td>10</td>
<td>-</td>
<td>18</td>
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</tr>
<tr>
<td>7.5</td>
<td>36</td>
<td>40</td>
<td>9</td>
<td>11</td>
<td>17</td>
<td>29</td>
</tr>
<tr>
<td>10.0</td>
<td>45</td>
<td>49</td>
<td>31</td>
<td>12</td>
<td>41</td>
<td>55</td>
</tr>
<tr>
<td>12.5</td>
<td>44</td>
<td>40</td>
<td>30</td>
<td>15</td>
<td>40</td>
<td>33</td>
</tr>
<tr>
<td>15.0</td>
<td>57</td>
<td>57</td>
<td>92</td>
<td>107</td>
<td>77</td>
<td>89</td>
</tr>
</tbody>
</table>

**DPW - Analytical model, MOSA1**

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>MD1</th>
<th>MD2</th>
<th>S1</th>
<th>S2</th>
<th>MD3</th>
<th>MD4</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>31</td>
<td>31</td>
<td>9</td>
<td>9</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>7.5</td>
<td>32</td>
<td>32</td>
<td>9</td>
<td>9</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
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</tr>
<tr>
<td>12.5</td>
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<td>41</td>
<td>24</td>
<td>38</td>
<td>46</td>
<td>60</td>
</tr>
<tr>
<td>15.0</td>
<td>45</td>
<td>52</td>
<td>68</td>
<td>85</td>
<td>44</td>
<td>75</td>
</tr>
</tbody>
</table>

**DPW - Analytical model, MOSA2**

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>MD1</th>
<th>MD2</th>
<th>S1</th>
<th>S2</th>
<th>MD3</th>
<th>MD4</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>31</td>
<td>31</td>
<td>9</td>
<td>9</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>7.5</td>
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<td>16</td>
</tr>
<tr>
<td>10.0</td>
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<td>10</td>
<td>11</td>
</tr>
<tr>
<td>12.5</td>
<td>36</td>
<td>35</td>
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<td>24</td>
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<td>19</td>
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<tr>
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<td>45</td>
<td>26</td>
<td>27</td>
<td>53</td>
<td>29</td>
</tr>
</tbody>
</table>
It was proven that when the vessel is in ballast much higher dynamic forces are generated in the mooring lines than when she is in full load. It is recommended that whenever possible some ballasting should be done as the vessel is being unloaded in order to avoid light load conditions. When adverse weather conditions are expected the vessel should never be left lightly loaded.

As a general rule, the line forces can be expected to increase with increasing wave height. Much more important though is the wave period in relation to the natural periods of the particular vessel. When the wave periods are near or equal to the vessel's natural periods, a state of resonance could result causing complex vessel motions and high mooring line forces. It is recommended that operators be familiar with the dynamic characteristics of their vessels and pay special attention when such a situation is expected to develop.

It was shown that in model studies it was unnecessary to simulate the steady wind at the site. The important wind effects are represented by the wave and sea conditions arriving at the site and these should be reproduced on the model in the best methods feasible.

The elongation characteristics of the mooring lines can be simulated by linear springs as an acceptable model simplification without having to sacrifice the practical accuracy of the results. A plunger system was devised for duplicating the non-linearity of the line elongations and this is recommended as a practical solution for added model refinement especially if severe site conditions are anticipated.

Results obtained from a specific analytical model MOSA/DPW showed that the analytical approach to moored ship dynamics is not yet fully developed and should be used with extreme caution. It is recommended that when severe conditions are expected at the terminal site, the situation should be studied on a physical model. The analytical approach would be useful as a back-up to such studies.

(For additional comparisons between the physical model and two versions of the DPW analytical model - MOSA1 and MOSA2 - refer to Table 1. These results were received from DPW at the time this paper was ready for printing. They show that, especially in critical situations such as when the vessel is in ballast, the analytical results deviate greatly from those of the physical model).

REFERENCES


Figure 1. Site and model layout

Figure 2. Mooring arrangement

Figure 3. Force distribution in the system
Figure 4. Line pretensioning

Figure 5. Addition of nylon-tails
Figure 6. Effect of loading

Figure 7. Effect of wave height

Figure 8. Effect of wind
Figure 9. Simulation of rope elongation

Figure 10. Comparison of physical and analytical models
ABSTRACT

The forthcoming "TERMPOL" Code prepared by the Canadian Government recommends the provision of quick release mooring hooks and capstans for all future Canadian terminals handling ships in excess of 140,000 DWT. It further recommends that if severe wind, wave, current or ice action are a possibility, mooring hooks should be fitted with instrumentation to continuously monitor mooring line loads. Then, if the line loads become excessive, appropriate action can be taken.

Mooring force measurements were first made by EXXON Engineering in Milford Haven U.K. as a means of balancing mooring systems for VLCC's subjected to severe loads imparted by passing ships. Similar applications exist at major terminals where extreme forces can be imposed on mooring lines and fenders not only from environmental sources, but also from operational procedures such as offloading or line tending.

Over the past two years, an instrumentation system has been developed and installed at the Come-By-Chance oil terminal wharf. This instrumentation is designed to act both as an operational tool giving a constant readout of forces in mooring lines with an audio visual warning should line forces exceed specified limits, and also as a means of gathering engineering data on moored ship response and resultant forces for measured excitation forces.

The purpose of this paper is to cover the experience gained in developing the instrumentation system, including technical details, problems encountered, and factors to be considered in future installations.

INTRODUCTION

In 1974 the Canadian Government entered into an agreement with Japan on the exchange of technical information on the design of major marine terminals. After a state of the art review of current design procedures and problem areas it was decided to proceed in these areas:
(i) The development of mathematical models and computer programs to analyse the motions and forces generated by moored ships subjected to wind, wave and current.

(ii) The carrying out of extensive hydraulic studies for verification of computer results and general studies.

(iii) The instrumentation of an existing facility to determine actual loads developed; this information being used to ultimately verify the mathematical and hydraulic approaches as well as to provide general engineering data currently not available.

This paper will focus on the mooring instrumentation phase of this study.

The instrumentation system developed for the Come-by-Chance facility has a dual purpose. In the first instance the system has been designed to provide information on mooring line and fender loads developed by the moored ship. These data are correlated to wind speed and direction, wave action, current speed and direction, tide and ship characteristics including offloading practice, trim and overall geometry. Data are also collected on mooring line characteristics and general procedures followed while the ship is at the berth.

The system also provides an operational tool for dock personnel by giving a constant readout of all line loads thus identifying potentially dangerous situations so that appropriate action can be taken thereby preventing the breaking of lines or ultimately the ship breaking free with resultant damage to the structure, ship and the environment. To assist in this task an audio visual warning is provided should line loads exceed specified limits. Remedial action generally involves a balancing of the mooring systems so that all lines carry equivalent portions of the total restraining load. The need for balancing has been emphasized by studies conducted by National Engineering Laboratory, East Kilbride, U.K. which have demonstrated that in some cases 95% of the total mooring load for a VLCC was being taken by not more than three mooring lines out of a mooring pattern of up to twenty. Should a line break and the ship start moving a chain reaction may occur with the ship ultimately breaking loose from the dock. This is in part due to the fact that a rope subjected to a dynamic loading is appreciably weaker than its stated breaking load may suggest.

INITIAL INVESTIGATIONS

Once the decision was made to instrument the Come-By-Chance facility initial studies were carried out on the mooring hooks using both photoelastic techniques and field tests to determine hook sensitivity and stress distributions within the side plates. Based on these studies it was determined that mooring loads could be monitored by incorporating electrical resistance strain gauges within the horizontal swivel pin or alternately by attaching a transducer directly to the hook side plate. Because the terminal was in frequent use at the time it was decided to go with the transducer affixed directly to the side plate as this would not interfere with the normal dock operation.

Before proceeding with the system design a study of existing systems was carried out. The most directly related was the system installed at the Esso dock in
Milford Haven, U.K. This consisted of strain gauges attached directly to the hook side plates with a readout in the control room. Although there were plans for EXXON Engineering Co. to conduct data gathering at the site the primary purpose of this system was to balance the mooring system thereby reducing the danger of extreme forces developed by ships passing very close to the moored ship. The only other mooring instrumentation known of at the time was the use of a load sensor at the TETNEY Mono-bouy in the Humber. In all of these cases the objective was primarily operational with no real attempt made to correlate directly the monitored loads with the environmental conditions at the time.

**SYSTEM DESCRIPTION**

The system can be considered as two major components; the load transducer including cables, junction boxes etc and the display facility including the amplifiers and associated equipment.

The main display panel, as shown in figure 1, consists of 22 individual readout meters giving mooring line load as metric tonnes and 2 meters giving fender deflection in centimeters. Each indicator is equipped with an adjustable upper limit which will give an audio and visual alarm should mooring line or fender loads become excessive. In practice this upper limit is determined by the operating personnel depending on the mooring line type, size and condition. A mimic panel is used to attach magnetic scaled ship shapes and to indicate the mooring pattern using a grease pencil. For ease of operation the display panel is located within the main terminal control room where any overload situation noted can be transmitted directly to the ship's bridge or to the dock personnel.

To conserve space, the signals to this panel come from the main amplifier cabinet located in another room. This cabinet contains all amplifiers, power supplies and associated electronic hardware.

The transducer excitation voltage (approx. 10 V.D.C.) and signal amplification requirements are supplied by 24 load cell amplifiers. Meters on each amplifier are used for shunt calibration of the transducers and can read load to a resolution of .1% of full scale.

Direct data recording capability is provided by using an analog voltage signal to a terminal strip located in the rear of the cabinet. The full scale signal range is 0-1 volt DC from a 200 ohm short circuit protected source. The signal response time is 1.5 milliseconds for 90% of the step input. Thus with a high speed recorder it is possible to examine rapidly changing loads should dynamic characteristics be evident in the data gathering process. Provision is also made at this point to record the output of associated equipment including an anemometer, current meter, and wave/tide gauge.

The excitation voltage to and signal return from the transducers are transmitted by individual electrically shielded cables protected by rigid conduit. Each cable is terminated at a junction box in the proximity of the mooring hook cluster. At this point a terminal strip is used to connect to the wiring coming from the transducer which is protected by a flexible, hydraulic type hose.
The original transducer, as shown in figure 2, consists of a strain gauge type extension transducer welded to one of the shackle plates of each hook assembly.

The point of attachment to the side plate was determined by extensive strain measurements on the hook assembly taking into account the torsional effects noted in the side plates.

After each transducer was welded to the side plate the system was calibrated using a hydraulic jack assembly to apply a known load to the mooring hook. The output voltage of the transducer under load was measured, with the amplifier gain being adjusted to give a corresponding reading on the display panel. The system was also checked using a separate load cell placed in a towing line between the hook assembly and a 3500 HP tug. In general a good correlation was found.

PROBLEMS ENCOUNTERED

Although the short term performance of the system was satisfactory, the long term reliability was unacceptable.

Over relatively short periods of time, often a period of hours, considerable zero point drift was noted. For all hooks this exceeded the specified 5% accuracy and in some cases system drift in excess of 30% was noted. It is also worthy of note that this drift was not consistent in each channel thereby making it impossible to use a reference channel to apply a correction factor to readings taken or other channels. A typical record of system drift versus temperature is shown in figure 3, this variation occurred with no load applied to the hook.

After an extensive investigation the following reasons were identified as probable sources of system drift.

1. Moisture ingress; as the transducer, junction box, connecting hose and cabling system are constantly exposed to the marine environment every effort was made to thoroughly seal all components, with checks made on insulation resistance.

2. Inherent stresses; the type of mooring hook involved was of welded fabrication and it is very unlikely that any stress relieving was carried out either before or after assembly. Every time the hook structure is exercised, that is taken from zero load to some finite value of load and returned to zero, it is probable that these inherent stresses are redistributed about the assembly. Furthermore the position on the hook selected for installation of the transducer is an area of intensive welding where the described effects would be accentuated. This was confirmed by tapping the hook with a small hammer thereby causing significant zero point shift.

3. The positioning of the transducer; as this position was asymmetric to the line of action of the load it was impossible to forecast accurately the proportion of the total mooring load transmitted along the respective side plates. It is reasonable to assume that up to 10% variation can be expected
depending on the swivel bearings, position of the hook and torsional forces exerted by the mooring line.

4. Notwithstanding the validity of (1), (2) and (3) the main contributory cause of the substantial drift was identified as temperature. As the transducer was mounted in a very exposed position and its thermal capacity is much lower than that of the hook structure on which it was mounted the mass temperature effect would therefore be totally different. For example on a warm day, with direct sunlight the temperature of the transducer will rise very much faster than that of the massive hook structure. This differential expansion will place the transducer in compression which will make the output appear low as a result of this thermally induced strain. Although the transducer is rated as having a very acceptable coefficient of temperature compensation in isolation, the compensation networks are not able to differentiate between the changing characteristics of the transducer itself and the thermally induced strains imposed by outside agencies. This conclusion was verified by field observations. For example, referring to figure 3, a temperature rise of 50°F on day 6 produces a corresponding drop in perceived load of 4 tonnes; whereas a temperature drop of 80°F on day 4 produces a perceived load increase of 6 tonnes. This effect was also evident in a series of readings taken on hourly intervals.

It is of interest to note that the similar system installed at Milford Haven, U.K. is also subject to such a drift problem requiring frequent recalibration.

Several alternate solutions were considered such as some means of retarding the transducer response to thermal effects, provision of a rather sophisticated temperature compensating device or replacement of the transducer.

Neither of the first two alternatives was given serious consideration as a clear solution was not evident. It was decided to replace all transducers should an acceptable alternate, compatible with the existing system, be located. A review of current technology showed increasing use of instrumented shear pins at other facilities. To evaluate this approach a single pin was installed in one of the hooks and closely monitored over a period of several months. Although drift in the other transducers remained excessive, zero drift was noted in the test transducer. Based on this experience it was decided to proceed with the replacement of all existing transducers with instrumented shear pins.

In general these instrumented pins, as shown in Figure 4, are designed to replace the existing horizontal swivel pin through which the total load imparted by the mooring line may be considered to act; thus putting the pin in a condition of double shear. The active element of these transducers are electrical resistance strain gauges which produce a voltage output proportional to their degree of deformation when under load.

As the pin is an independent unit and not a fixed part of the hook structure any redistribution of residual stresses in the hook assembly does not affect it. Load distribution is not a factor with such a transducer as the total mooring load must act through the shear planes of the pin. A slight reduction (1 mm)
over the area of the shear planes ensures that the gauged area is not subject to compressive stresses.

Probably the major feature in favour of the instrumented pin, however, is that it is independent of the hook structure, thereby being free to expand and contract, with any variation in output being corrected for by the inherent compensation. Actual temperature differentials however are minimized by the mass of the pin (approx. 30 Kg) and the fact that the transducer is almost totally enclosed by the massive hook structure.

The technical specification of each type of transducer may be summarized as follows:

<table>
<thead>
<tr>
<th></th>
<th>EXTENSION TRANSUDER</th>
<th>INSTRUMENTED SHEAR PIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principle of Operation</td>
<td>Electrical Resistance</td>
<td>Electrical Resistance</td>
</tr>
<tr>
<td></td>
<td>Strain Gauge</td>
<td>Strain Gauge</td>
</tr>
<tr>
<td>Load Rating</td>
<td>0 - 100 tonnes</td>
<td>0 - 100 tonnes</td>
</tr>
<tr>
<td>Excitation Voltage</td>
<td>10 V.D.C.</td>
<td>10 V.D.C.</td>
</tr>
<tr>
<td>Sensitivity</td>
<td>0.4 to 0.85 MV/V</td>
<td>.44 MV/V</td>
</tr>
<tr>
<td>Temp. Effect</td>
<td>.005% of Full Scale</td>
<td>Fully Compensated</td>
</tr>
<tr>
<td>System Accuracy</td>
<td>5% of Full Scale</td>
<td>1.25% of Full Scale</td>
</tr>
<tr>
<td>Response Time</td>
<td>100 μ sec</td>
<td>100 μ sec</td>
</tr>
<tr>
<td>Bridge Resistance</td>
<td>480 Ω</td>
<td>700 Ω</td>
</tr>
</tbody>
</table>

Installation and calibration of the alternate transducer was a relatively simple matter as the pin was easily replaced and the load cell had been pre-calibrated thereby requiring only adjustments to the zero settings and gain on the existing amplifiers. A check was made on the system accuracy using the hydraulic jack with very good correlation. To date zero system drift has been observed.

The Canadian Government is presently in the process of preparing a code of recommended standards for the prevention of pollution at marine terminals, generally referred to as the "TERMPOL CODE". This code will be used to evaluate, from the safety viewpoint, proposals for future marine terminals to be constructed in Canada. A specific recommendation of this code will be the provision of a mooring load monitoring system for all terminals handling ships in excess of 140,000 DWT where large line loads can be anticipated. With this requirement it is felt a significant contribution will be made to general terminal safety in Canada and in this regard the experience at the Come-By-Chance facility should prove beneficial in the design of future installations.
DISPLAY PANEL
Figure 1.

EXTENSION TRANSUDER
Figure 2.
SYSTEM DRIFT (Typical)

Figure 3.

INSTRUMENTED SHEAR PIN

Figure 4
MOORED SHIPS EXPOSED TO COMBINED WAVES AND CURRENT

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ABSTRACT

This paper presents information on the behaviour of moored ships under the combined action of waves and current. This information was developed mainly through a program of hydraulic model testing, conducted as a part of engineering studies for a proposed oil terminal in the Bay of Fundy, in which the effect of strong tidal currents and waves from the open Atlantic, to which the site was exposed, were realistically simulated. The mooring lines on the ships and the fenders were modelled to incorporate their non-linearities, and the site wave records were used to develop the wave excitation in the model basin.

The results showed that the combined action of waves and current was not just the sum of the separate effects but a complex relationship dependent upon wave spectra, wave periods and current direction. For currents running nearly head or stern on, the yaw motions of the ship were amplified compared to wave action alone. For currents at an angle to the ship, all mooring forces and ship motions could be strongly amplified compared to wave action alone.

For the particular site considered, the analyses and model tests pointed to the use of a medium soft mooring system (wires with nylon tails and compatible fenders) as the best means of simultaneously minimizing the ship motions and the mooring line forces.

INTRODUCTION

During 1975-1976 design studies and hydraulic model tests were conducted by a joint venture of Eastern Designers of Fredericton, N.B. and Swan Wooster Engineering of Vancouver, B.C. to construct an oil terminal in the Bay of Fundy in Eastern Canada for tankers in the 26,900 DWT to 100,000 DWT range. The site is exposed to the Atlantic from the south and the significant wave heights exceed 1.2 m 20% of the time and 1.8 m 7% of the time in winter (Oct.-Mar.). The offshore and onshore winds exceed 20 knots about 5% of the time. The maximum tidal range is about 9 m and the maximum tidal currents exceed 1.7 knots about 100 days of the year.

From the beginning it was recognized that the waves would be the major cause of motions of ships moored to the berth. Also it was recognized that currents would give rise to additional forces due to statical effects and might also interact dynamically...
with the wave induced motions. Due to the significant wave and current exposure of the site, it was decided to investigate the problem in detail.

The fixed berth layout that emerged from the design studies is shown schematically in Fig. 1. It consisted of a central loading platform, two berthing dolphins faced by pneumatic fenders and two mooring dolphins on each side of the berthing dolphins.

This paper presents some results of the analytical and hydraulic model investigations of combined wave and current effects on moored ships.

**ANALYTICAL ASSESSMENT OF PROBLEM**

To fully understand the problem of the combined action of waves and current on moored ships, initially it is useful to consider their actions separately.

**Wave Action on Moored Ships**

The response of moored ships to waves is governed mainly by the proximity of the periods of the incoming waves to the natural periods of the moored ship. For large tankers the periods for heave, pitch and roll are in the order of 7-15 seconds and they can be in resonance with the incoming waves. Usually, however, the large damping in the heave and pitch modes keeps movements in these modes down. The natural periods of the horizontal motions sway, surge and yaw, on the other hand, are somewhat longer. Typically they are in the range 20-80 seconds depending on the stiffness of the mooring ropes. This means that they are far longer than the ordinary wave periods and consequently the response of the ship in these modes directly to waves is weak. Instead the ship responds to the wave radiation stress (also known as wave drift force) which is a unidirectional stress acting in the direction of the wave propagation. For regular waves it is constant in time and gives rise to a total force over the water depth approximately equal to

\[
F = \frac{1}{16} \gamma H^2 (1 + \frac{2}{L} \frac{2\pi h}{\sinh \frac{2\pi h}{L}}
\]

in which \(F\) is the Force per unit width, \(\gamma\) is the unit weight of water, \(H\) the wave height, \(h\) the water depth and \(L\) the wave length (Longuet-Higgins, 1971). In irregular waves, however, this stress will vary with the difference in wave height, i.e. it will vary with the wave groups as shown in Fig. 3; but it will still act in the direction of the wave propagation. The periods of the wave groups will from time to time be close to resonance with the horizontal modes giving rise to very large ship motions and mooring forces. The damping in these modes is very small because of the long periodic motion and so usually it is the wave group effect, which governs the magnitude of mooring forces and ship motions for large tankers. In Fig. 3 the response of a moored ship to waves is shown and it is obvious that it is the wave group period, which dominates the response. Unfortunately at present, no theory is fully capable of determining the response of ships to variations in radiation stress in irregular waves so hydraulic model tests are necessary for obtaining reasonable results. However, a theoretical formulation can be developed as shown in equations 6 to 8.
Current Effects on Moored Ships

The effects of current on moored ships depend on the angle of incidence of the current, the form of the hull as well as motions of the ship. In the following, only small angles of incidence are considered because wharfs are usually aligned with the current. Furthermore, it is assumed that the ship motions are small. Hence linearized expressions for current forces on ships can be applied. They are as follows with definitions and orientation of coordinate system as shown in Fig. 2.

1. A drag force $D$ acting along the longitudinal axis of the ship

$$D = \zeta_0 \rho V^2 L b$$

in which $\rho$ is the density of water, $V$ the current velocity, $b$ the beam of the ship, $L$ the length of the ship and $\zeta_0$ a non-dimensional coefficient.

2. A lateral force $N$ acting at right angles to the axis of the ship upstream of the centre of gravity of the ship.

$$N = -\xi_1 \rho V^2 L d\theta_e$$

in which $d$ is the draft of the ship, $\xi_1$ a non-dimensional coefficient and $\theta_e$ the effective angle between ship and current whether the ship is at rest or in motion.

$$\dot{\theta_e} = \dot{\theta} + \arctan \frac{\dot{y}}{V}$$

in which the dot means differentiation with respect to time and $y$ is the transverse coordinate. $N$ is acting at a distance 'a' upstream of the center of gravity of the ship. Empirically 'a' is found to be in the range $\frac{L}{4}$ to $\frac{L}{3}$. $N$ is acting at a distance 'a' upstream of the center of gravity of the ship. Empirically 'a' is found to be in the range $\frac{L}{4}$ to $\frac{L}{3}$.

3. A side force $H$ due to the water particle motions induced by the rotation of the ship around a vertical axis through the center of gravity (Magnus Effect)

$$H = -\xi_2 \rho V L^2 \dot{\theta}$$

$H$ is acting downstream of the centre of gravity at a distance $c = 0.6 L$ which means that this force will tend to damp any rotation.

All the coefficients $\zeta_0$, $\xi_1$ and $\xi_2$ and the moment arms $a$ and $c$ are dependent on the type of ship, the depth of water and the mode of motion.

Current does not only induce steady forces on moored ships. For some mooring configurations and for certain hull forms, motions of the ship can be induced. These motions are self-sustained and are related to aerodynamic phenomena such as flutter of aeroplane wings and galloping of transmission lines. As shown in Fig.7 and 9 the phenomenon can induce rather high loads in the mooring ropes. The problem has been treated in references (Hansen, 1968; Ottesen-Hansen, 1976; and Wickers, 1976).

The Combined Effect of Waves and Current

The analysis is most conveniently carried out by considering the equations of motions for the ship. Neglecting the small contributions from roll, heave and pitch motions and assuming all movements small compared with the dimensions of the ship the equations of motion are as follows using the reference system shown in Fig. 2.
\[
M \ddot{x} + 2B \omega \omega M \dot{x} + k_{xx} \dot{x} + k_{xy} \dot{y} + k_{x\theta} \dot{\theta} = F_x + D_{rx} \tag{6}
\]
\[
M \ddot{y} + 2B \omega \omega M \dot{y} + k_{yy} \dot{y} + k_{y\theta} \dot{\theta} = F_y + N + H + D \sin \theta \tag{7}
\]
\[
I_\theta \ddot{\theta} + 2B \omega \omega I \dot{\theta} + k_{\theta x} \dot{x} + k_{\theta y} \dot{y} + k_{\theta\theta} \dot{\theta} = M_r - N.a + H.c \tag{8}
\]

\(M_x\) and \(M_y\) are the virtual masses of the ship for the surge and sway motion respectively, \(y\) the virtual moment of the yaw motion, \(F_x\) and \(F_y\) are the resulting components of the radiation stress acting on the ship, \(M_x\) the resulting moment of the radiation stress around the centre of gravity of the ship, \(k_{xx}, k_{yy}, k_{\theta\theta}, k_{xy}, k_{x\theta}\) and \(k_{y\theta}\) are the stiffnesses for the mooring system for the surge, sway, yaw and coupled yaw-sway-surge motions. Finally \(\beta_x, \beta_y\) and \(\beta_\theta\) are the ratios between the actual damping and the critical damping of the wave induced motions.

To show how the presence of a current changes the wave induced motions the equations (6), (7) and (8) are rewritten to express the change in movements due to currents when the motions due to waves are known.

Considering the movements \(x, y, \theta\) generated by the waves alone then the total movements \(x, y\) and \(\theta\) due to the combined effect of waves and current can be expressed as:
\[
x = x_w + \Delta x, \quad y = y_w + \Delta y \quad \text{and} \quad \theta = \theta_w + \Delta \theta \tag{9}
\]
in which \(\Delta x, \Delta y\) and \(\Delta \theta\) are the corrections to the wave-induced motion. Inserting (9) in (6), (7) and (8) \(\Delta x, \Delta y\) and \(\Delta \theta\) can be expressed by means of \(x_w, y_w\) and \(\theta_w\)
\[
M_x \ddot{x}_w + 2B \omega \omega M \dot{x}_w + k_{xx} \dot{x}_w + k_{xy} \dot{y}_w + k_{x\theta} \dot{\theta}_w = D(x, y, \theta) \tag{10}
\]
\[
M_y \ddot{y}_w + 2B \omega \omega M \dot{y}_w + k_{yy} \dot{y}_w + k_{y\theta} \dot{\theta}_w = N(x, y, \theta) + H(x, y, \theta) + D(x, y, \theta) \sin \theta \tag{11}
\]
\[
I_\theta \ddot{\theta}_w + 2B \omega \omega I \dot{\theta}_w + k_{\theta x} \dot{x}_w + k_{\theta y} \dot{y}_w + k_{\theta\theta} \dot{\theta}_w = -N(x, y, \theta) a + H(x, y, \theta) c \tag{12}
\]

Provided the expressions for \(x, y\) and \(\theta\) are known, solutions for \(x, y\) and \(\theta\) can now be found. Assume that \(y_w, y_w\) and \(\theta_w\) are given by
\[
x_w = x_0 e^{i\omega t}, \quad y_w = y_0 e^{i\omega t} \quad \text{and} \quad \theta_w = \theta_0 e^{i\omega t} \tag{13}
\]
in which \(i\) is the imaginary argument and \(\omega\) is the cyclic frequency of the wave groups then the expressions for \(\Delta x, \Delta y\) and \(\Delta \theta\) are
\[
\Delta x = y_1(x_0, y_0, \theta_0) e^{i\omega t}, \quad \Delta y = y_1(x_0, y_0, \theta_0) e^{i\omega t}, \quad \Delta \theta = \theta_1(x_0, y_0, \theta_0) e^{i\omega t} \tag{14}
\]
in which \(x_1, y_1\) and \(\theta_1\) are the linear functions of \(x_0, y_0\) and \(\theta_0\). The general solution to eq. 14 is dependent upon many parameters so a complete solution is not presented here. Instead solutions for beam-on waves are shown in fig.4 in which it
is seen that the current amplifies the wave-induced bow motion and decreases the sway motion when the wave group periods are close to the ship natural periods whereas virtually no change takes place for periods far away from resonance. Using information given in Fig. 10 the theory is compared with measurements in Fig. 5 and the agreement is satisfactory. The bow motion is amplified by the waves with 5.7 and 10.5 sec peak spectral period, \( T_p \), but not by the waves with \( T_p = 7.2 \) sec.

**MODEL SET UP**

The experiments were conducted in a 26 x 16 m basin in which a model of the site was built to a scale 1:100. The coastline and sea bottom were modelled 1000 metres to each side of the terminal and approximately 600-800m off the coast. The layout of the berth shown in Fig. 1 was kept during the experiments.

Two different tankers were used in the investigation, a 100,000 DWT and a 26,900 DWT model of existing ships. They were fitted with mooring lines attached to a system of springs giving them the same non-linear stiffness characteristics as that of the real ropes. The fendering systems too were modelled by a system of springs which were adjusted to give the necessary non-linear relationship between force and deflection.

The waves used in the model tests were samples of recordings taken at the site in an earlier year. This assured that the group properties of the model waves were exactly the same as those of the real waves. The technique for the wave reproduction is described by Sorensen(1973).

**Results of Experiments**

The results of the different experiments are shown in Fig. 5 to 9. All motions (double amplitude) and forces shown are the maximum recorded in 20 minutes. The wave trains are identified by their peak spectral periods, \( T_p \), although these periods are not as important as the wave group periods. Of the waves used those with periods of 5.7 sec, 6.2 sec and 12 sec have pronounced group properties corresponding to the natural periods of the ship, whereas the group properties for the 7.2 sec and the 10 sec waves are somewhat weaker. All mooring lines have a 134 kN pretension.

Generally the presence of currents in waves tends to increase the yaw motion and decrease the sway motion. The effect is clearly seen in Fig. 5 and 6 in which the results for a 100,000 DWT tanker moored with very soft ropes are shown. The effect is pronounced too when stiffer ropes are used as shown in Fig. 7 and 8 where the 100,000 DWT tanker is moored with steel wire ropes with nylon tails. The sway motion is somewhat decreased whereas the bow motion is both increased and decreased depending on the wave train. The pure yaw motion is not shown but comparing the magnitudes of the sway and bow motions it is seen that in every case the yaw motions have been amplified by the current. Finally in Fig. 9 the combined effect of waves with relatively strong currents are shown. For a current running parallel with the ship the sway motions remain constant whereas the yaw motions and consequently the bow motions are amplified. For a 3 kn. current running with an angle of 10° to the ship the picture is more confused as the movements are decreased by the 5.7 waves whereas they are amplified by the 7.2 sec. waves.

The stiffness of the mooring system is important for the magnitude of forces and motions. Soft mooring lines such as nylon give rise to large motions but rather small mooring forces whereas a stiffer system like wire hawsers limits the movements.
but increases the forces. For the particular project the tests pointed towards the use of a medium soft mooring system (wire hawsers with nylon tails and compatible fenders) as the best way to simultaneously minimize ship motions and mooring forces. The selection of the mooring system is described by Khanna and Birt (1977).

MODEL TESTS VERSUS ANALYTICAL METHODS

It is of interest to compare the results from the model tests with the analytical methods discussed earlier. Two different comparisons have been made. In the first comparison the current forces have been calculated by the methods given by OCIMF (1977) and these forces have been added to the test results for wave action alone. In the second comparison the current forces are calculated as in the first case whereas the wave forces are calculated by a method similar to those discussed by Hsu et al (1970) and Rye et al (1975). The added mass and damping coefficients have been evaluated from the model test records and shown in Fig. 10 and the wave radiation stress given by eq. 1 have been used in the calculations. The comparisons are shown in Figs. 11 and 12. It is seen the calculations are not too far off for the situations shown in Fig. 8 whereas the agreement between calculations and tests is poor for the situation shown in Fig. 9. Both calculation methods use the method of superposition i.e. the effect from wave action and current action considered separately are added to give the combined effect. This is not true as demonstrated by the theory leading to eq. 14. For quick and rough assessments the superposition principle can be applied but for more detailed evaluation model tests are considered necessary until better analytical methods are developed.

CONCLUSIONS

It is concluded that, in general, the combined action of waves and current is not just the sum of the separate effects but a complex relationship dependent upon wave spectra, wave periods, wave group properties, and current direction. For currents running nearly head or stern on, the yaw motions of the ship were amplified compared to wave action alone. For currents at an angle to the ship, all mooring forces and ship motions could be strongly amplified compared to wave action alone.

ACKNOWLEDGEMENTS

The hydraulic model tests were conducted at the Danish Hydraulic Institute, Copenhagen, Denmark.

REFERENCES


FIG. 1 LAYOUT OF OIL TERMINAL AND MOORING SYSTEM.

FIG. 2 DEFINITION OF REFERENCE SYSTEM.

FIG. 3 RESPONSE OF A 100,000 DWT TANKER FULLY LOADED TO WAVES WITH $H_s = 1.2$ m AND MEAN SPECTRAL PERIOD $T_p = 5.7$ sec.
FIG. 4
SOLUTION TO EQ. 14
FOR BEAM WAVES AND
CURRENT RUNNING
PARALLEL WITH THE SHIP.
SHIP: 100,000 DWT
FULLY LOADED.
MOORINGS: #65 mm NYLON
ROPS.

\[
\begin{align*}
\alpha & = 0.3 \text{ lpp} \\
b & = 0.6 \text{ lpp} \\
\end{align*}
\]

FIG. 5
MOVEMENTS AND MOORING
FORCES FOR 100,000 DWT
SHIP MOORED WITH #65 mm
NYLON ROPES

SITUATION

LEGEND
- WAVES + CURRENT
- WAVES + CURRENT
- WAVES
- CURRENT

FIG. 6
POWER SPECTRA FOR THE SWAY
MOVEMENT AND THE MOVEMENT
OF THE BOW.
100,000 DWT SHIP IN BALLAST
MOORED WITH #65 mm
NYLON ROPES.

SITUATION

LEGEND
- WAVES & CURRENT
- WAVES
- CURRENT
FIG. 7 100,000 DWT. TANKER
MOORED WITH WIRE
HAWSERS WITH NYLON
TAILS (#81 mm, LENGTH 4.5 m)

SITUATION

LEGEND.

WAVES + CURRENT
WAVES
CURRENT

PRETENSIONING IN ALL MOORING LINES 134 kN

FIG. 8 100,000 DWT. TANKER
MOORED WITH WIRE
HAWSERS WITH NYLON TAILS (#81 mm, LENGTH 4.5 m)

SITUATION

WAVES $H_s = 1.2$ m

ALL FORCES SHOWN ARE THE MAXIMUM RECORDED FORCE IN 20 min.
ALL MOVEMENTS SHOWN ARE THE MAXIMUM RECORDED DOUBLE AMPLITUDE IN 20 min

FIG. 9 100,000 DWT. TANKER
MOORED WITH WIRE
HAWSERS WITH NYLON TAILS (#81 mm, LENGTH 4.5 m)

SITUATION

WAVES $H_s = 1.2$ m

LEGEND

- WAVES + CURRENT 4 kn.
  • WAVES + CURRENT 3 kn.$^{-1}$
  CURRENT 4 kn.
  CURRENT 3 kn.$^{-1}$
FIG. 10  ADDED MASS AND DAMPING COEFFICIENTS FOR 100,000 DWT TANKER

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>65mm NYLON ROPE</th>
<th>81mm NYLON TAILS</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADDED MASS COEFFICIENT SWAY</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>ADDED MASS COEFFICIENT SURGE</td>
<td>0.08</td>
<td>0.08</td>
</tr>
<tr>
<td>ADDED MASS COEFFICIENT YAW</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>DAMPING COEFFICIENT SWAY</td>
<td>0.17</td>
<td>0.11</td>
</tr>
<tr>
<td>DAMPING COEFFICIENT SURGE</td>
<td>0.21</td>
<td>0.30</td>
</tr>
<tr>
<td>DAMPING COEFFICIENT YAW</td>
<td>0.26</td>
<td>0.0165</td>
</tr>
<tr>
<td>RATIO WATER DEPTH TO DRAFT</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

*) THE RELATIVELY HIGH DAMPING IS DUE TO THE FRICTION BETWEEN THE SHIP AND THE FENDERS (RUBBER AGAINST STEEL)

FIG. 11  MAXIMUM MOORING FORCES FOR COMBINED ACTION OF WAVES AND CURRENT. COMPARISON BETWEEN ANALYSIS AND MEASUREMENTS.

SITUATION AS IN FIG. 8

LEGEND
- - - - MEASURED
--- --- WAVE FORCES MEASURED, CURRENT FORCES ACCORDING TO OCIMF (1977) ADDED
--- --- WAVE FORCES CALCULATED, CURRENT FORCES ACCORDING TO OCIMF (1977) ADDED

FIG. 12  MAXIMUM MOORING FORCES FOR COMBINED ACTION OF WINDS AND CURRENT. COMPARISON BETWEEN ANALYSIS AND MEASUREMENTS.

SITUATION AS IN FIG. 9

LEGEND
4 kn 3 kn/10°
• • MEASURED
□ □ WAVE FORCES MEASURED, CURRENT FORCES ACCORDING TO OCIMF (1977) ADDED
△ △ WAVE FORCES CALCULATED, CURRENT FORCES ACCORDING TO OCIMF (1977) ADDED
The paper describes the establishment and operation of a distorted movable model of Taichung Harbor located on the west coast of Taiwan (Fig. 1) where sand migration from the littoral drift, partially nourished by the Ta-Chia River, makes extensive dredging necessary to keep the harbor inlet channel open.

The model employed is a three-dimensional movable-bed Froude model for all prototype depths less than 50 m (164 ft.). At depths in excess of this value the influence of sediment transport is considered negligible and a fixed-bed is used.

The objective of the model study is to investigate the performance of various outer breakwater configurations and to select the optimum configuration which will stabilize the harbor inlet channel, i.e., protect it from excessive shoaling caused by the seasonal wave action and the littoral drift. The wave used in the model corresponds to a wave of 5 m (16 ft.) in height and 12 seconds in period from a northerly direction. The optimum breakwater configuration is determined such that dredging of the harbor inlet will not be necessary during the first 15 years of the structure’s lifetime. After this period of time has elapsed minor yearly dredgings are expected to be needed.

From the results of the investigation, it is recommended:

a) That only the north groin be constructed to supplement the storage capacity of the north breakwater;
b) That no further consideration be given to the construction of a groin system south of the south breakwater;
c) That the sea bottom south of the south breakwater be periodically nourished by discharging whatever material is dredged from the entrance channel during maintenance operations, and that the hopper dredge proposed for such operations be of the type that can pump out the materials in its hoppers and discharge it directly on the beach south of the south breakwater;
d) That sea bottom depths along the proposed seawall south of the south breakwater be determined periodically. When and if they approach critical depths, the toe of the seawall should be reinforced by placing revetment consisting of heavy stones.
Littoral drift model experiments were performed in the large test basin (60 m x 43 m x 1 m) at the Taichung Harbor Hydraulic Laboratory. The model reproduced the area between the mouths of the Ta-Chia and Ta-Thu River, an area of approximately 16 km x 5 km. Therefore, it was necessary to use a distorted movable-bed model with a horizontal scale of 1/500 and a vertical scale of 1/100, a distortion factor of 5.

For a littoral drift model study, there is so far no exact law of similarity between the model and prototype. The relevant way of study is based on the experience of many different tests to determine a semitheoretical, semi-experimental law. In other words, a number of tests under different wave and tide conditions and littoral material supplies are carried out first. By comparing the topographic development in each test with that of the prototype, the test which closely resembles the prototype is selected as the true representative of the prototype. This selection yields the correct wave and tide conditions and littoral material supply for the movable-bed model. The time scale of sediment transport is thus deduced from comparing the corresponding topographic development times in the model and the prototype.

The model was built in accordance with sounding taken in 1911 and the existing breakwaters (see Fig.2.3). During the calibration period, a wide variety of test conditions based on data of meteorological observations and field measurements were tried in the model.

**PRELIMINARY TESTS**

Loose Boundary Model Experiments consisted of determining the best layout of the outer breakwaters to minimize shoaling. In order to conduct these tests, it was first necessary to undertake so-called "Auxiliary Tests". These are more commonly termed "Base Tests", and their purpose is to ascertain the model operating program that resulted in the best simulation of known prototype hydrography at the end of the test. The model bed was moulded to a known prototype hydrography (May 1911) and it was hoped that at the end of the test, the model hydrography would be substantially that of a prototype hydrography determined in June 1913.

Three different model operating programs for these base tests were performed, listed, as shown below.

<table>
<thead>
<tr>
<th>Case</th>
<th>Wave Conditions</th>
<th>Tide</th>
<th>Duration</th>
<th>Rate of introduction of sand supply</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>N</td>
<td>5 m</td>
<td>12 sec</td>
<td>Mn Spring</td>
</tr>
<tr>
<td>2</td>
<td>NNE</td>
<td>5 m</td>
<td>12 sec</td>
<td>Mn Spring</td>
</tr>
<tr>
<td>3</td>
<td>N15°E</td>
<td>5 m</td>
<td>12 sec</td>
<td>Mn Spring</td>
</tr>
</tbody>
</table>

The tide simulated, when plotted against time as the abscissa, was in the form of a sine curve. The location of the point of introduction of the sand supply was at the south bank of Ta Chia River. As stated before, the initial hydrography moulded in the model was that of May 1911. At intervals during the abovedescribed tests, the bed of the model was surveyed and hydrographic charts prepared. The model accumulations were determined in each of seven "Zones" located north of the existing north breakwater, (see Fig.2.3), also the totals for the seven zones. The total accumulation north of the existing north breakwater was taken as 1,200,000 m³/year (prototype) between 1940 and
1945. (Fig. 3b) From Fig. 5, it is known the net littoral drift is from north to south at a rate of about 1.2 million cubic meters per year.

**MAIN EXPERIMENTS**

Based on the preliminary test results, two sets of test conditions for the main experiments were adopted.

(1). Wave direction: N, Wave height: 5 m., wave period: 12 Seconds, Tide: spring tide., the supplied sand quantity: 2 liters/min. Time Scale of Sedimentation: 24 minutes in model = 1 yr in prototype.

(2). Wave direction: N15°E., wave height: 5 m., wave period: 12 seconds, Tide: spring tide., the supplied sand quantity: 2 liters/min. Time Scale of Sedimentation: 35 minutes in model = 1 yr in prototype.

Four arrangements of the outer breakwaters as shown in Fig. 4 were tested for N direction in the model., and three arrangements (two better arrangements selected from N-direction test and another modified arrangement as shown in Fig. 6) were tested for N15°E direction in the model. The results are summarized in Table 1 and Table 2.

**Table 1. Four Different Arrangements with respect to N-Direction Waves and their deposition Phenomena**

<table>
<thead>
<tr>
<th>(1) Arrangement</th>
<th>(2) Maximum deposition rate at harbor entrance (E part) $(10^1 \text{ m}^3/\text{hr})$</th>
<th>(3) Maximum depth at harbor entrance after 8 hr wave action $(\text{m})$</th>
<th>(4) Minimum depth at harbor entrance after 8 hr. wave action $(\text{m})$</th>
<th>(5) Maximum deposition rate in the entire measuring area $(10^1 \text{ m}^3/\text{hr})$</th>
<th>(6) The critical time of obvious shoaling around harbor entrance (min)</th>
<th>(7) The elapsed time before littoral drift started to deposit (min)</th>
<th>(8) Time of littoral drift massively intruded into harbor entrance (min)</th>
<th>(9) Equivalent time of (8) in prototype (yrs)</th>
<th>(10) The depth at harbor entrance before sand transported obviously into harbor $(\text{m})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(2) Maximum deposition rate at harbor entrance (E part) $(10^1 \text{ m}^3/\text{hr})$</td>
<td>50</td>
<td>70</td>
<td>100</td>
<td>120</td>
<td>-3.5</td>
<td>-5.7</td>
<td>-11.1</td>
<td>-1.9</td>
<td>400</td>
</tr>
<tr>
<td>(3) Maximum depth at harbor entrance after 8 hr wave action $(\text{m})$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-3.5</td>
<td>-5.7</td>
<td>-11.1</td>
<td>-1.9</td>
<td></td>
</tr>
</tbody>
</table>
Table 2. Three Different Arrangements with respect to N 15° E Waves and their deposition Phenomena

<table>
<thead>
<tr>
<th>(1)</th>
<th>Arrangement</th>
<th>No. I</th>
<th>No. III</th>
<th>No. V</th>
</tr>
</thead>
<tbody>
<tr>
<td>(2)</td>
<td>Maximum deposition rate at harbor entrance (E part) (10^1 \text{ m}^3/\text{hr})</td>
<td>70</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>(3)</td>
<td>Maximum depth at harbor entrance after 14 hr wave action (\text{m})</td>
<td>-15.0</td>
<td>-12.0</td>
<td>-15.1</td>
</tr>
<tr>
<td>(4)</td>
<td>Minimum depth at harbor entrance after 14 hr wave action (\text{m})</td>
<td>-3.2</td>
<td>-4.0</td>
<td>-8.4</td>
</tr>
<tr>
<td>(5)</td>
<td>Maximum deposition rate in the entire measuring area (10^1 \text{ m}^3/\text{hr})</td>
<td>180</td>
<td>240</td>
<td>160</td>
</tr>
<tr>
<td>(6)</td>
<td>The critical time of obvious shoaling around harbor entrance (min.)</td>
<td>720</td>
<td>780</td>
<td>1080</td>
</tr>
<tr>
<td>(7)</td>
<td>The time elapsed before littoral drift started to deposit (min)</td>
<td>280</td>
<td>380</td>
<td>240</td>
</tr>
<tr>
<td>(8)</td>
<td>Time of littoral drift massively intruded into harbor entrance (min)</td>
<td>440</td>
<td>400</td>
<td>840</td>
</tr>
<tr>
<td>(9)</td>
<td>Equivalent time of (8) in prototype (min)</td>
<td>13</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>(10)</td>
<td>The depth at harbor entrance before sand transported obviously into harbor (\text{m})</td>
<td>-9.5</td>
<td>-14</td>
<td>-13</td>
</tr>
</tbody>
</table>

The function of a north groin: (See Fig. 6(c))

A 780 m long groin with its head at a depth of 5 m was added to the arrangement No. III at a location 2,400 m to the north of the north breakwater.

Test results indicated that the north groin could delay the time of massive sand intrusion into the harbor entrance for about 120 to 150 minutes in the model or 4 to 5 years in the prototype. Therefore, the groin could prolong the effective structure life about 4 or 5 years. (See Fig. 6(c))

The function of south jetties:(See Fig. 6(d))

Erosion by eddy currents in front of the south seawall took place in an area about 700 m to the south of the south breakwater. To deal with this problem, a groin field was used along the south seawall. Each groin was 200 m long at a spacing of 500 m and was placed normal to the south seawall. Test results showed that when the groins were built with energy absorbing material, the scouring at the footings of the south seawall was eliminated.

CONCLUSION AND SUGGESTION

Test results show that the predominant direction of littoral drift at the coast near Taichung Harbor is from north to south. It is apparent that a
longer north breakwater will have a larger sediment impounding capacity to the north of the breakwater. On the other hand, an angle between the inclined part and the straight part of the north breakwater should be suitably selected so that northerly waves reaching the north breakwater will be reflected in such a way that they become onshore waves to facilitate the transport of sand into the areas between north breakwater and the north groin.

Arrangement No. V is designed under the above concepts and the consideration of wave sheltering effect and the prevailing conditions of arrangement No. I and arrangement No. III. Therefore test results indicated that arrangement No. V is the optimum solution to the harbor shoaling problem. Its construction work should be carried out in a short time period, since model construction is rapidly performed, and the whole arrangement of the whole Taichung Harbor Project is planned as Fig. 7.

Based on the test results, phenomena of sediment transport and wave pattern of the harbor basin, the design sections of breakwaters, groins, seawalls and wharfs are suggested as shown in Figs. 8 (a), (b) & (c), 9, 10, 11 and 12.

ACKNOWLEDGEMENT

The authors express their appreciation to the suggestions from Dr. L.W. Tang and technical guidance from Dr. Shoji Sato. Thanks are due to Mr. J.T. Chuang and Mr. S.C. Liang who helped to carry out experiments successfully.

REFERENCES


FIG. 1 LOCATION OF TAICHUNG HARBOR

Fig. 2 Existing Taichung Harbor and Its Surrounding Area.

Fig. 3 Topography around Taichung Harbor Coast in 1941. (a) Initial Topo. of Preliminary Test Symbols

(b) Accumulated Sand Volume Per Year
Fig. 4. Four different outer-breakwater arrangements and their measured zones and points used for N-direction Wave Test.

A, B, C, D, E, F, G, H are measured zones.
1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11 are measured main points.

Fig. 5. Wind Rose of Taichung Harbor (data from 1961 to 1970)
Fig. 6(a) Arrangement No. 7.

Fig. 6(b) Water Depth Change Diagram of Main Points (Case No. 7, N15°E dir. Waves)

Fig. 6(c) The Test Results of the function of North groin.

Fig. 6(d) Erosion-Prevention Test of South Seawall.
Fig. 7 The Whole arrangement of Taichung Harbor Project
(a) Caisson Composite Section of North Breakwater

(b) Rubble Mound Section of South Breakwater

(c) Caisson Section of South Breakwater

Fig. 8 Typical Cross Section of Breakwaters
Fig. 9 Cross-Section of North Grain

Fig. 10 Cross-Section of Seawall
Fig. 11  Caisson (Solid) Type Wharf Section

Fig. 12  P.C. pile - Platform Type Wharf Section
Historical

Berlevaag is a small fishing port of some 1800 inhabitants, situated on the Norwegian coast north of the Arctic Circle, at 71° N, midway between the North Cape and the port of Vardø, not far from the Russian border. The climate is harsh, and fishing is the sole means of livelihood. For over a century, however, the bounty of the sea, hardly varying from year to year, has assured prosperity of the coastal population in this province of Finnmarken, whose shores are much exposed to winds of gale force.

There are few good harbours in this region, which also suffers a lack of natural protection in the form of reefs and islets, and The Arctic Ocean displays on occasion a savage power. Berlevaag is particularly vulnerable to storms from N to E. The only other harbour of any size in the area is Baatsfjord where the annual catch, on the average of 20,000 tons, is larger than any other landed at Norway's northern coast (see map, fig. 1).

Harbour improvement at Berlevaag started at the beginning of the 20th century with the dredging of the natural harbour basin of "Vaagen" which constitutes the port proper. In 1909, however, a large number of the fishing vessels in this harbour were destroyed by a NNE storm. As a result, the Varnes breakwater was built, and completed in 1926.

A scheme comprising the construction of two new breakwaters was proposed to extend the area of available calm water inside the harbour (fig. 3). Construction of the Svartoksen breakwater, the most westerly one, was started in 1920. The structure was of the rubble mound embankment type. The quarry was five kilometres east of the harbour and the link was provided by a railway line. Unfortunately the rock fill available was of very poor quality and therefore a larger quantity had to be quarried before blocks of the size required for the armour could be obtained. Nevertheless, work proceeded at a steady pace until in 1932 a violent north-easterly gale destroyed the structure. It was then decided to rebuild and reinforce the breakwater, retaining a rock
fill armour. By 1940, the structure was still not complete and construction work was slowed down much during the war. After being bombed and set on fire during World War II, the whole area was destroyed, including the port installations.

First it was necessary to rebuild houses, next, to put the fishing harbour back into working order. It was not until somewhere around 1950 that work on the breakwater could be started again. By 1958, the Svartoksen breakwater was within 50 metres of completion. The facing on the seaward side was built up from 15 ton concrete cubes. On the 6th of January 1959, however, another fierce gale, this time from ENE, destroyed 90 m of the seaward end of the breakwater. All the concrete blocks were washed out on to the harbour side. Although the damage was less than in 1932, the breakwater tip had to be rebuilt once again. In 1959, after scale model tests, it was decided to use tetrapods in the facing.

Brief Survey of Local Conditions

Before proceeding with a detailed examination of the problems raised by the use of tetrapods in Arctic waters, some account of the particular conditions at Berlevaag should be given.

The tidal range is relatively large, 3.60 m. for spring tides, the average range is 2.50 m. Summer is short, but on occasions temperatures may be high, temperatures of over 30°C have been recorded, not lasting more than a few days. Winter temperatures may be as low as -20°C and even -25°C, generally accompanied by strong winds. This means that the breakwaters are covered with a thick layer of ice during the winter. On the other hand the sea never freezes over at Berlevaag and the breakwaters are therefore free from damage caused by ice pressure. Sea temperatures remain above freezing, and the concrete structures affected by the tidal range are subjected to freeze and thaw twice daily. These conditions call for the use of exceptionally high quality concrete (fig.4 and 5).

Breakwater Profile Design Studies

Design studies were carried out to determine the profile to be adopted for the new structure on the Svartoksen breakwater and the Revnes breakwater from the other side. Model tests were carried out in Grenoble in the laboratory of Sogreah, (la Société Grenobloise d'Etudes et D'Applications Hydraulique's) on behalf of Sotramer Grenoble (la Société d'Exploitation de Brevets pour Travaux à la Mer). An essential condition was to re-use the existing 15-ton blocks which were stored in reserve and salvage those that had been washed away by the storm.

The final design is shown in fig. 6. The breakwater armour is built of 15-ton (6.3 cu.m.) tetrapods with rows of 15-ton
cubic blocks forming a platform for the mobil crane operating at the (+6.00) level. The mound between the (+6.00) and the (+8.20) levels consists of four 15-ton cubic blocks placed from the crest of the breakwater after completion of the main work, and just before removal of the crane (fig. 7 and 8).

This section was designed to cope with storms producing waves up to 9 metres amplitude without serious damage. For the more vulnerable breakwater head, however, more efficient protection was necessary. After further tank tests in Grenoble it was decided to use 25-ton (10 cu.m.) tetrapods on a truncated cone-shaped breakwater head, the stability of which was practically the same as that of the running section.

Other experimental studies at scale 1:110, were carried out by the Trondheim Laboratory in order to investigate methods of countering waves from the open sea and to clarify how the waves are damped inside the harbour.

The object of these tests was to determine the layout and exact length of the Revnes breakwater to be built on the eastern side and to find the best site for a quay wall for vessels up to 3,000 tons.

Up to 1974 ships of this tonnage could not berth at Berlevaag and had to anchor on the roadstead, being loaded and unloaded by lighters and barges.

The Construction Works

Owing to the arctic climate it has been impossible to maintain a very rapid rate of construction. The winter lasts until May and it is impossible to work on the site before the end of that month. Thus only some four months are available for construction work, since the equinoctial gales start at the end of September or during October. A temporary breakwater head of 15-ton tetrapods therefore had to be built early in the autumn every year in the hope that it would offer sufficient protection until the following spring.

Special care was bestowed on the concrete. In view of the risk of frost damage, especially in the area affected by the tide, the concrete used for the tetrapods contains 400 kg. of cement per cubic metre. An air-entraining agent in the form of a resinous oil was added and the mix, which was as dry as possible (water/cement ratio below 0.45) was carefully vibrated. Specimens were taken regularly for compression tests in the laboratory, and the mean compressive strength after 28 days always exceeded 300 kg./sq.cm.

The majority of the blocks were provided with a lifting hook for easier removal each spring and very rapid positioning when a storm was due. The use of these lifting hooks has, however, been kept to a minimum, since they represent a source
of corrosion. Wherever possible, slings have been used to place the tetrapods.

To obtain precise data on wave periods and amplitudes during storms, the Norwegian Port Authority has operated a wave recorder for 10 years some 400 metres out in the ocean from the main breakwaters. The pressure variations on the sea bed was relayed to the shore station by cable. In this way a continuous record was kept of the waves during storms. The maximum amplitude recorded was 9.80 m for a period of about 12 seconds, with no damage done to the breakwaters.

Works Finished during 1903-77

The west, or Svartoksen, breakwater was rebuilt and completed during the years 1960-64, after which the equipment was removed to the east side. The old railroad was abandoned, and the construction of the Revnes breakwater started in 1966 was finished 1976. On this side all transport were carried out by heavy lorries and semi-trailers.

To make the inner harbour basin calmer a small secondary breakwater was built during the winter (!) 1973-74 from Troendernes towards the Varnes mole. The most important structure for the population and the fishing industry of Berlevaag, however, was the large dock on the east side. It was constructed in the years 1973-74 and opened the 30th November 1974.

The first ship to go alongside was the coastal express m/s FINNMARKEN (2,200 DWT.). On the opening day the NE wind increased to 10 (small storm) on the Beaufort scale, but the vessel maneuvered through the 240 m. opening between the main breakwater heads and clapped to the quai. The results obtained were in agreement with the model tests.

It has been of great advantage in planning and carrying out the works that there has been continuity in the leadership, problems being managed by the same persons for the last 20 years.

Costs and Work Still to be Done

The total cost of the harbour works at Berlevaag is 71 million Norwegian Crowns (~14 mill. U.S. $) including all works from 1903 until 1977. The main harbour for greater sea going vessels is planned in the Revnes Bay, on the east side. The shallow water will be deepened by cutter suction dredger. The sand will be pumped ashore to build necessary land for the expanding fishing industry and service functions. This plan, called Berlevaag II, is estimated to cost approximately 12 mill. Crs. (~2.5 mill. U.S. $). These works have not yet been finally adopted.
Fig. 1. Sketch map of the area.

Fig. 2. The old steam crane on Svartoksen Mole.
Fig. 3
Site plan of Berlevåg Harbour, Finnmark.
Fig. 4 and 5. Ice and snow covering the tetrapod armour.
Fig. 6 Svartoksen mole extension.
Fig. 7. The model breakwater head in a wave tank.

Fig. 8. The model breakwater profile in a wave flume.
Fig. 9 Tetrapod armour in the summer.
Fig. 10. Norway's Prime Minister Odvar Nordli (the tall man to the right) visiting Berlevaag. Author left.
Fig. 11 Svartoksen Mole. Ice cover on the carapace.
INTRODUCTION

This paper is a continuation of paper presented at the Poac-73 in Reykjavik, Iceland, dealing with the concept and theory of the wave pump (2).

The wave pump utilizes the momentum of waves at or close to breaking to produce a current. Since 1973 a considerable amount of laboratory experiments have been undertaken and the first prototype is about 200 m long and has a 30 m wide, 50 m long funnel entrance narrowing down to a 10 m 1.5 m deep channel has been built, (Fig. 4).

The conclusion is that the wave pump seems to be a useful tool in generation of currents. Its function, the wave action and its time history depends upon shore geometry as well as upon the geometry of the trap itself, depth being a most important factor.

One application is for flushing of basins in low tidal range areas. Another could be for power generation in areas with remote location where nature itself has produced wave pumps or only has to be given a small hand to do it.

A third application if for flushing of ice in harbor basin where the sea outside the harbor stays open due to wave agitation while the basin with its calm waters freezes over early. There are several examples in Norway on this.

THE MECHANICS OF THE PUMP

The principle involved in the pump shown in Figs. 4, 5 and 8, is increase of mass transport by shoaling. Referring to (10) four equations governing the shoaling process may be written:

\[
\frac{c^2}{gh} = 1 + \frac{H}{h} A
\]

\[
U = 16/3 m K^2
\]

\[
L = c T
\]

\[
\frac{E_{rr}}{\rho g} = H^2 L_r B = H^2 L B
\]

where for convenience the following definitions

\[
A = A(m) = 2/m - 1 - 3E/(mK)
\]
\[ U = U(m) = H \frac{L^2}{h^3} \]  \hspace{1cm} (6)

\[ B = B(m) = \frac{1}{m^2} \left[ \frac{1}{3} (3m^2 - 5m + 2 + (4m - 2) \frac{E}{K}) - (1 - m \frac{E}{K})^2 \right] \]  \hspace{1cm} (7)

were introduced. \( K \) is a complete elliptical integral of first kind. \( E = c\sigma g \int \sigma^2 dt \), \( m \) is an elliptic parameter in a Jacobian function. Ref. (10) has diagrams showing the variations in \( A \) and \( B \).

The above expressions may be combined in one transcendental equation in \( m \) by eliminating the other three unknowns, \( H \), \( L \) and \( C \), all values corresponding to water depth \( h \). This equation is called "the shoaling equation" (10) which may be solved numerically on a digital computer. This gives a solution to all unknown from the eqs. above. See also refs. (1) and (8).

In the case, where the initial wave is specified at a depth so great that sinusoidal wave theory must be applied instead of cnoidal, the energy transport

\[ E_{tr} = \rho g \int \sigma^2 dt \]

where

\[ B = \frac{1}{16} \left( 1 + \frac{2k h}{\sinh 2k h} \right) \]  \hspace{1cm} (9)

and \( k = \frac{2\pi}{L} \) is the wave number.

This result is of particular interest because it makes it possible at any water depth to compare the results of the two theories for a wave which is characterised by data at any other water depth including deep water.

Fig. 1 (10) shows the variation of \( B \) according to Eq. (9). Since the abscissa \( U \) does not specify the \( kh \) parameter uniquely the value of \( B \) in this plot depends on another parameter too, which is for convenience chosen as the deep water steepness \( H_0/L_0 \). Comparison between the curves representing Eqs. (7) (cnoidal theory) and (9) (sinusoidal theory) shows that the amount of energy which according to the two theories is transported by a wave of a certain height and length varies in a very different way. Thus we can expect considerable differences in the wave heights and wave lengths predicted for the same period and energy transport.

Ref. (10) explains how it is possible to determine wave height \( H \) by eqs. (4) and (6).

As mentioned in ref. (10) it can be shown that for large values of \( U \)

\[ H/H_0 = \text{const} \ h^{-1} \quad (U \to \infty) \]  \hspace{1cm} (10)

This variation is illustrated by the dotted curve in Fig. 1. It shows that the wave height in very shallow water increases considerably faster with decreasing \( h \) than the variation as \( h^{-1/4} \) predicted by the classical linear long wave theory. For large values of \( U \) each of the crests in the cnoidal wave train more and more resembles a solitary wave in profile. Thus for large \( U \) the wave will show all the features of the solitary wave. It is therefore relevant to recall that the same result was obtained directly for a solitary wave by Grimshaw (5).
Another of the solitary wave characteristics can be demonstrated if we consider the energy transport. Substitute $|B+2/3K|$ for $B$ and $3/16H^2/h^3$ for $K$ (from Eqs. (2) and (6)) into Eq. (4). We then get

$$E_{tr}/\rho g = \frac{2}{3} H^{3/2} h^{3/2}$$

which shows that the energy transport does not depend on the wave length $L$.

Although in most practical cases this limit corresponds to $H/h$ values which are far in excess of any breaking point the tendency that the wave height grows faster than $h^{-1/4}$ is felt even at moderate values of $U$.

Using Shuto's theory for shoaling of cnoidal waves (9) with special reference to situations with a big Ursell parameter, $U_p = gHT^2/d^2$ one has for values of $U_p > 100$

$$Hbd^{5/2} |\sqrt{gHT^2/d^2} - 2\sqrt{3}| = \text{constant}$$

which for big $U_p$'s may be written:

$$Hdb^{2/3} = \text{constant}$$

which is the law of shoaling for solitary waves as mentioned above.

Shuto in (9) gives the following "Laws of Shoalings" which have been applied for analyses of the tests mentioned below, (Table 1).

<table>
<thead>
<tr>
<th>$gHT^2/d^2$</th>
<th>Law of shoaling</th>
</tr>
</thead>
<tbody>
<tr>
<td>$2&gt;$</td>
<td>$Hd^{1/4}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$2$- $6$</td>
<td>$Hd^{1/6}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$6$- $15$</td>
<td>$Hd^{1/7}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$15$- $25$</td>
<td>$Hd^{1/8}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$25$- $30$</td>
<td>$Hd^{1/5}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$30$- $40$</td>
<td>$Hd^{2/7}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$40$- $65$</td>
<td>$Hd^{1/3}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$65$-$100$</td>
<td>$Hd^{1/2}b^{1/2} = \text{const.}$</td>
</tr>
<tr>
<td>$100&lt;$</td>
<td>$Hd^{5/2}b(\sqrt{gHT^2/d^2} - 2\sqrt{3}) = \text{const.}$</td>
</tr>
</tbody>
</table>

Camfield and Street (4) conducted experiments to determine the various effects on the shoaling, breaking and run-up of solitary waves resulting from the bottom configuration. An initial set of experiments investigated the effect of the initial bottom slope on the breaking and run-up of a wave on a second, higher slope. A second set of experiments considered the effect of a continental shelf configuration on the transmissibility of waves in the shoreward direction, and the decomposition of the waves due to the shallower water depth on the continental shelf. It was found that, in order to make predictions at or near the shoreline for waves generated in deep water, it is necessary to consider the total configuration of the bottom leading to the shoreline.
Results are shown in Fig. 2 with the following notations:

\begin{align*}
D_T & \text{ the "deep water" depth at the toe of the beach slope} \\
D_S & \text{ the water depth on the horizontal beach shelf} \\
\alpha & \text{ the angle of inclination of the beach slope (Fig. 3)} \\
H & \text{ the solitary wave height}
\end{align*}

Accordingly, the test parameters were:

\[
H/D_T = \text{relative deepwater wave height} \\
D_T/D_S = \text{water depth ratio} \\
K_T = \frac{V_S}{V} \text{ where } V_S \text{ is the wave volume on the shelf} \\
V = \int_0^\varphi \eta \, dx = \left(\frac{16}{3} D^3 H\right)^{1/2} \text{ the volume of water above the still water line in the wave}
\]

It may be seen that the volume of water in the solitary wave crest when passing over the shelf increases with the volume of water in the crest in deep water. The \(K_T = V_S/V\) increases with decreasing \(D_T/D_S\) and with increasing \(V/D_T^2\). This in turn is in qualitative agreement with the result of tests mentioned later.

Practical tests on the Wave Pump carried out at the National Electricity Laboratory in Reykjavik, Iceland, are described in report by the laboratory to the sponsor of the tests, the PALMAS DEL MAR company, Puerto Rico, and in ref. (3).

All tests were run with depth 1.5 m and wave heights varying from 0.3 to 0.9 m.

Attempts were made to explain the experimental results of the tests by cnoidal as well as solitary wave theories. Due to the fact that the Ursell-parameter \(U_p = gHT^2/d^2\) was 50 to 100 a solitary wave theory proved most suitable (9). The wave height at the entrance to the straight channel was used for computation of the volume-transport. Energy loss by friction was minor as proved by field tests (3). Turbulence, however, could in some cases cause considerable loss of energy (3).

The volume-transport velocity averaged from surface to bottom was then given by:

\[
V = 4H^2/T \sqrt{\nu/3} \quad \gamma = H/h \quad (14)
\]

Volumes for a 5 ft (1.5 m) deep, 33 ft (10 m) width channel are shown in Table 2.

| Table 2 Volume Transport Velocities in Solitary Wave (3) |
|------------------------|-----------------|-----------------|-----------------|-----------------|
| \(H_t\) m | \(T\) sec | \(V\) m/sec | \(Q\) m³/sec/10 m |
| 0.36 | 6 | 0.28 | 4.2 |
| 0.46 | 8 | 0.24 | 3.6 |
| 0.63 | 8 | 0.28 | 4.2 |
| 0.41 | 10 | 0.18 | 2.7 |
| 0.68 | 10 | 0.24 | 3.6 |

It may be noted that discharge decreases with period and increases relatively little with wave height.
The question of pile up by wave breaking in front of the funnel is discussed in ref. (3) and it was concluded that this could be of considerable importance particularly in the field if the shore geometry contribute further to the pile up.

Table 3 is a review of data on discharges for various wave data by theory and combined model and theory, experiments and computations (3). It also includes the result of a single test in the prototype, Fig. 4, PALMAS DEL MAR, Puerto Rico, East Coast which includes a 120 m channel and a 50 by 30 m narrowing to 10 m funnel and a 10 m channel.

From Table 3 it may be seen that theoretical laboratory and combined solitary wave computations generally have the same order of magnitude apart from an exceptional deviation for 8 sec waves of 0.63 m and for 0.68 m waves of 10 sec when the sole laboratory discharge data are highest.

In the field wind pile up must have been negligible due to a weak wind and a very narrow continental shelf. In the laboratory it did not exist apart from some tests which were run with wind (3) and unpublished report by the Icelandic Laboratory. They showed a slight increase in discharge but the model law applied was questionable and require further verification.

<table>
<thead>
<tr>
<th>H m</th>
<th>T sec</th>
<th>Theory Airy (friction)</th>
<th>Laboratory experiments</th>
<th>Solitary on lab. data</th>
<th>Field</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.36</td>
<td>6</td>
<td>4.9</td>
<td>3.1</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.46</td>
<td>8</td>
<td>3.7</td>
<td>3.3</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.63</td>
<td>8</td>
<td>5.0</td>
<td>8.0</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.41</td>
<td>10</td>
<td>2.9</td>
<td>2.8</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.68</td>
<td>10</td>
<td>4.6</td>
<td>5.5</td>
<td>3.6</td>
<td>~8</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>6/8</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The conclusion of the tests described above was summarized as follows:

The "Wave Pump" seems to be a useful tool in generation of currents. Its function depends upon the shore or other geometry outside its funnel entrance and of the funnel shape.

The optimal funnel shape depends upon the direction of wave propagation and its deviations in either direction from the centerline of the funnel. Reflection of any
kind should be avoided. Cross wave action, however, occur for certain periods and direction of wave propagation. This causes energy loss and should be looked into further by experiments.

The H/h ratio is very important for the performance of the trap. Discharge seems to increase with increasing H/h until a certain limit. Discharge seems also to decrease with increasing wave period for the same wave height. In the case of small waves (H/h < about 1/3) raising of the ramp will increase the discharge somewhat but the ramp obviously should not be raised so high that waves break on it as plungers because this will cause an increased loss of energy by turbulence, thereby decreasing the discharge.

The influence of wind pile up on the performance of the trap is probably relatively small. The wave set up, however, may have considerable influence and this may be taken into consideration in the location of the entrance of the pump.

Improvements of the trap are possible e.g. by streamlining and by a partition wall in the middle of the funnel. This is subject to further study in relation to funnel geometry, channel geometry, wave characteristics, etc. So is the development of a fully rational theory combining waves and currents.

TEST UNDERTAKEN IN 1976 - 1977

Similar tests assuming a scale of about 1:40 and waves of 2 to 4 meters were undertaken by Kjelstrup using a different test set up which is shown in longitudinal section and plan view in Fig. 5 (ref. 7).

For the conditions in the laboratory experiments described below the \( U_p \) had the following approximate characteristic values in a number of tests, (Table 4).

<table>
<thead>
<tr>
<th>d (m)</th>
<th>T (sec)</th>
<th>H (m)</th>
<th>( U_p = \frac{gHT^2}{d^2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>4</td>
<td>1.2</td>
<td>20.9</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>1.6</td>
<td>43.6</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>1.0</td>
<td>39.2</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>1.1</td>
<td>58.8</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>1.5</td>
<td>132.4</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>1.6</td>
<td>174.4</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>1.8</td>
<td>237.4</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>1.7</td>
<td>10.7</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>1.6</td>
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<tr>
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<td>6</td>
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<td>24.0</td>
</tr>
<tr>
<td>5</td>
<td>7</td>
<td>2.6</td>
<td>50.0</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>2.5</td>
<td>62.8</td>
</tr>
<tr>
<td>5</td>
<td>9</td>
<td>3.2</td>
<td>101.7</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>3.8</td>
<td>149.1</td>
</tr>
<tr>
<td>5</td>
<td>11</td>
<td>3.4</td>
<td>161.4</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>3.0</td>
<td>169.5</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>2.4</td>
<td>16.4</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>2.0</td>
<td>19.6</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
<td>2.0</td>
<td>26.7</td>
</tr>
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<td>6</td>
<td>8</td>
<td>2.4</td>
<td>41.9</td>
</tr>
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<td>2.0</td>
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</tr>
<tr>
<td>6</td>
<td>10</td>
<td>3.1</td>
<td>84.5</td>
</tr>
<tr>
<td>6</td>
<td>11</td>
<td>2.7</td>
<td>89.0</td>
</tr>
</tbody>
</table>

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The U-figures show that tests in half of the cases are close to the solitary wave range ($U > 100$). In the remaining tests ($U < 100$) tests are in "cnoidal ranges".

Kjelstrup's introductory tests (Fig. 5) induced a wave damping basin, a weir and a box for measurement of the discharge. This system allowed a recording of the water level in the damping basin and thereby a registration of the influence of wave period and wave steepness on pile up. Results are shown in Figs. 6-7 for prototype water depth = 5 m periods varying from 2 to 9 sec and steepness $H_o/L_o$ varying from 0.02 to 0.09. The ratio between the wave height at the entrance $H_e$ and at the inner end of the 60 m long funnel $H_{60}$ is shown in Fig. 7. The rise in water table took place solely in the basin (see later). From Fig. 6 it is apparent that the rise in water table was most predominant for waves of 6 to 7 sec and for steepness ratios of 0.05 - 0.07 corresponding to medium - not extreme - storm waves. This is qualitative agreement with the results of previous tests as very steep waves entering the funnel will lose energy in the funnel by wave breaking and longer waves simply produce less flux of wave energy per unit time than waves of shorter period. These results became more apparent by additional tests using an open channel model as shown in Fig. 8 similar to the previous test procedure. Results for water depth 5 m are shown in Tables. The entrance energy $E_e$ and the energy flux $E_f$ is computed the latter using Eq. 8. The efficiency of the pump may be defined as $Q/E_e$ (dimensionless) and by the ratio $E_f/E_e$.

Jonsson in ref. (6) and earlier publications by him, gives the following expression for the energy flux:

$$E_f,\text{MWL} = E_{sp} c_g + E_{sp} S + E_{sp} \left(\frac{2c_g}{c} - \frac{1}{2}\right)S + \frac{1}{2} \rho D S^3$$

It is called to the attention, however, that the reference level for potential energy in Eq. 15 is the mean water level, which in general is not horizontal.

In the present case, as described above, the mean water level was (almost) constant, the energy level was not entirely horizontal because of the energy dissipation due to bottom friction and turbulence. Bottom friction, however, was minor but some loss by turbulence occurred. Choosing the horizontal bottom as reference level, we find from Eq. 15 a preliminary approximation

$$E_f = E_{sp} c_g + E_{sp} S + E_{sp} \left(\frac{2c_g}{c} - \frac{1}{2}\right)S + \frac{1}{2} \rho D S^3 + \gamma D^2 S$$

Jonsson rewrites as follows:

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\[ E_f = (\frac{1}{2} \gamma D^2 + F_{wp})S + E_{sp} \frac{c g}{c} + (\frac{1}{2} \gamma D^2 + \frac{1}{2} \rho DS^2 + E_{sp} + F_{wm})S \] (17)

with

\[ F_{wp} + F_{wm} = F_w \] (18)

and

\[ F_{wp} = E_{sp} \frac{c g}{c} - \frac{1}{2} \]

\[ F_{wm} = E_{sp} \frac{c g}{c} \] (20)

As explained by Jonsson, the first two terms in Eq. 17 in fact represent the mean work done (per sec) by the pressure forces on an "impermeable membrane" moving with velocity \( S+U \), and the third term is the mean increase in total energy over mean length \( S \).

Eq. 16 becomes, in a mathematically simpler form

\[ E_f = E_{sp} \frac{c g}{c} + E_{sp} \left( \frac{2c g}{c} + \frac{1}{2} \right)S + \frac{1}{2} \rho DS^3 + \gamma D^2 S \] (21)

Table 5 shows an example of computations for depth = 5 m. The last column describes the wave condition in the entrance - that means whether waves are breaking or not. See Figs. 9 and 10. Results are plotted in Figs. 11, 12 and 13 for each separate wave period.

**SUMMARY OF RESULTS OF TEST AND CONCLUSION**

The data material available is still scarce to draw very definite quantitative conclusions.

From Table 5 and diagrams Fig. 11 (3 m), Fig. 12 (5 m) and Fig. 13 (6 m), it is, however, possible to draw the following general qualitative conclusions:

1) The wave condition in the pump is very important for its efficiency. Maximum capacity is always obtained for waves at or close to breaking but breaking must not be of a character which causes high energy-loss e.g. plunging. Spilling or surging are preferable.

2) Particularly for depths of 5 m and 6 m, it is obvious that the highest efficiency is obtained for relatively small wave period when waves break in the funnel. This produces maximum momentum over a relatively wide area. For longer periods it appears to be an advantage that waves break in the channel itself (not in the funnel). Too much momentum is lost by relatively infrequent breaking in the funnel. Losses by breaking in the channel are smaller.

3) There is apparently an optimal wave period for which all other conditions being equal - it is possible to obtain maximum efficiency of the pump.

4) The importance of depth in the funnel solely lies in its relation to wave
characteristics through wave breaking and its character.

5) The importance of the $U_p$-factor is that for a given $T$ maximum efficiency of the pump is obtained for the lower $U_p$ values. For the larger $T$'s highest efficiency for the same depth is obtained for the largest $U_p$'s.

6) The importance of adjustments in depth e.g. by means of a ramp, is demonstrated by item No. 1. But the ramp must neither cause reflection nor plunging breaking.

7) As it is important to obtain wave breaking or conditions close to breaking, an optimization is necessary before final design considering the available wave and tide conditions with special reference to water depths (tidal ranges included) and wave periods.

8) Funnel geometry should never cause reflection and energy losses by friction should be as small as possible. It is selfexplanatory that sediment transport to the funnel should be avoided.

9) More detailed research on the influence of the breaking condition in relation to $U_p$, $Q$ and $T$ is needed.

10) Regarding funnel layout and geometry reference is also made to refs. (2) and (3). Edging of the waves without or with a minimum of reflection is desirable.

11) Properly designed and operated the "Wave Pump" should be a very useful tool for flushing of basins whether they are polluted by normal pollutants or by ice. With respect to the former reference is made to the PALMAS DEL MAR marina on Puerto Rico (Fig. 4). The latter is demonstrated by nature in many examples in the Northern part of Norway and "artificial cases" may be considered to assist in keeping the harbor or other enclosed area as ice-free as possible partly by

(a) hindering the formation of ice on the surface by a current. This is of particular importance where a fresh-water sheet or water of lower salinity may rest on the top of layers of higher density. The danger of this, however, is least during the winter season but many harbors have outlets for sewage of any kind including industrial waters and this presents a danger to the formation of sheet ice in the top-layers

(b) moving ice already formed in the harbor out in the ocean

(c) moving ice which penetrated during high tides back in the ocean again thereby assisting the ebb currents

12) There is needless to say, also a chance that the pump may be useful to generate electrical power under suitable circumstances. It may be most practically to utilize the fluctuations in pressure due to the concentrated wave motion.
<table>
<thead>
<tr>
<th>T (sec)</th>
<th>D (m)</th>
<th>$H_e$ (m)</th>
<th>$H_e/D$</th>
<th>$C_g$ (m/sec)</th>
<th>$E_e$ (N*m/m²)</th>
<th>$V_m$ (m/sec)</th>
<th>$E_f$ (Nm²/m²sec)</th>
<th>$Q$ (M*L)</th>
<th>$Q<em>10^3$ (M</em>L/T²L)</th>
<th>$E_f$ (E_e waves)</th>
<th>Condition of waves</th>
<th>$U_p$</th>
</tr>
</thead>
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<tr>
<td>4</td>
<td>5</td>
<td>1.76</td>
<td>0.35</td>
<td>3.70</td>
<td>3872</td>
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<td>208213</td>
<td>3.75</td>
<td>0.96</td>
<td>53.8</td>
<td>Surge overall</td>
<td>11.0</td>
</tr>
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<td>4</td>
<td>5</td>
<td>1.76</td>
<td>0.35</td>
<td>3.70</td>
<td>3872</td>
<td>1.06</td>
<td>289840</td>
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<td>1.37</td>
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<td>11.0</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>1.52</td>
<td>0.30</td>
<td>3.70</td>
<td>2888</td>
<td>1.12</td>
<td>300137</td>
<td>5.6</td>
<td>1.94</td>
<td>103.9</td>
<td>Breaking in funnel</td>
<td>9.5</td>
</tr>
<tr>
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<td>5</td>
<td>1.6</td>
<td>0.32</td>
<td>4.64</td>
<td>3200</td>
<td>1.44</td>
<td>391667</td>
<td>7.2</td>
<td>2.25</td>
<td>122.4</td>
<td>Surge in channel</td>
<td>15.7</td>
</tr>
<tr>
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<td>5</td>
<td>1.6</td>
<td>0.32</td>
<td>4.64</td>
<td>3200</td>
<td>1.03</td>
<td>281771</td>
<td>5.1</td>
<td>1.59</td>
<td>88.1</td>
<td>Breaking outside entrance</td>
<td>15.7</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>1.8</td>
<td>0.36</td>
<td>4.64</td>
<td>4050</td>
<td>0.84</td>
<td>237180</td>
<td>4.2</td>
<td>1.04</td>
<td>58.6</td>
<td>Breaking in funnel</td>
<td>17.6</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>1.76</td>
<td>0.35</td>
<td>5.26</td>
<td>3872</td>
<td>0.75</td>
<td>215189</td>
<td>3.75</td>
<td>0.97</td>
<td>55.6</td>
<td>Surge overall</td>
<td>24.9</td>
</tr>
</tbody>
</table>
REFERENCES


4) Camfield, F.E. and Street, R.L., 1969, "The Effects of Bottom Configuration on The Deformation, Breaking and Run-up of Solitary Waves", Proc. 11th Conf. on Coastal Engineering, Chapter 11, Printed by the ASCE.


Fig. 1. B versus U

Fig. 2. $K_T$ versus $V/D_T^2$
Fig. 3. The Continental Shelf Configuration
Fig. 4. PALMAS DEL MAR, Puerto Rico with Wave Pump installed
Fig. 5. Tests using a wave damping basin

Fig. 6. Rise in water table in damping basin (mm) versus wave steepness as indicated and period $T$
Fig. 7. $\frac{H_{60}}{H_e}$

Fig. 8. Tests using an open discharge channel
Fig. 9a, b. Wave passing channel without breaking
Fig. 10, a,b. Wave breaking in the funnel (a) and in the channel (b)
Symboles:

- T = 3 sec.
- 4
- 5
- 6
- 7
- 8
- 9
- 10
- 11

Fig. 11. Results relating \( \frac{E_f}{E_e} \) to \( \frac{Q \cdot 10^3}{E_e} \) for \( D = 3 \) m.

- \( E_f \) and \( E_e \) are the effective and equilibrium electric fields, respectively.
- \( Q \) is the electric charge.
Fig. 12. Results relating $\frac{E_f}{E_e}$ to $Q \cdot 10^3$ for $D = 5$ m.
SYMBOLS:

- O  T = 4 sec.
- △  5
- X  6
- T  7
- □  8
- Ø  9
- A  10
- N  11

Fig. 13. Results relating $\frac{E_f}{E_e}$ to $\frac{Q \cdot 10^3}{E_e}$ for $D = 6$ m.

FIG. 13
ABSTRACT

During a hurricane or very large storm conditions, massive soil movements have been found to occur. This type of phenomenon occurs typically in areas of rapid sedimentation and may cause large lateral loads on pile foundations which are subject to sea floor movements. A model investigation of this phenomenon was performed and the effects of varying the associated parameters involved during the testing program were evaluated. The magnitude of the experimental loading force is compared to an established prediction method. The data obtained from this investigation indicates that there is a slight increase in the soil loading force over that predicted by state of the art analysis. The results of this investigation will allow engineers to predict the magnitudes of the lateral loads imposed on pile foundations with more confidence.

INTRODUCTION

Lateral loading on pile foundations has been an area of extensive research by the geotechnical community for many years. This research has concentrated on the lateral loading of pile foundations at or near the ground surface of the soil-pile system. For the design of foundations for offshore structures, the lateral loading of pile foundations are typically a resultant of wind and wave loadings. In specific offshore areas where weak seafloor sediments exist, slope stability failures may cause large lateral loads in addition to the wind and wave loadings (Henkel, 1970). These slope failures have occurred in many locations and are described in papers by Bjerrum (1971) and Terzaghi (1956). Among the many possible causes for these submarine slope stability failures are rapid deposition of weak sediments on steep slopes (Bea, 1971), large wave occurrence (Joyce, 1973), seismic loading, erosion, and unusual currents.

In the Gulf of Mexico, off the Louisiana coast, the Mississippi River discharges sediments in the near shore zone at such a rapid rate with respect to the permeability of the sediments, that the surface sediments of the seafloor are unconsolidated. This underconsolidated layer can exist to very great depths where the weak cohesive shear strengths may range between 5000 Pa and 10000 Pa.

The Gulf of Mexico is subject to very large storms such as hurricanes during the summer months. Associated with these large storms are severe wave conditions which can greatly affect the seafloor bottom (Henkel, 1970). Using Stoke's 5th order waves,
a maximum bottom differential pressure between 5000 Pa to 50000 Pa may develop from these waves (Bea, 1971). When these large bottom differential pressures occur in combination with the weak seafloor sediments and the gravity force due to a sloping bottom, a pressure couple can be created which could exceed the maximum resistance the soil can develop, thereby initiating a seafloor slope failure, as shown in Figure 1. If a pile foundation is located within the failure zone, a lateral load in addition to those of the wind and wave loadings will occur due to the relative horizontal downslope movements of the sediments.

BOTTOM STABILITY

The problem of designing an offshore structure to withstand the forces imposed on the pile foundations supporting it gained significance after Hurricane Camille passed through the Gulf of Mexico in 1969. During this hurricane, the Shell Oil South Pass 70 "B" and the Gulf Oil South Pass 61 platforms were destroyed. The results of the investigation conducted by Sterling and Strohbeck (1973) after the passing of Hurricane Camille showed that the structures failed primarily because of massive sea floor movements.

Prediction of these slope failures is difficult, however several techniques have been developed. Henkel (1970) developed a limiting equilibrium procedure for a sinusoidal wave loading which concluded that if the water depth is comparable to the wave height, the bottom pressures exerted by the surface waves may be great enough to induce underwater landslides. Wright and Dunham (1972) developed a finite element procedure to predict the state of stress and deformation in the sea floor sediments under wave induced loadings. Their results show that a failure may occur to depths of 50 meters below the mudline during a wave pressure loading. Arnold (1973) and Bea and Arnold (1971) contributed to the application of the finite element modeling of the sea floor slide loading of a pile foundation. Doyle (1973) performed large wave tank studies to trace the movements of clay sediments subjected to differential pressure loading. His studies showed that a lateral translation of soil sediments will occur when the sediments rest on a sloping sea floor and are subjected to differential pressures.

PILE LOADING

The lateral loading of a pile foundation is due to the relative movement of the soil-pile system. The movements can be of two types, that of the pile moving with respect to the soil or that of the soil moving with respect to the pile. It is the first condition that has received the vast majority of study.

The movement of the pile with respect to the soil is typically due to an external loading function, such as wind and wave loadings. The ultimate loading that can be sustained by a pile due to external loading has been researched by many engineers. The conventional formula for the lateral soil resistance force is:

\[ P = N_p \cdot c \cdot d \]  

\[ \text{.............(1)} \]

where:

- \( P \) = ultimate lateral soil resistance per unit length of pile
- \( N_p \) = dimensionless lateral resistance coefficient
- \( c \) = undrained soil shear strength
- \( d \) = pile diameter or width

Values of \( N_p \) have been found to vary with depth due to different failure conditions.
Reese (1958) stated that $N_p$ may vary linearly up to a maximum of 12 with depth. Matlock (1970) developed the following equation for the variation of $N_p$ with depth:

$$N_p = 3 + \frac{\sigma_x}{c} + 0.5 \frac{x}{d} \quad \cdots \cdots \cdots \cdots (2)$$

where:
- $\sigma_x$ = effective overburden pressure at depth $x$
- $x$ = depth from mudline at which $N_p$ is required
- $c$ = average undrained cohesion of the soil within depth $x$
- $d$ = pile diameter

The factor $N_p$ has been found to have a limiting value at a particular depth. Matlock (1970), through correlations with field and laboratory data, determined a limiting $N_p$ value of 9 occurring at a depth, $x_r$, which can be determined by equation 2.

The limiting value of $N_p$, reported in the literature, varies between eight and twelve below this particular depth, and the following values have been recommended:

<table>
<thead>
<tr>
<th>Investigator</th>
<th>$N_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhoff (1951)</td>
<td>11.42</td>
</tr>
<tr>
<td>McClelland and Focht (1958)</td>
<td>11</td>
</tr>
<tr>
<td>Reese (1958)</td>
<td>12</td>
</tr>
<tr>
<td>Matlock (1970)</td>
<td>9</td>
</tr>
</tbody>
</table>

Broms (1964) calculated the lateral resistance coefficient as a function of the shape of the cross-sectional area and roughness of the pile surface with values ranging between 8.28 and 12.56. Shown in Figure 2 is the distribution of lateral earth pressures typically considered in laterally loaded piles with free heads. The soil on the opposite side of the loaded surface will be pushed upwards and away from the pile.

The second movement condition, that of the soil moving with respect to the pile, has received less attention. The most intensive research, to date, on this topic was performed by Marti (1976). Marti considered the loading on a cylindrical surface to be a resultant of the drag forces imposed on the cylinder during the flow of the soil past the pile. A dimensionless drag parameter analogous to the lateral coefficient was determined using slip-line theory. A rough contact between the soil and the pile surface was assumed. The slip-line model considered is similar to the classic bearing capacity theory by Terzaghi (1966). The value of the dimensionless drag coefficient, which Marti found most consistent with the results of his model tests, was 10.1. This value is valid if the undrained vane shear strength includes the effect of velocity through an equivalent strain rate and the influence of confining pressure. In his work, Marti employed a shear box to move the soil past various diameter piles of 1.27 cm, 2.54 cm and 3.81 cm which were simultaneously being moved inside the shear box. Bending moments on the pile were recorded during the test in order to obtain the forces being imposed on the pile.

**EXPERIMENTAL PROGRAM**

The forces developed on a pile foundation during a sediment instability were determined using a laboratory investigation which simulated the soil failure. The tests were conducted using equipment arrangement shown in Figure 3. To fail the soil past
the pile foundation, an overburden air surcharge was placed on the upstream side of
the pile. The surcharge pressure was increased by adjusting an in-line pressure
regulator until a sufficient quantity of soil was moved against the pile to create
an ultimate loading condition. A baffle was placed in the tank so that the depth
of failure could be modified as desired. A horizontal extension was attached to
the baffle so that when the air bag was inflated it would only load the exposed
soil surface. The effect this had was to maintain the soil movement in a horizon­
tal plane during failure.

Two model piles of 2.39 cm and 4.83 cm diameters were used in the experiment. The
piles were marked at 5 cm intervals over their entire length so that consistent
embedment depth could be achieved. The markings also allowed for the measurement
of any heave that the soil would undergo. The forces developed on the pile were
measured using two proving ring transducers. Both of the proving rings were strain
gaged with a four arm bridge thereby eliminating the need for temperature compensa­
tion. The proving rings were calibrated using an unconfined compression apparatus.

The soil used during the investigation was a processed kaolinitic clay. The specif­
ic gravity of the clay was 2.58. The clay has a liquid limit of 48.1% and a plastic
limit of 25.9% yielding a plasticity index of 22.2%. The clay is classified as a
CL soil (inorganic clay of low plasticity) according to the Unified Soil Classifi­
cation System. The clay was mixed to a water content of 60.0%, yielding a liquidity
index of 1.5. The degree of saturation of the soil during the experiments ranged
between 95% and 97%. The shear strength of the soil could be measured at various
depths in the test tank using a vane with a height and diameter of 2.54 cm. The
vane was used with a constant strain Wykeham-Farrance Vane Shear Device.

The procedure performed during the testing sequence consisted of mixing the soil
thoroughly in the tank in order to obtain a uniform shear strength profile with
depth. Vane shear strength measurements were taken at 5 cm depth intervals in the
vicinity of the pile. The instrumentation was connected to a six channel Sanborn
strip chart recorder and recalibrated. The baffle, air bag and cover were placed
and the soil surface downstream of the pile was leveled to smooth the displaced
soil due to the placement of the baffle. The pile was embedded to its desired
depth, with its location being varied in different tests at 15 cm., 20 cm., and 25
cm. distances downstream of the baffle. The proving rings were placed and adjusted
to just contact the pile surface. The air pressure regulator and air pressure
transducer were then attached to the air bag valve extensions on the cover. The air
pressure was then increased to the point when failure of the downstream soil mass
was evident. The above procedure was followed for each test performed during the
investigation.

DISCUSSION

During the first phase of testing, the baffle embedment depth, pile to baffle dis­
tance and pile diameter were varied. Three different baffle depths of 2.5 cm., 10
cm., and 19 cm. were studied. The two shorter baffle depths created failure zones
which were shallower than the embedment depth of the test pile therefore making the
analysis of the pile loading difficult, because the depth of failure was not precise­
ly known. The larger baffle depth was observed to create a failure zone which
extended almost the entire length of the test box thereby creating a failure depth
that extended below the pile embedment depth.

Three different pile to baffle distances of 15 cm., 20 cm., and 25 cm. were studied.
The various pile-baffle distances made it possible to determine if the pile location with respect to the baffle was critical. The results of this portion of the program showed that the variation of the averaged $N_p$ value with the pile-baffle distance for a pile embedment length to diameter ($\frac{L}{D}$) ratio of 5 was very small. For all of the remaining tests, the pile was placed at the median pile-baffle distance of 20 cm.

Two different pile diameters of 2.39 cm. and 4.83 cm. were used to investigate the scaling effects of the model pile sizes. For a constant $\frac{L}{D}$ ratio, using the two different piles, the corresponding $N_p$ values were determined, and the variation of the averaged $N_p$ value with the different pile diameters was also very small.

The model used to determine the value of the lateral loading coefficient, $N_p$, was developed on the assumptions of a rigid pile which was free to rotate and translate in the soil as the soil moved against the pile. Figure 4a shows the pile placed in the soil in an unloaded condition. During the testing procedure, the pile rotated counterclockwise, as shown in Figure 4b. During this loading sequence four forces were considered to be acting on the pile. These were:

$P_1 = \text{Rotational reaction load measured with a gaged proving ring.}$

$P_2 = \text{Lateral reaction load measured with a gaged proving ring.}$

$P_3 = \text{Resisting load developed by the soil.}$

$R = \text{Soil loading force.}$

The vertical location of $P_1$ and $P_2$ was measured prior to the testing, and the magnitudes of $P_1$ and $P_2$ were measured on the strip chart recorder. The rotation of the pile created a void behind the pile to a certain depth, as shown in Figure 4b: and the depth of the void was measured during the testing sequence. Therefore, $P_3$ was assumed to be the fully developed average ultimate lateral soil resistance below this depth on the backside of the pile with $P_3$ being assumed halfway down the remaining soil contact length (1). The magnitude of $P_3$ was determined using Matlock's criteria of :

$$P = N_{p3} c d_1 \quad \text{.............(3)}$$

where $N_{p3}$ is calculated from equation 2 at the midpoint of soil contact length at the back of the pile. The value of $R$ was determined by simple static analysis using a summation of the forces acting on the pile. From this value of $R$, an experimental lateral loading coefficient $N_p$ was determined.

The values of $N_p$ obtained during the testing program were for a shallow failure condition. A comparative value of $N_p$ was obtained for each test based upon Matlock's criteria. This comparative value of $N_p$ was determined by using equation (2). A comparison of the experimental value of $N_p$ and the Matlock value of $N_p$ is shown in Table 1. As the $\frac{L}{D}$ ratio was increased, the corresponding $N_p$ value also increased. Plotting the average $N_p$ values against the corresponding $\frac{L}{D}$ value, as shown in Figure 5, the experimental value of $N_p$, is consistently greater than the Matlock value of $N_p$. Table 2 shows the percentage increase of the experimental lateral loading coefficient $N_p$ over the Matlock value of $N_p$. The range of the differences varies between 6.8% and 16.0% with an average increase of 10.9% over the predicted value of the lateral loading coefficient. Marti (1976) in his work determined that there was a 12.2% increase in the lateral loading coefficient over the predicted value of 9 for an ultimate soil failure condition. The results of this investigation and Marti's investigation are in close agreement.
CONCLUSIONS

The information presented in this investigation is of a preliminary nature, however, it can be concluded that: the failure condition that existed during the testing program was of a shallow failure type. The lateral loading coefficient $N_l$ increased linearly with the depth of the failure and the magnitude of the lateral loading coefficient $N_p$ due to a soil instability loading was, on the average, 11% larger than that predicted using Matlock's criteria. For offshore pile foundations whose design includes a low factor of safety, this increased lateral loading caused by submarine slope stability failure may place the offshore pile foundation into a critical loading condition.

ACKNOWLEDGEMENT

This investigation was jointly supported by Fugro Gulf, Inc. (Houston, Texas) and the Department of Civil Engineering, University of Houston (Houston, Texas).

REFERENCES


Surface Wave

Mean Water Level

Differential Pressure Amplitude

Hydrostatic Pressure of Mean Level

Pressure Couple

Shear Resistance

Down-Slope Weight

Shear Resistance

Figure 1: Slope Failure Mechanism

Lateral Load, P

2 to 4 Cd

Soil Movements

Figure 2

Distribution of Lateral Earth Pressures

(from Broms, 1964)
FIGURE 3 EXPERIMENTAL EQUIPMENT

- TANK IS 46 cm WIDE
- PRESSURE REGULATOR
- AIR BAG
- BAFFLE
- GAGED PROVING RINGS
- TEST PILE
- CLAY SOIL

Dimensions:
- 61 cm height
- 122 cm width

Note: The figures indicate that the tank is 46 cm wide, and the dimensions of the experimental equipment are 61 cm in height and 122 cm in width.
a) UNLOADED PILE CONDITION  b) LOADED PILE CONDITION

FIGURE 4  MODELING SCHEME TO ANALYZE THE FORCES IMPOSED ON THE PILE

FIGURE 5  COMPARISON OF THE EXPERIMENTAL $N_p$ WITH MATLOCKS $N_p'$
<table>
<thead>
<tr>
<th>TEST #</th>
<th>$\frac{L}{D}$</th>
<th>$N_p^\prime$</th>
<th>$N_p$</th>
<th>MAT-LOCK</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>4.9</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
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<td>5</td>
<td>4.8</td>
<td>4.6</td>
<td></td>
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<td>4</td>
<td>5</td>
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<td>4.6</td>
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<tr>
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<td>5</td>
<td>4.6</td>
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<td></td>
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<td>5</td>
<td>5.5</td>
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<td>9</td>
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<td>5.8</td>
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</tr>
<tr>
<td>14</td>
<td>7</td>
<td>5.7</td>
<td>5.0</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 1** COMPARISON OF EXPERIMENTAL $N_p$ AND MATLOCKS VALUE OF $N_p$

<table>
<thead>
<tr>
<th>$\frac{L}{D}$</th>
<th>$N_p^\prime$ EXP.</th>
<th>$N_p$ TH.</th>
<th>% INCREASE</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>5.02</td>
<td>4.56</td>
<td>10.1</td>
</tr>
<tr>
<td>7</td>
<td>5.80</td>
<td>5.00</td>
<td>16.0</td>
</tr>
<tr>
<td>10</td>
<td>6.25</td>
<td>5.85</td>
<td>6.8</td>
</tr>
</tbody>
</table>

**TABLE 2** PERCENTAGE INCREASE OF EXPERIMENTAL $N_p$
The casting of reinforced concrete structures under water is an operation demanding knowledge, experience and a responsible attitude from all those involved.

Several methods of casting have been used with varying degrees of success. Amongst these are the "skip" or "toggle-bag" method, the grout injection method, the Tremie-method and more recently, the hydrovalve method.

Of the above methods, the skip method dominates the field of casting unreinforced concrete under water - especially where this is temporary. The casting of reinforced concrete under water however, is usually performed by the Tremie-method.

**ESSENCE OF THE TREMIE-METHOD**

The Tremie-method consists of placing the fresh concrete through a watertight pipe. This enables all but the initial pour to be deposited within the mass of concrete already in place.

In theory at least, the initial pour of concrete is forced upward or forward and forms a "front" behind which all subsequent pours are deposited. The laitance and any impure concrete within this "front" is skimmed off at the end of the work where this is above water level. Below water level it is better to remove this layer after it has hardened, in order to diminish the chances of reducing the cement content of the remaining concrete.

Where the laitance layer does no harm, it can of course be left in position providing it is not included in the "structural" concrete for design purposes.

**DEVELOPMENT OF THE TREMIE METHOD**

The forerunner of the modern Tremie-method was that used by the French engineer M. Heude to cast bridge foundations in the River Loire in 1881. The method described by Heude (Heude 1885) enabled concrete to be placed within a wooden funnel and tube onto the surface of a mass of concrete already deposited.

Heude used a plug within the tube, but did not attach importance to the fact that the lower end of the tube should be within the mass of concrete already placed.
Some years later in 1894 the American contractor W.H. Ward was using an 8" pipe to deposit concrete in what was virtually the Tremie method as known today (Ward 1894).

By the turn of the century several important underwater mass concrete jobs had been completed in the U.S.A.

A young Norwegian Engineer, Mr. August Gundersen, returned home from the States in 1910 and took out a patent entitled "A Method for casting concrete columns and such like under water" (Gundersen 1912). This patent revealed a comprehensive understanding of the method as we now know it.

Gundersen was then with Høyer-Ellefsen A/S, Oslo (later of Condeep fame) and the method was generally known as the Contractor method - named after Høyer-Ellefsen's Swedish subsidiary firm - A.B. Contractor, Stockholm.

Outside countries within the influence of German engineering practice, the method was known as the Tremie-Method.

The method has remained basically the same since the turn of the century. Strange enough perhaps, remarkably little research or development work has been performed on the method. In fact few other countries have even given it the detailed attention it has received in Norway.

It has been ideally suited to Norway's requirements, necessitating the casting of many small quays on rock in isolated locations without sophisticated equipment.

A recent study made in Holland by the TNO Institute for Building Materials and Building Structures was described in a recent report. (CUR report no. 56.)

This report covered most aspects of underwater concreting and is perhaps the most comprehensive study of the subject to-date.

GENERAL PRACTICE FOR TREMIE CONCRETING IN NORWAY

After approximately 65 years of usage in Norway, a certain measure of agreement has been reached between parties involved, concerning what is considered to be good practice or otherwise.

The Norwegian Concrete Society has formed a standing working committee which recently produced a "Tentative Code of Practice for the design and construction of concrete structures under water" (N.B.F. 1976).

The greater part of this code is devoted to Tremie concreting and one hopes it will be of some assistance to those who specify or utilize the method.

There are some dozen or so Norwegian firms who are recognized as possessing adequate experience in the construction of concrete structures underwater by the Tremie-Method.

It is uncommon for Tremie contracts to go to other companies. The Code of Practice suggests certain criteria concerning the experience demanded from both consultants and contractors who are involved in Tremie work.

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It is common to use an 8" pipe with watertight flanges. This in turn restricts the minimum concrete dimensions, generally accepted to be 60 cm for reinforced walls and 70 cm for reinforced columns.

The distance through which concrete flows from any one pipe is generally kept to 2 or 2½ metres in order to avoid segregation.

The pipe is filled from a tract or funnel, supported on a platform in such a way that the pipe can be shortened as work proceeds.

In recent years one contracting company has produced excellent results from pumping concrete direct to the point of deposition (Dahl 1975).

A common arrangement for the basic equipment is shown in figure 1.

Great attention should be given to the start-up operation.

Beforehand, those concerned should assure themselves that alternative sources of concrete, or alternative equipment for its production, is available, should any fault occur in the main supply.

At the start, the Tremie pipe is full of water with its lowest point 10 cm or so from the bottom of the formwork. A plug is placed within the pipe to ensure that the cement content of the initial portion of concrete remains as it should be prior to it leaving the tube.

This plug usually consists of a metal plate of various designs, suspended by a wire from above. Football "bladders" are also used.

As concrete is added the plug moves downwards until it leaves at the bottom of the pipe. The addition of each new batch of concrete produces a characteristic "thump" within the tube. Any significant change in this sound is a sign of trouble and it therefore represents a rather good control indicator. The lower end of the pipe should be kept immersed approximately 70 cm in the mass of the concrete. This should be kept approximately constant as the level of the concrete within the formwork rises.

Usually the difference in concrete level within the pipe and the formwork is about 1/3 of the water depth within the formwork.

The rate of pouring should be kept under close control to ensure that the concrete does not harden within the formwork above the lower end of the pipe and thus prevent fresh concrete from leaving the Tremie pipe. A rapid rate of casting can, in the case of slender columns, entail high pressures on the formwork. A rate of 3 m per hour is not uncommon.

Where several tubes are involved, casting should commence at the lowest point. Concreting from adjacent pipes takes place when the ends of these are sufficiently immersed. Just prior to that, these pipes should be filled with concrete.

By filling the pipe gradually and carefully one can avoid pockets of air being trapped within the pipe.
It has been found that a poor concrete surface within the tidal- and splash-zones results if Tremie casting continues substantially over the water level existing at the time of casting. This is possibly due to the laitance being restrained by the frictional effects of the "dry" formwork surface.

It therefore seems wise to leave an opening in the formwork at water level, through which the laitance can be removed. Casting should then proceed by "normal" methods above this level.

It is of the utmost importance that no water be allowed to enter the concrete during casting. This can happen if the bottom of the pipe is lifted from the concrete.

It can also happen if the level of the concrete in the Tremie pipe is reduced simultaneously with a reduction in the immersion of the pipe. Therefore a minimum immersion of 70 cm is recommended.

Pockets of pebbles and poor quality concrete are also formed if quantities of fresh concrete are allowed to fall over the edge of the tract or funnel whilst filling is taking place. This error must be considered just as serious as water entering the pipe.

Should the worst happen and water enter the pipe at the initial stages of the work there is only one satisfactory solution i.e. to dismantle the formwork, "wash-out" and re-commence work.

Should water enter at a later stage, consideration can be given (with the consent of the designer) to a construction joint. All laitance is chiselled away and a cross sectional enlargement made in order to ensure total covering of the joint with reinforced concrete.

SOME "DESIGN ASPECTS" OF THE TREMIE METHOD

When designing underwater concrete structures to be cast by the Tremie method, one should take into account certain requirements which determine the easy flow of the concrete.

In Norway it is stipulated in the Concrete Standard that the outer 10 cm of the cross section of all joints made underwater should be neglected in the bearing capacity of the unit. This is warranted because the laitance and any other unwanted matter tends to collect at the periphery during the placing of the first batch of concrete.

Structures should not be designed with "overformwork" or with sloping surfaces flatter than about 45°. Laitance collects on such formwork surfaces and results in poor quality concrete at this position.

The casting of thin slabs by the Tremie method is both difficult and inadvisable. The upper surface will have a natural slope (of say 1:10) and the distance between pipes being less than 5 m, will give a concrete mass where adjacent pours have not the depth available to become a homogeneous unit.

Any attempt at screeding concrete surfaces under water is thwart with the danger of producing a weakened layer of concrete with a much reduced cement content.
Should screeding be nevertheless a necessity, the poured concrete level should be higher than the "final" level at all points. The screed can therefore proceed in one direction and remove the surplus concrete from the construction.

Vibration should likewise not be used under water as this can cause segregation.

Figure 2 shows a typical combined column and footing unit intended for Tremie work. It should be noted that the change in cross sectional dimensions is made gradually to avoid joints and ensure smooth flow of concrete.

The formwork for Tremie work is almost always prefabricated on land and floated out with all reinforcement in place. The required stiffness is obtained by using 10 cm x 10 cm "gussets" at all corners.

Until recently it was normal to use impregnated material from within 4 meter under low water level to 4 meter above highwater. Copper nails etc. were also used in this zone.

This part of the formwork was then left in place as a protection against ice damage etc.

Nowadays it is considered advisable to inspect all concrete cast underwater wherever this is at all possible.

Buoyancy forces caused by Tremie casting can be great wherever surfaces occur which are not vertical. Where these forces have been neglected or underestimated, the formwork has often lifted with disastrous consequences.

In general it is important the detailer should have in mind the need to ensure smooth flow of the underwater concrete in all directions.

FERRY BOAT HARBOUR NEAR STAVANGER

The construction of a new Ferry Boat Harbour to accommodate traffic between the Stavanger area and Skudeneshavn some 40 km across the West Coast Fjord area, commenced in the spring of 1977.

Two finger piers - each of 70 metres length are being constructed behind a rubble mound breakwater. The three berths provided are capable of taking vessels with a draught of up to 5,8 metres (see figure 3).

Loads from the finger piers are transmitted to the bearing strata through walls and columns cast underwater by means of the Tremie method (see figure 4). The water depth where Tremie casting is taking place varies from 6 metres to 11 metres.

Joining both finger piers is a reinforced concrete deck supported by Tremie-cast walls. This deck forms the edge of a filled-area used for marshalling the ferry traffic and at the same time forms the ramp-ends of the berths.

Horizontal forces are taken up by Tremie-cast walls which in part form a stone-filled caisson.
The wall thickness varies with a minimum of 60 cm and the columns are 75 cm in diameter. The Tremie cast concrete has a minimum cement content of 400 kg per cubic metre and contains 5% air entraining agent. The water-cement ratio must be less than 0.5.

Tremie concreting is carried out in accordance with the "Tentative Code of Practice for the Design and Construction of Concrete Structures in Water", the design and execution of the work being along the lines previously described in this paper. The total volume of Tremie cast concrete is approximately 1000 cubic metres.

Work is due to be completed so that the berths are ready to take the summer traffic of 1978.

It is hoped that several cores can be taken of the concrete as cast in place. This will enable a comparison between the relative strengths of the concrete before and after placing under water.

By colouring the concrete of adjacent Tremie pipes it is furthermore hoped to study the nature of the interface between them. In this connection it is of interest to ascertain whether a contact area of laitance is formed, whether the joint is capable of transferring forces and whether it is to be considered as watertight.

In addition there are other minor aspects of Tremie cast concreting which it is hoped can be looked into.

Actual Tremie concreting started in mid 1977 somewhat later than planned owing to problems with the sea bed conditions.

DRYDOCK FOR HAUGESUND MEKANISKE VERKSTED, HAUGESUND

A "Fitting-out quay" at Haugesund in S.W. Norway is to be incorporated into a new drydock of sufficient size to take vessels of 130,000 tons dead weight (see figure 5).

The water-retaining walls of the dock are cast by the Tremie method onto the bedrock.

The eastern wall of the dock is formed as a Tremie-cast gravity wall with a trapezoidal cross section (see figure 6) and prestressed to the bedrock. In addition there are rows of rockbolts reinforcing the joint between concrete and rock.

At the deepest part, the gravity wall is 15 m high and had a width at the base of 10 metres. The Factor of Safety is 1.3 against overturning.

Much of the wall is constructed under an existing quay which causes certain problems and modifications to a normal casting sequence.

Under each crossbeam of the existing quay, the contractor has chosen to cast a narrow cross section of the gravity wall. These portions are then used as a guide to formwork used for the longer portions in between (see figure 7).

Formwork material was lowered through slits made in the existing decking and assembled under water.

The western wall is cast progressively outwards in sections of 10 and 6 metre lengths.
The formwork is fabricated in two parts, joined later underwater. This lower part is made to conform to the rock profile at the point in question and acts as a support for the heavier upper formwork with reinforcing enclosed.

Horizontal water pressures are transferred to bedrock via prefabricated sloping columns which are positioned under water and then cast in place at each end. The lower end is cast into a foundation on bed rock whilst the upper end is cast into the water retaining wall (see figure 8).

The sloping columns occur only within the 10 m long sections. Horizontal forces acting upon the intermediate 6 m long sections are transferred to the 10 m sections through keyed joints. There is also a load transfer via the new quaydeck above.

At the same time the prefabricated vertical columns supporting the crane beam and the quaydeck are placed under water and cast into foundations upon the rock.

The crane beams and quaydeck are cast so that the whole west side of the dock is completed progressively.

Problems have arisen in casting the eastern wall sections. Temperatures of 60°C within the concrete have been measured. This has caused side sections to be eased outwards and some cracking has occurred.

Within both walls, tubes have been cast in order to allow boring down into the bedrock and subsequent injection of possible leakages prior to dewatering of the dock.

The dock entrance will be cast in sections. The exact method of partitioning the concrete is being discussed with the contractor and is not finalized.

The concrete quantity within the larger section will be 800 - 1000 cubic metres.

The dock-cill will be cast in sections and the casting will of course be finished underwater.

At the head of the dock and at the cill there will be cast a provisionary stop for the dock gate. After dewatering, permanent fixtures for the Dock Gate will be cast.

The total quantity of concrete cast under water is approximately 16000 cubic metres of which half has been poured up to the summer of 1977.

An interesting aspect of the work is that the inner surface of the Tremie cast concrete will be accessible in a dry state when the dock is dewatered.

In addition it will be possible to examine in a dry state existing quay columns cast by Tremie methods some years ago.

The dock will thus provide valuable experience and a possibility to examine Tremie cast concrete in detail. It is thus hoped that this method of casting concrete under water will be further advanced.
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Fig. 1 Working arrangement for Tremie-casting.
Section A-A with formwork.

Elevation of combined column-footing.

Fig. 2. Combined Column and footing suitable for Tremie-casting.
General arrangement plan.

Fig. 3. Layout of ferry boat harbour. Mekjavika near Stavanger.
Fig. 4. Tremie-cast wall and columns.
Plan of drydock - (schematic)

Fig. 5. General layout of dock - Haugesund.
Fig. 6. Section thro' eastern wall
Fig. 7 Part-plan of eastern wall
Fig 8. Typical section thro' West Wall
1. Introduction

I/S Norwegian Contractors, a joint venture formed by the three Norwegian Contractors Ing. F. Selmer A/S, A/S Høyer Ellefsen, and Thor Furuholmen A/S, received their first large order for an offshore concrete structure the 14th May 1971. This was the "Eccofisk Tank", a Doris design, that marked the introduction of concrete in large scale as building material offshore. Since then seven large structures have been constructed, of which five are of Norwegian Contractors own design, the Condeep, and they are all now successfully towed from their building sites to final positions in the North Sea.

These structures are:

- Eccofisk Tank
- Beryll A Condeep
- Brent B Condeep
- Frigg Total
- Brent D Condeep
- Statfjord A Condeep
- TCP2 Condeep

These platforms ranging from 250 000 tons to 400 000 tons displacement and with tow-out draft ranging from 50 m to 120 m.

The construction of such objects is initially carried out on dry land in a building dock. Here the "Base section" is built, that is the lower part of the floating caisson and so much of the caisson walls that make it float with safe margin of freeboard.

This paper will give an account of the maritime operations performed by "Norwegian Contractors Maritime Operation Group" related to construction of the Condeep platforms constructed in Stavanger and Andalsnes, Norway, and towed out to final destinations in the years 1975-76 and 77.
2. The scope of work

The scope of work can be divided into the following tasks:

- Preliminary investigation of the structures hydrostatical and hydrodynamical properties and behaviour.
- Geographical location of the construction site and possible tow out routes to final location.
- The building dock, maritime considerations.
- Design and installation of mooring system on "deep water" site.
- Floating off "base section" and towing from building dock to moorings at "deep water" site.
- "Deck mating" operation.
- Investigating and survey of towing routes.
- Tow out from "deep water" site to final destination.
- Positioning on site.

3. Organization

Right from the start of the Condeep project it was regarded necessary to form a group taking care of the involved maritime problems, find solutions, make designs and operation plans as well as taking care of the actual operations. In other words, dealing with all maritime problems from A to Z.

This group consisted of people with various background, experience and education. Mechanical Engineers, Naval Architects, Mariners, work supervisors and a handpicked crew of highly skilled workers with a wide range of experience.

We made it the general rule that the one who should be responsible for the execution of a job, he should also have a central position in the planning group. This ensured a very detailed planning, and the tasks were carried out according to plans because the operators knew and understood "the whole story" and did not feel tempted to "short cuts" or shortsighted improvisations.

All planning had to be documented in writing with drawings, charts, calculations and documented information as model tests, weather systems, charting etc etc.

To satisfy the Underwriters all documentations had to be approved by Noble Denton and Ass. of London, and this Company had also their inspectors on board during operations. Documentations and operational plans were also checked and approved by Det Norske Veritas, Maritime directorate, Board of Trade, Harbour authorities, Client etc etc.

4. Preliminary investigations of the structures hydrostatical and hydrodynamical properties and behaviour.

The hydrostatical properties of a structure like a Condeep can be worked out analytically from drawings and assumed weights etc.
These calculations, however, must be worked out for every step during the building phase to ensure proper stability during the whole building process.

These calculations certainly grow more and more complicated as the structure closes in on completion. The object is to maintain acceptable safety as to stability and seaworthiness during tow-out combined with max. possible payload, thus bringing the structure as close to completion as possible before tow out to the oilfields. Before tow-out several inclining experiments are performed.

The hydrodynamical properties are found mainly from model tests in tanks. Behaviour under influence of waves is important for tow-out and final positioning. From observations during actual operations we have found good correlation with the model results, and we have been able to advance criteria for waves and wave periods during positioning and "touch down". Towing tests have given us valuable information on obtainable speed relative to available bollard pull from tugboats. From these tests were also established self-stopping distance, behaviour when towing into turns etc.

5. Geographical location of the construction site and possible tow out routes to final location.

The Norwegian west coast is possibly the most suitable area for the construction of large concrete objects for the oil industry. Deep fjords are found in this area, well sheltered from wind and waves, and deep passages exist out to open sea, where the so-called Norwegian Trench forms an excellent "sea road" to all known oilfields in the North Sea and the Atlantic.

In the Norwegian fjords, relatively flat, low lying areas consisting of moraine and sandy clay are also found. Such ground lends itself extremely well to the construction of building docks because it is watertight, possesses high bearing capacity, and minimizes risk of soil failure. Next to the dock area, large areas are needed for stores, barracks, offices, work areas etc. High quality sand and gravel are found in the fjords and may easily be transported by small ships to the building site. It is therefore required that the building site should have a small harbour. The building dock must be provided with an approach canal to deep water, through which the base section can be floated.

Outside the building site a "deep water site" must be established. Heavy moorings must be installed into which the base section can be secured, and remain during the process of concreting, deck-mating and outfitting before towage to the North Sea. This site must be so deep that the whole concrete structure can be submerged, apart from the remaining 6 m of the shafts, when the deck is installed. Water depth in Stavanger is 243 m.
A towing route from the deep water site to open water outside the coast must be found and thoroughly investigated and charted before the site can be finally selected.

The biggest advantage with a concrete platform is that it can be made almost complete before tow-out. This means that one will be aiming for the highest possible deck load. High deck load can be obtained by an increase of the concrete structures volume (added displacement), that will increase carrying capacity and stability, or increase the tow out draft. The latter being far the cheapest solution.

However, max draft calls for towing routes permitting the platform to pass with safety and a min. of 15 m under keel clearance should not be surpassed. From Stavanger max draft is 80 m, from Andalsnes 90 m, from Stord southeren route 115 m and from Vats 200 m. Condeep Brent D was towed at 120 m draft 1976.

The only way a towing route for a particular platform can be established is by special charting of the route where every square metre of the sea bed is covered. Special contour charts are prepared and the towing route is drawn in with due consideration to depth, side clearance, currents and good seamanship.

Before the final selection is made among the alternative building sites, it is strongly advisable to bring the underwriter's maritime consultants into the discussion, since this body will later approve all maritime operation plans before they can be put into action. These discussions are regarded as extremely important and fruitful and may reduce a number of problems later in the process.

6. The building dock, maritime considerations.

The building dock is formed partly by excavated sea bed and partly by a dike made by sheet piles stabilized with sand and gravel filling on both sides. Drains and drain wells are put in and electric pumps keep the dock dry from rain water and possible leakage. The dock is filled by means of electric pumps pumping water over the dike. This is considered safer than direct sluicing. When the dock is filled, a minimum safe clearance between bottom of dock and bottom of structure of 50cm is established. The dock is opened to the fjord by retracting the sheet piles and dredging out the sand and gravel dike. A safe clearance of 20 m from the base section to each side is required and the depth is checked by echo sounder and a horizontal steel bar hanging under a small tug. The safe canal is marked with buoys showing the extreme clearances on each side. There is a system of warping winches in strategic positions outside the dock area and a system of warping wires, through snatch blocks, connected to the structure. There is one connection on each side to control transverse movements and one at the stern for braking action. The warping system has two purposes: to keep the base section securely moored between lift-off and tow-out, and to
perform the early stages of tow-out by winching. The tugs gradually take over the job as the platform moves through the canal to open water. The warping winches used have a capacity of approximately 20 t and a maximum hauling speed of 5 m/min. Practical operating speed is about 2 m/min.

The Condeep platforms are fitted out with so-called steel skirts underneath. These skirts form a number of cells and may be filled with compressed air, thus increasing the buoyancy and reducing draught. This system is of immense value as the excavation of the building dock is reduced by more than 3 m. Furthermore, the concrete structure rests on its foundations even after the dock has been filled, and all tests before launching can be performed in full safety. Only hours before the actual tow-out the air is injected as the float-off takes place. This is also the reason why the warping winches can be used as moorings inside the dock before tow-out.

7. Design and installation of mooring system on "deep water" site.

The main purpose of a deep water mooring site is to accommodate the concrete structure during the production phase. Apart from slip-form construction, this phase includes deck mating and final outfitting till the platform is ready for final tow-out. As the mooring site will have to accommodate the platform during the deck mating operation, a major requirement is sufficient water depth at the site. It is also required that the mooring system be flexible and easy to operate and have sufficient strength to withstand specified weather situations.

Both the design of the mooring system and the laying operation are inspected and approved by the underwriter's maritime consultants. It is required that the forces in the most highly stressed mooring chain should not exceed 70% of the guaranteed breaking load of the chain with wind, waves and current acting simultaneously in the same direction. The wind speed used is that given by the meteorological offices for a 100 year return winter storm. Current speed and wave height are based on measurements on the site. The meteorological offices involved in this have been IMCO and the Norwegian Meteorological Institute, and when comparing the figures given by these two bodies they agree very well, the only difference being that the Institute tends to give reductions if the site is protected by hills, whereas IMCO tends to recommend open coast values inshore.

As an average of the total forces in the chains, the wind forces account for the greatest part with 80%, current forces 17% and wave forces 3%; this is with the caisson submerged and only the three towers protruding above the surface, and with the steel deck mounted. Before the final selection of the mooring site the area must be charted; this is done by echo sounding equipment in a close grid system and the data form the basis for our 1:2000 anchoring maps. The mooring system is made up of three
legs at 120° to each other and where the maximum load in one chain will be increased by 30° if the direction of load is perpendicular to one of the two other chains. For the moorings in Gandsfjorden in Stavanger where the bottom of the middle of the fjord is flat, and slopes gently in to shore, it is most suitable to have one chain to shore and two sea legs. The land stations consists of a hydraulically powered anchor winch and a chain locking device, and the two sea chains each lead to two shovel type mud anchors well submerged in the sea bed.

All three connexion points fitted to the concrete structure, have been designed to have the same breaking strength. This is in excess of the breaking load of the chain. The sea legs are fitted with heavy chains near the ends, and concrete weights so as to limit the lifting angle along the sea bed to less than 5° and hence avoid the mud anchors breaking out of the sea bed. The depth at the deep water sites is about 240 m and the required chain length for the sea legs about 650 - 800 m.

Some of the fjords where we have deep water sites are too narrow to accommodate these long sea legs in the 120° configuration. The sea bed may be of a quality less suitable for the mud anchors, the soil being present only in thin layers, the thicker layers being probably situated nearer the fjord opening. On these more difficult sites the geometry of our mooring system can be maintained only by using one shore connexion in the form of a rock anchor and two land stations, each fitted with a winch and a chain stopper.

The chain laying operation is carried out with a chain laying barge moored alongside a flat-top pontoon. The laying barge is fitted with a messenger wheel in front, a chain lock, a heavy winch, a sheave/pulley system and a slide passage over to the flat-top pontoon moored alongside it. The pontoon measures about 15 m x 60 m and has ballasting and trim tanks. The chain lengths are stored and coupled on board the pontoon before they are pulled over to the laying barge. The chain is paid out from the laying barge and to give sufficient tension in the chain a tug will pull this rig in the direction instructed by the surveyor, who relies on theodolites and cross-bearings. When the mooring chains are laid the three ends are left hanging in three pontoons each positioned so that they are about 30 m from the caisson when the platform arrives.

8. Floating off "Base section" and towing from building dock to moorings at deep water site.

When the caisson has reached a height of about 15 m it has enough strength in its midship section to float and overcome imposed forces. At this stage slip-forming is stopped and the base section is transferred to the deep water site. At this stage the base section is inspected, and all installations, that later will remain under water, are approved.
Inspection at any later date will have to be done by divers. The building dock is cleared of all obstructions and water-filling can commence. The water is then pumped into the dock until water levels outside and inside the dock are equal. The base section still rests on the dock base because the weight is far greater than the obtained buoyancy.

To bring it afloat just before tow-out compressed air is introduced under the skirts and as these continue below all nineteen cells this extra buoyance can be distributed as desired so as to reduce the still water bending moments. After waterfilling, the sheet pile wall forming the dock opening is retracted, and the remaining dike can be removed by dredging. When the 60,000 m$^3$ of gravel in the dike had been removed, the sea bottom level is checked with a small tug towing a horizontal bar underneath, back and forth, in the dock opening and exit canal. Any irregularities on the bottom can then be detected and rectified. The first part of the tow-out of the bottom section from the dock is very difficult and is a critical operation. The major problem is to limit the transverse movements; and this has been taken care of with the three dock winches described previously. To assist the maneuvering of the unit a harbour tug is tied up to the stern of the platform and will act as stern thruster. When the air cushion, mentioned earlier, is introduced, the base section is afloat and is held in position by the dock winches until the towing starts.

On the evening before tow-out, the meteorologist gives the final briefing on the weather situation, and if favourable the tow-out starts early the following morning. The three main tugs are anchored outside the dock opening with their towing gear attached to the platform. When the actual tow starts, the tugs pull in on their towing winches as the dock winches pay out gently. When clear of the dock opening the tugs hoist their anchors and proceed down the buoyed canal to the deep water site.

Navigation during the tow is done the same way as during the anchor laying, with one surveyor on board and two theodolite stations on shore. When approaching the deep water site, small tugs are coupled to the pontoons to pull these away from the structure as it arrives. The pontoons are then pushed back into contact with the structure on the places where the anchor chains are to be connected with anchor shackles.


The steel deck that shall be mounted onto the "shafts" or towers of the concrete structure, is mainly constructed at a shipyard some distance away from the "deep water" site.

The decksection is transported on two barges as a catamaran where the horizontal distance between the two barges permit the deck to float in over the "shafts" of the Condeep.
Free height from waterlevel to up under the deck section being approx. 6 - 8 m. Before deck-mating the concrete structure will be ballasted down so that the top of the "shafts" protrudes only so much above water surface permitting the deck catamaran to slide in over the shafts.

When mating the deck for Condeep TCP2 in Åndalsnes this year, the deck having a weight of 8 400 tons was towed to the concrete structure by 6 tugs, one at each corner and two moored alongside each barge. 4 of the tugs were so-called "Shottel-tugs" and gave us perfect control. The tugs brought the deck in over the shafts. When nearing the position, wires were connected to the shafts from 4 winches on the deck of the barges. From now on positioning was taken over by operating the winches.

When the deck had been positioned to an accuracy of approx. 20 cm from theoretical position, 6 hydraulic cylinders fitted to vertically sliding supports, were lowered down and into position. These cylinders acted against the shafts and were manoeuvred from a central operating panel on board one of the barges. This system made it possible to position the deck within a tolerance of +/- 1 cm, and the 428 connecting bolts all found their respective holes without trouble.

When the deck was brought into final position and locked in place by the cylinders, the Condeep was gradually deballasted and the deck lifted off the two catamaran barges.

10. Investigating and survey of towing routes.

The towing route from "deep water" site to open water outside the coast, and across the banks from the Norwegian Trench to the target area must be thoroughly investigated and charted.

Normal sailing charts, neither on the coast, nor in the North Sea, are of such accuracy that they can be used for this purpose. Large scale special charts must be made (1:10 000 and 1:5 000) with contourlines for every 10 m drawn in. On these charts the theoretical courseslines and sailing corridors are marked up.

When fixing the courseslines and sailing corridors, due consideration must be taken to safe underkeel clearance, proper side clearance and necessary room for operating the tugs in the narrows between the islands. Min. underkeel clearance is 10 m and side-clearance at the bottom of the structure should never be less than 40% of the structures bottom diameter.

When fixing the towing route due consideration must be given to currents and wind conditions, that may have a major influence on the success of the towing operation.
The charts made of the final target area are of great accuracy. Scale being 1:5 000 or even 1:2 500 with contourlines down to 0.5 m. Every obstacle like well heads, rocks, marker buoys, scrap etc. that might come in conflict with the structure, is either removed or drawn in on the charts.

The navigation system used when towing through the fjords to open sea is based on shorebased manned theodolite stations from where the Condeep is continously sighted. The bearings are transmitted by VHF to the navigational bridge on the platform, where the bearings are plotted on the sailing charts and position found. Accuracy is down to a few meter and we can receive one plot every half minute. In addition and as backup system we are using Motorola Mini Range, a radio based system giving distances between platform and shorebased transponders. This system gives great accuracy and through a computer gives information of sailed distance, deviation from theoretical courseline, speed, direction and sailing time to next course alteration.

In addition to above systems Decca navigator is installed and the narrow passages marked up with marker buoys. Ocean navigation is carried out by Decca navigator.

Positioning at the field is mainly done by aid of ATNAV acoustic positioning instrumentation. Prelaid transponders on the seabed respond to acoustic signals from the Condeep. Plotting is automatic through computer. When any other fixed platforms are within sight, theodolites and Motorola are used in addition to ATNAV or replace this all together.

11. Towouf from "deep water" site to final destination.

70 000 IHP have been regarded necessary for giving a Condeep platform acceptable safety during the summer season in the North Sea and on the Norwegian Coast. In still weather and long towlines (approx. 800 m) 2.5 knots will be obtained. This towing power has been developed by five tugs, each ranging from 22 000 to 10 000 IHP and totally producing a bollard pull of 550 to 600 tons. Towing through the fjords where space is restricted, the tugs will have to operate with min. length of towline (approx. 100 m) in order not to run aground through the windy narrow passages one will encounter. In addition three more tugs are connected astern for steering purposes and be able to slow down or stop the tow on short distances and time. During fjord tow as much as 85 000 IHP may be connected altogether.

The towlines are connected to the upper part of the main structure, that may be as much as 50 m below the water surface. Towing with short towlines would mean that the towing wire from the tug would have an inclination of approx. 30° over the stern and make it quite impossible for the tug to manoeuvre. Therefore we installed a powerful winch for each towline on the deck, and from these towing wire is paid out and connected to the main towing wire with a shackle approx. 100 m from the connecting point. This forms
a vertical bridle to which apex the tugboats own towline is connected. By adjusting the winches we can keep the towing wire from the tug horizontal at any draft and give the tugs their needed freedom to manoeuvre.

However, when towing offshore, where tugs pay out up to 1000 m towline, the winches are slacked off so that the towing forces are transmitted straight into the concrete structure.

On final destination at the oilfield, the tugs are rearranged around the Condeep forming a "star" and as such can move the platform in all direction and hold it firmly during the actual "touch-down" and first part of the penetration. Special pontoons are connected to the towlines at this stage of operations for the same reason as for the winches, but also that the pulling forces from the tugs are transmitted through the centre of buoyancy to prevent heeling of the platform at the point of making contact with the seabed.

All orders to the tugs are given from a "Captains Bridge" mounted forward on the deckstructure. This bridge is fitted out with all above mentioned navigational instrumentations as well as ecco sounders, Sonars, Gyro compasses, VHF and SSB radio transmitters, radar etc. Telephone system connects the bridge with the ballasting operation centre and the computer central, so that also these are receiving their operational orders from the bridge during positioning.

During tow-out the normal crew consist of approx. 45 persons as follows:

- Chief of operations
- 7 men attending the ballast systems
- 7 " taking care of generators and other lifesupports installed on the deck
- 15 " as navigation crew including captain, towmasters, officers and surveyors
- 5 " catering crew
- 10 " representing client, underwriters, Veritas, wireless operators, medical, met. services etc.

When reaching the destination, the crew is nearly doubled by people specialised in the various fields as geotechnical, concrete design, computer operators, penetration crew and personell being ready to start grouting operations when the platform has been penetrated into the seabed.

12. Positioning on the site.

Before the actual final positioning can take place, the weather situation must be the very best, and most important is that waves are below certain set limits. At the moment of "touch", the platform must have no vertical movements that can cause "pumping action" or cause severe stresses to the parts of the structure.
that makes contact with the seabed. From model testing it is found that a Condeep may come into motion when 2 m significant waves have a period exceeding 8-10 seconds.

Beneath the platform are the steel skirts that will penetrate into the sea bed. In addition there are three "dowels", each projecting below the edge of the skirts by 4 m. These will be the first parts to make contact with the sea bed. Gradually, as the dowels penetrate the sea bed, they will stop all horizontal movement, enabling the steel skirts to start penetration without any side strain.

The position procedure for a Condeep is very much as landing an aircraft, as it slowly moves into the target area, the platform is ballasted down until the dowels gently "scrape" the seabed, and the whole process can be stopped by quick ballasting when on the point. As an example of accuracy can be said that TCP2 positioned at the Frigg this summer was 1.9 m from theoretical target, the orientation was fully in accordance with specifications and the clearance between the concrete base of the _TCP2 and the neighbouring TPl platform was 25 m. A bridge will now be built between the two platforms.
MAINTAINING AN ICE-FREE HARBOUR BY PUMPING OF WARM WATER

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THE HEAT BUDGET

Freezing reflects an imbalance between loss and supply of heat at the water surface. Such an imbalance may occur over a range of heat fluxes. Typical heat losses through an open water surface are 100-500 \text{Wm}^{-2}. On a clear night the radiation loss is 120-150 \text{Wm}^{-2}. The higher rates of heat loss are caused by wind. Actual freezing, however, occurs most frequently at low to moderate wind speeds when the heat flux is not excessive.

Freezing is preceded by a period of cooling during which the available stores of heat are depleted. A heat balance equation will yield the time history of the water temperature provided the various terms in the heat budget are sufficiently well known. General results are available primarily for the heat flux \( q_h \) per unit area of the water surface (Freysteinsson, 1968, McPadden, 1976).

\[
q_h = q_{lw} - q_{sw} + q_e + q_c
\]

(1)

\( q_{lw} \) - outgoing long wave radiation
\( q_{sw} \) - incoming short wave radiation
\( q_e \) - evaporation loss rate
\( q_c \) - convection loss rate

Much less is known quantitatively about the natural processes that supply heat to the water surface from the bulk of water under consideration.

Suppression of ice

By artificial means it is possible to increase the amount of available heat to make up for the lost heat. In lakes a commonly used method is to release compressed air at the bottom and let the rising air bubbles generate a vertical flow of warm water. A summary of this technique was given by Ashton (1974). Occasionally air bubbling has been attempted in sea water, but without much success, because natural convection efficiently depletes the heat right down to the bottom. An exception is the layered estuary which has large quantities of heat trapped beneath a pycnocline (Carstens, 1971).
Pumps for horizontal transport

If there is any surplus heat stored in the water column, air bubbles will bring it up to the surface. Whenever vertical transport only is required, the release of compressed air is an expedient and flexible method. Energywise, its efficiency is poor, however.

If the heat source is found at some distance from the heat sink so that horizontal transport is required, air bubbling is ruled out. Pumping now becomes for all practical purposes the only alternative. However, as is demonstrated below, it is an attractive solution for almost or mostly accessible sea and lake ports troubled by ice within the basin and in the nearest approaches.

Another use may be to prevent damage to docks and piers in icebound harbours during the winter. Pumping of warm water then maintains open water near the structures, relieving them of any ice pressure.

Waste heat - An indication of the possibilities inherent in pumping is afforded by the polynyas at the outlet of cooling water from thermal power stations or factories such as paper mills. Frequently the release of modest quantities of water, 1 m³/s, say, with from 5 to 10°C water temperature, is seen to maintain substantial surface areas ice free. In many cases these cooling water polynyas are not desired because they create ice fog. To prevent such unintended holes in the ice cover has proved difficult, however.

Fig. 1 shows the permanent opening in the vicinity of a cooling water outlet from a paper mill in the port of Oulu, Finland. In this particular case the warm water is beneficial as it eases the work of the harbour icebreaker. The example from Oulu demonstrates what can readily be shown simple computations theoretically, namely that pumping is a practical method of preventing freezing in many cases.

PUMPING OF WARM WATER

We consider a tidal harbour and write a heat balance made up of the following four terms:

1. Rate of change of stored heat = \( c \rho \frac{\Delta W}{\Delta t} \)
   \( c \) - specific heat = 1 cal gram \(^{-1} \) °C \(^{-1} \)
   \( \rho \) - density = 10 \(^6\) grams m \(^{-3} \)
   \( \Delta W \) - volume of basin at MSL
   \( \Delta T \) - change in water temperature \( \Delta T \) per tidal period \( \Delta t \)

2. Net heat supply by pumping = \( c \rho Q_p (T_p - T) \)
   \( Q_p \) - pumped discharge, m \(^3\) s \(^{-1} \)
   \( T_p \) - water temperature at the intake
   \( T \) - water temperature in the basin

3. Net loss with the tide = \( c \rho \frac{\Delta \Psi}{\Delta t} (T - T_i) \)
   \( \Delta \Psi \) - tidal volume = \( A \cdot H \)
1. Water temperature of the incoming tide.

4. Net loss through the water surface = $q_h \cdot A$

$$q_h = \text{heat loss to the atmosphere, cal s}^{-1} \text{m}^{-2}, \text{given by (1).}$$

We assume that no change of state takes place, so that the heat balance equation becomes

$$c_p \frac{\Delta T}{\Delta t} = c_p Q_p (T_p - T) - c_p \frac{\Delta W}{\Delta t} (T - T_i) - q_h A$$

We are primarily interested in the steady state temperature or equilibrium temperature with pumping, which is obtained by putting (6) equal to zero:

$$c_p Q_p (T_p - T) - c_p \frac{\Delta W}{\Delta t} (T - T_i) - q_h A = 0$$

yielding

$$T = \frac{Q_p T_p + \frac{\Delta W}{\Delta t} T_i - \frac{q_h A}{c_p}}{Q_p + \frac{\Delta W}{\Delta t}}$$

In a non-tidal harbour, say, in a lake, this equation simplifies to

$$T = T_p - \frac{q_h A}{c_p Q_p}$$

We shall apply (8) for a real tidal harbour and (9) for a hypothetical lake harbour to explore the feasibility of pumping schemes to secure ice-free conditions.

**Tidal harbour**

In (8) some assumption must be made about the temperature $T_i$ of the incoming tide. Clearly this will depend greatly on local conditions of circulation and mixing within the basin and the coupling of these processes to the weather.

For the shallow bays that are most prone to freezing, we have looked at the simplest case when horizontal transports do not affect the heat balance.

During freezing episodes the temperature $T_i$ of the incoming tide rapidly approaches the freezing point $T_f$. In the absence of advected heat the local rate of cooling would be

$$T_i = T_{i0} - \frac{q_h n \Delta t}{c_p d_i}$$

where $n = \frac{\Delta t}{\Delta t}$ is the number of tidal cycles after time $t_0$ when $T_i = T_{i0}$; $d_i$ is the local depth. Table 1 gives the time in tidal cycles required for a water column of height $d_i$ m and initial temperature $T_{i0} = 0^\circ C$ to reach the freezing point $T_i = T_f = -1,9^\circ C$, i.e.

$$n = \frac{c_p d_i T_f}{q_h \Delta t}$$
It is clear from the table that shallow bays are cooled from 0°C to the freezing point in a day or two except when the heat loss is very weak.

For the present purpose it is sufficient to make the conservative estimate

\[ T_i = T_f = -1.9°C \]  
(12)

corresponding to a salinity of some 33 ᵒ/ₒo.

Application to Kiberg harbour - We now apply (8), using assumption (12), to the harbour of Kiberg which has the following characteristics:

- Surface area at MSL: \( A = 78 \cdot 10^3 \text{ m}^2 \)
- Mean tidal range: \( H = 2.5 \text{ m} \)
- Mean tidal volume \( \Delta V = A \cdot H = 0.2 \cdot 10^6 \text{ m}^3 \)
- Heat source temperature \( T_p = 2°C \)

Insertion of the known values gives the steady state temperature as a function of the pumped discharge \( Q_p \) and the surface heat loss \( q_h \):

\[ T = \frac{2Q_p - 8.34 - 0.078 q_h}{Q_p + 4.39} \]  
(13)

This formula is plotted in Fig. 2, and it is seen that a pumped discharge of \( Q_p = 2 \text{ m}^3/\text{s} \) is sufficient to prevent freezing.

Lake harbour

Take as an example a small harbour basin of the same acreage as Kiberg \( (78 \cdot 10^3 \text{ m}^2) \) and set \( T \) in (9) equal to the freezing point 0°C for fresh water. The pumping rate is given by

\[ Q_p = 0.078 \frac{q_h}{T_p} \]  
(14)

which is shown in Fig. 3. A likely intake temperature is \( T_p = 3°C \) which makes \( Q_p = 2 \text{ m}^3/\text{s} \) suffice except for very strong cooling.

It appears that the pumping requirements are fairly similar for the two cases. The explanation is that for sea water the depression of the freezing point lowers the need for heat by about the same amount as tidal losses raise it, compared with the non-tidal fresh water harbour.
MIXING OF PUMPED WATER WITH THE AMBIENT

The derivation of (8) is based on the assumption (4) of ideal mixing of the pumped water with the resident water. This is not merely a convenient mathematical assumption, it is also essential to prevent partial freezing. The warmer pumped water is slightly heavier than the ambient; fresh water because of the negative coefficient of thermal expansion, sea water because of higher salinity. Unless vigorous mixing is secured, the released warm water is likely to form a density underflow that carries the heat out of the basin again.

For Kiberg harbour a 1:120 scale Froude model was built to study the mixing. (Billfalk and Mathisen, 1976.) A density surplus of 1 kg m\(^{-3}\) was assumed for the warm water, which is found at a depth of say 20 m about 250 m from the harbour. An outlet velocity of 5 m s\(^{-1}\) gave plume lengths of some 200 times the outlet diameter. At this point a density front formed, and the plume sank below the surface. However, the measured dilution in the escaping undercurrent was close to the prediction based on ideal mixing.

Fig. 4 gives a sequence of pictures from the model. The warm water is released as two equal jets, and the mixing appears to be satisfactory. The escaping undercurrent can be seen at high tide in Figs 4b and 4c.

SUMMARY

No new technology is needed for pumping warm water to make up for the heat loss from a harbour. The various parts of such a pumping scheme are commented on below.

1. The suction pipe may become a costly item if it is long and exposed to wave action (submerged pipe) or requires insulation (overland pipe in permafrost).

2. The pumping station requires a power supply of the order 1-10w per m\(^2\) of water surface to be kept open. The pump must supply the kinetic energy required for mixing, which is of the order of 1 m, and the energy absorbed by friction which may range from, say, 2 m to perhaps 20 m. The demand for potential energy is insignificant.

In sea water the cooling water pumps developed for turbine engines are well suited for our purpose.

3. The warm water jet or jets should be directed so as to entrain as much as possible of the resident water. Pipes or jets should not make navigation hazardous.

4. A feedback loop from a thermistor properly located in the basin controls the supply of heat by switching the pump on and off (except for waste heat).

5. Pumping as envisioned here does not eliminate an ice problem that arises from imported ice, ie ice that is drifting into a harbour or blocking its entrance. However, ice problems arising from a local heat deficit appear to be neatly avoided by pumping of warm water.
CONCLUSIONS

Along the rim of the true Arctic there must be many harbours resembling those we have encountered, where an ice cover can be eliminated by transporting warm water a few metres vertically or a few hundred metres horizontally.

Where a warm water source exists within reasonable distance from the harbour, say a few kilometres, the technical problems of transporting the necessary heat and mixing it into the basin are trivial.

The cost of a pumping scheme depends crucially on the length of the pipe. The other cost components, including power for operation, are rather modest.

ACKNOWLEDGEMENT

The ideas developed here emerged during a study of the Kiberg harbour in Finnmark, Norway commissioned by the Norwegian State Harbour Board.

REFERENCES


Fig. 1. Ice-free area near outlet of cooling water, Oulu, Finland.

Fig. 2. Water temperature vs heat loss for a tidal harbour.

Fig. 3. Pumped discharge vs heat loss for a lake harbour.
Fig. 4a. Distribution of pumped water 1/4 tidal period after start, t = 1/4 T.
Fig. A. Same as 4a,
$t = 1 \frac{1}{4} T$. 
Fig. 4d. Same as 4c,
\[ t = 1 \frac{3}{4} \]
INTRODUCTION

Following the great oil and gas discoveries on the North Slope of Alaska in the 1960's and subsequent construction of the trans-Alaska hot oil pipeline, attention has recently been focused on drilling and recovery techniques which may be utilized for exploration and production drilling in offshore oil and gas fields in the Beaufort Sea, a portion of the Arctic Ocean. Ice islands, gravel embankments and man made subterranean caverns have been proposed as possible means for well drilling and production operations. Prefabricated drilling platforms of steel and concrete have also been envisioned. Union Oil Company has drilled exploratory wells from an ice island near Prudhoe Bay, Alaska and British Petroleum has constructed a small gravel island in the same vicinity. Canadian firms have concentrated their offshore structures near the mouth of the MacKenzie River and at least eight permanent offshore structures have been constructed in this area. Offshore ice islands have also been constructed in this area. (Pimlott, 1974).

To date, offshore embankments constructed of sand and gravel have become the favored technique for supporting offshore drilling and production efforts in the Beaufort Sea. Although ice islands have been successfully used for exploratory drilling, their long term suitability as stable drilling platforms seems doubtful. The use of subterranean caverns is still in the conceptual stage. From an economic standpoint prefabricated drilling platforms appear to be limited to deep water areas of the Arctic Ocean and Beaufort Sea.

It is the intent of this paper to briefly discuss concepts for the design and construction of an offshore gravel embankment utilizing thermal piles in water depths of less than 60 feet. Particular emphasis is placed on the effect of short and long term thermal considerations which could be expected to influence the performance of a man-made island in offshore areas underlain by permafrost as well as in areas free of permanently frozen soil.

BACKGROUND

Climate is the dominating factor in the high latitudes of the Arctic. Mean January temperatures range from -20°F to -10°F while July temperatures average from 34°F to 46°F (Johnson & Hartman, 1969). The mean annual temperature is probably less than 10°F. Typical wind chill values for Point Barrow, Alaska are
around 1650. A freezing index of 8500 is common on the North Slope of Alaska with a 10 year maximum of approximately 9500 degree days. Another consequence of the severe weather conditions is the Arctic ice pack. Often, open water along the coastline is present for only 6-8 weeks each year. Shore fast ice, although less of a problem then pack ice, can significantly influence offshore structure development.

Other environmental conditions which must be considered in the design of offshore structures include sea floor soils, storms and storm tides, ocean currents, and other physical oceanographic parameters. For nearshore structures, coastal influences can be significant. Although a realistic quantitative evaluation of the influence of these factors on a completed structure is very difficult, the effect of the more significant factors on the thermal regime can be reasonably approximated.

**Sea Floor Soils in the Beaufort Sea**

Of particular importance for design of offshore structures in the Beaufort Sea is a determination of sea floor soil properties and the thermal state of the soil upon which an offshore structure will be founded. The influence of frozen state soil behavior (permafrost) on long term performance of such structures is still not fully understood and little documentation is available.

Permafrost is defined as, "The thermal condition in soil of having temperatures below 0°C persist over at least two consecutive winters and the intervening summer" (Brown, 1974). This definition may be misleading for subsea soils because the saline content of the near bottom soil pore water depresses the freezing point so that soil below 0°C may behave as in a thawed-state. The terminology used by Osterkamp and Harrison (1974) is suggested as a modification. "Bonded" permafrost refers to those soils which are ice cemented, frozen, and behave as a bonded mass. "Unbonded" permafrost is soil which is at a temperature below 0°C yet has the characteristics of a thawed soil. There is probably some cyclic annual movement of the isotherms that define the boundary between "bonded" and "unbonded" permafrost and between permafrost and non-permafrost areas.

The existence of offshore "bonded" permafrost may pose significant design questions. Although only nearshore "bonded" permafrost has been confirmed, its existence at other offshore locations cannot be precluded. Figure 1 shows an interpretation of the distribution of subsea permafrost in the Beaufort Sea.

The existence of subsea permafrost in the Prudhoe Bay region has been known for some time and reported by many, including Rogers, et al (1975), Osterkamp and Harrison (1976), Sackinger (1977), Osterkamp and Harrison (1976), Lewellen (1974) and Judge (1974). Figure 2 depicts a profile of offshore permafrost. Permafrost beneath and adjacent to offshore barrier islands has been reported (Alexander, 1975).

Osterkamp and Harrison (1976) report that shallow bonded permafrost near Prudhoe Bay extends offshore to a water depth equal to the approximate average annual thickness of bottom fast ice. Beyond this depth (approximately 5 meters) the bonded permafrost table dips rapidly to 50 meters at a distance of 3 kilometers offshore. Data presented by Sackinger (1977) suggests that bonded subsea permafrost may be present at depths of 100 meters or more at 5 kilometers or further offshore.
Massive ice in offshore soil profiles has yet to be discovered although it may have been recorded near Pt. Barrow (Alexander, 1975) and could exist in submarine pingos which have been reported by Shearer (1971). According to Shearer (1971), the origin and genesis of submarine pingos is unknown. The fact that they exist must be an indication that other formations of massive submarine ice may be present in offshore seabottom soils.

The limited amount of subsurface soil exploration accomplished to date has provided some indication of general soil conditions that probably will be encountered in near shore developments; however, much additional information must still be obtained. Certainly, a detailed soils investigation is required to define subsurface soils and their physical status before design of an offshore structure can begin.

**THERMAL REGIME**

When considering the thermal regime of offshore soils, two thermal boundary conditions are immediately apparent: air temperature and water temperature. Although they are not truly independent variables, they are considered so in this paper. Near bottom sea floor sediment temperatures closely reflect water temperatures; however, with increasing depth, the natural geothermal gradient governs soil temperatures. With these variables in mind, an approach for the use of thermal piles to stabilize the thermal regime of offshore frozen embankments may be formulated. As pointed out earlier, this discussion is limited to embankments in water less than 60 feet in depth.

If near surface bonded permafrost is present three thermal scenarios are possible:

1) The permafrost table may degrade
2) The permafrost table may aggrade
3) The permafrost table may remain stable

Since a large offshore structure would tend to influence thermal equilibrium, scenario three would be implausible. If degradation of the bonded permafrost is unacceptable, the only remaining alternative is to enhance aggradation of permafrost. Since rapid natural aggradation is unlikely, artificial means must be employed. Thermal piles may be the most logical choice for encouraging this aggradation.

If near surface bonded permafrost is absent only two scenarios are possible:

1) Subsurface bonded permafrost will remain absent
2) Subsurface bonded permafrost will be created

It is unlikely that the thermal regime in "unbonded" permafrost or non-permafrost areas will be necessary to maintain unless future discoveries indicate that "unbonded" permafrost is thermally unstable. Thermal piles could be employed to create subsurface bonded permafrost in areas of unbonded permafrost, if desired. (It is recognized that ice lensing and heaving could pose significant problems if unaccounted for in the design process).
Structural Enhancement

When considering construction of an offshore structure, the creation of a frozen, bonded soil bulb using thermal piles may be desirable for several reasons.

For areas underlain by unbonded permafrost and non-permafrost soils:

1) The shear strength of frozen soil may be many times that of identical thawed soil thus resistance to sliding failure is increased proportionally (see figure 3).
2) The creation of a large frozen mass (although some buoyancy forces may be created) adds to overturning resistance by significantly increasing the subsurface area of the failure plane (see figure 3).
3) Development of frozen "bonded" soil will radiate horizontally, thus promoting potential ice lens growth in the vertical direction. This ice lens orientation is considered to be significantly more favorable for limiting vertical frost heave forces.

For areas underlain by "bonded" permafrost:

1) The frozen bulb created could become an inherent part of the underlying bonded permafrost as shown in Figure 4.

Thermal Piles

A thermal pile may be defined as a structural heat pump which utilizes artificial or natural refrigeration to remove heat from below ground locations. Natural refrigeration resulting from cold ambient air temperatures is the only type considered in this paper.

Two basic heat removal techniques are used: the single phase system and the two phase system. The single phase thermal pile employs the heat capacity characteristics of a circulating medium to remove heat. An example is an air convection pile. An air convection pile can be as simple as a closed pipe placed into the soil profile and left partially exposed to the ambient air. When air temperatures near the top of the pile are cooler than soil temperatures, natural convective heat transfer will begin as the cooler, denser air at the top of the pile moves downward, displacing warmer air upward. This warmer air in turn cools as heat gained from the soil is transferred to the surrounding air. Typically, efficiency is low; this disadvantage may be offset by advantages such as ease of fabrication and inherent thermal stability. Air convection piles have been tested (Jahns, 1973) and an experimental air convection pile is shown in Figure 6.

A two phase convective system employs the latent heat characteristics of a circulating medium to remove heat. Phase change is usually from liquid to gas and gas to liquid. Typically a heat transfer medium with a relatively low boiling point is chosen. Ammonia and propane are two examples. The gas (or liquid) is pressurized in a closed system such that phase change will occur when ambient air temperatures cool the gas to some predetermined temperature (i.e. 15°F). When this pressurized system has been placed in the ground the heat transfer sequence is easily envisioned. As ambient air temperatures drop below the "boiling" point of the pressurized gas, condensation occurs which releases
to the air the latent heat associated which that phase change. (The effects of sensible heat are small). The condensate migrates downward until the warmer soil temperatures vaporize or "boil" the condensate. A fixed amount of heat is required to boil the condensate. This heat comes from the soil and is effectively removed as the vaporized condensate convects upward. As the vapor condenses near the top of the thermal pile, the heat is transferred to the surrounding air thus completing the cycle. Two phase thermal piles are relatively efficient; they can be rather expensive to manufacture however. High pressures are normally required to reduce the boiling point of the circulating medium and long term stability has not yet been proven under rigorous field conditions. Several types of two phase thermal piles have been employed to maintain or aggrade permafrost. The most familiar type is probably the vertical support member (VSM) used by Alyeska Pipeline Service Company for support of above ground portions of the trans-Alaska pipeline. (Alyeska, 1975). Figure 7 shows details of this pile configuration. The "Long" (Long, 1973) thermopile (Figure 8) has been successfully used in permafrost areas. Special driven thermal H-piles have also been successfully employed in permafrost. Details of this system are given in Figure 9.

When a thermal pile has been chosen for a certain use, the problem of successfully employing it in a suitable structural design is only half solved. The thermal pile must be installed without compromising its heat removal characteristics or its structural integrity.

Thermal piles are usually fabricated in two ways. The inherent thermal pile such as the "Long" thermal pile shown in Figure 8 and the air convection pile shown in Figure 6 are "single components". Both thermal and structural properties come from one self-contained pile. The "dual component" thermal pile consists of two separate components. The first component is the structural portion of the pile. It is often a conventional pile or, where conditions warrant, a fabricated pile with special structural characteristics (Rooney, Nottingham, Davison, 1976). Figure 7 shows the VSM design used for the trans-Alaska pipeline system and Figure 9 shows a special H-pile design. After the structural component is placed, the second component, the thermal portion, is installed within the structural component to complete the installation of the thermal pile.

Single component and dual component thermal piles are common in the Arctic and numerous examples of their applications are evident. Table 1 provides a general categorization of the various types of thermal piles.

<table>
<thead>
<tr>
<th>Table I - Types of Thermal Piles</th>
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<tbody>
<tr>
<td><strong>Single Component</strong></td>
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<tr>
<td>Single Phase</td>
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<td>1. Air convection pile</td>
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<tr>
<td>2. Thermo-Dynamics</td>
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<tr>
<td>Two Phase</td>
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DESIGN CONSIDERATIONS

The justification for thermal piles in offshore embankments is based upon two considerations:

1) Is the thermal regime critical and must it be maintained or modified through the use of thermal piles?
2) Can thermal piles be used to enhance the structural integrity of the offshore embankment?

For embankment structures located in very shallow water depths where shore fast ice develops, it may not be necessary to utilize thermal piles for structural purposes. Thermal piles may be necessary further offshore where pack ice forces become significant. Design parameters may also require the use of thermal piles for prefabricated platforms.

Construction of Offshore Islands

To date, all of the permanent offshore islands constructed in the Beaufort Sea have been in shallow water. Two successful wells have been drilled from very shallow man-made islands in the Mackenzie Delta. (Cozens, 1976). The first artificial island constructed in the Beaufort Sea, Immerik, was conceived by Imperial Oil. It was built of gravel dredged from the sea bottom and transported to location by pipeline. A second island, Adgo was built by filling a gravel ring with silt and freezing the entire mass in place. The third island constructed by Imperial utilized gravel hauled over 50 miles in conventional trucks and placed through holes cut in the winter ice. (Pimlott, 1974). Sun Oil has constructed offshore islands in this vicinity, but construction details are unknown to the authors. As of June, 1975 eight artificial islands had been constructed in water depths up to eleven feet. (Hnatiuk, 1975). It is postulated that the maximum feasible depth for offshore islands is around 60 feet (Offshore, 1975). No attempts to use thermal piles for stabilization of offshore thermal regimes or structures are known to the authors.

Types of thermal piles and installation techniques used for stabilization of offshore structures will depend on several factors:

1) Is bonded permafrost present?
2) Is the pile to be used for thermal stabilization only or for structural stabilization or for both.

If bonded permafrost is absent, pile installation can be carried out in a conventional manner. Piles can be driven or jetted into place. Displacement or non-displacement piles can be employed. Either "single component" or "dual component" thermal piles may be used. It should be noted that currently available "single component" thermal piles cannot be driven due to low allowable driving stresses. However, they could be jetted into place or, if feasible, installed in oversized predrilled holes. "Dual component" thermal piles are considered the most feasible for installation in saturated cohesionless soils.

If bonded permafrost is present, a "dual component" thermal pile is almost mandatory. Furthermore, unless oversize holes are augered, a non-displacement pile must be employed. This could limit pile choice to an H-pile (See Figure 9) or an open-ended pipe pile.
Driving conventional H-piles in onshore permafrost has generally been successful only if favorable soil and temperature conditions are present. When sustained driving resistance exceeds roughly 100 blows per foot, the structural soundness of the pile can be impaired. Failures of driven H-piles may occur during pile driving if the actual structural capacity of the pile is reduced due to buckling, twisting, or other excessive deformations. However, "dual component" thermal piles utilizing a hybrid H-pile have been successfully used in Interior Alaska (Rooney, Nottingham, Davison, 1974). Blow counts of up to 1000 blows per foot with moderate hammer energy have not damaged this type of pile or precluded penetration to required tip elevations.

If open-ended pipe piles are employed as non-displacement piles, they must continually be augered out to prevent a soil plug from forming in the end.

The type of thermal pile chosen will depend upon the existing subsea soil conditions. "Bonded" permafrost which can be expected nearshore will present the most difficult problems. It will probably be necessary to preserve the bonded state of the permafrost which implies a dual component thermal pile, probably driven in place. Installation of the thermal piles could be the most difficult phase of construction. However, since near surface "bonded" permafrost is currently only known to exist in shallow water, excessively massive structures would not be required as ice forces would be relatively low when compared to further offshore.

Further offshore and in other areas of "unbonded" or non-permafrost areas, pile installation may not be as difficult and a wider choice of thermal piles is available. The function of the thermal piles may be more critical in these areas as they would provide the basis for the structural resistance to massive ice forces. Construction timing would be even more critical as construction and stabilization would probably be required in as little as six months. Initial freezing of the island could be hastened with artificial refrigeration. It may be necessary to use drag lines or a dredge through the first winter to prevent significant ice forces from developing against the island.

Temperature stress and strain instrumentation would be required for the first offshore embankments utilizing thermal piles. Of course, thermal behavior, strength parameters, and other properties of the embankment material as well as the underlying soil would be required. The state-of-the-art of offshore soil drilling and sampling in offshore areas of the Arctic is reasonably well developed. No major problems are foreseen in the acquisition of the required design information.

CONCLUSIONS

The first small steps toward the large scale employment of offshore embankments to support oil and gas exploration in the Beaufort Sea have been taken. Commercial production of these reserves will begin in the near future and future exploration and production will evolve further offshore in the Arctic Ocean. Offshore embankments stabilized with thermal piles provide one possible means of moving into water depths as great as 60 feet. The design, technology and construction techniques are currently in existence.
Bibliography


Figure 1 Estimated Distribution of Subsea Permafrost in the Beaufort Sea (After Environmental Impacts, 1977).

Figure 2 Subsea Permafrost Profile in Prudhoe Bay, Alaska (After Osterkamp & Harrison, 1974).

Figure 3 Schematic of an Offshore Gravel Structure in Unbonded Permafrost or Thawed Soils, Stabilized with Thermal Piles.
Figure 4 Schematic of an Offshore Gravel Structure in Bonded Permafrost Stabilized with Thermal Piles.

Figure 5 Single Phase, Single Component Experimental Air Convection Pile (After Jahns, 1973).

Figure 6 Two Phase, Dual Component Thermal Pile (After Alyeska, 1975).
Figure 7 Two Phase, Single Component Thermal Pile (After Long, 1973).

Figure 8 Two Phase, Dual Component Thermal Pile (After Rooney, et. al., 1976).
CONSTRUCTION OF A DEEP-SEA DOCK IN THE ARCTIC

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BACKGROUND

The establishment of the first deep-sea dock in the Canadian Arctic originated with a submission of a proposal by an international mining company to the government of Canada, to develop a lead and zinc mine at a place now named Nanisivik (an Inuit/Eskimo word meaning a place where people find things) located on the south side of Strathcona Sound, Baffin Island latitude 73° north, longitude 84.5° west (see plate 1).

Aside from a mine and its associated crusher, concentrator, conveyor, storage and tailing disposal systems, the proposal included the development of a deep-sea dock, a town, an airport and highways, including a highway between Nanisivik and an existing Inuit settlement of Arctic Bay, approximately 15 miles west of Nanisivik.

To realise the development, estimated to cost 60 million dollars, the mining company requested the participation of the government of Canada in the development. Pursuant to its policy to encourage development of its northern territories the government of Canada agreed to provide funds required to construct the airport, highways, town and the dock, estimated at 18 million dollars. In return the Government of Canada will have an equity, amounting to 18%, in the mine.

The Department of Public Works of Canada is responsible for the design and construction of the dock, to be completed by this current year (1977).

REQUIREMENTS

The construction of the dock included the provision of the following facilities (see plate 2).

(a) a dock of approximately 250 feet in length and a berth with a minimum depth of water of 40 feet below Chart Datum for a minimum length of 600 feet either side of the centreline of the dock face;

(b) A land fill area of 325 feet by 125 feet and a 75 ft. wide access corridor, including a 15 ft. wide conveyor belt section, appropriately graded to a minimum elevation of 16 feet above Chart Datum at the dock face and completed by October 31, 1976;

(c) two springline bollards on each of the two outer cells, 2 breasting bollards to the rear of the dock head and 2 mooring bollards further inshore as shown on the plan and designed to withstand line pulls imposed by a 50,000 DWT vessel berthed at the dock in maximum storm conditions occurring in the area;
(d) Fendering designed for:

1) absorption of kinetic energy of a 50,000 DWT vessel berthing at the dock at a velocity of 9”/sec. and an angle of up to 10° with the dock face;

2) transmission of reaction forces to the cells consistent with their strengths;

3) stability against forces imposed by the vessel longitudinally to the dock face, and

4) to provide a buffer to keep the vessel clear of the cells including any bulges that may result in the cells from fill.

(e) a kerb along the inside of the periphery of each cell;

(f) a reinforced concrete decking on each end cell by March 31, 1977 following settlement of the cell fill;

(g) lighting of the dock structure to a minimum level of illumination of 3 to 5 footcandles and adequate electrical power points.

SITE CONDITIONS

The land is hilly, rugged, barren and covered with talus varying from fine material to large rounded boulders. The reason is that Nanisivik is located in the zone of past and active glaciation and continuous permafrost with extreme cold conditions and little precipitation comparable to that of deserts of the tropics. From around the end of April to about the middle of August the sun never sets. In winter time, from about the end of November to approximately the beginning of February, there is almost total darkness. The annual mean temperature is -14°C with a high of 10°C in summer and low of -40°C in winter. The average freezing index is 5000 degree days and the thawing index is 450 degree days in Centigrade system. Precipitation is less than 6 inches per annum.

The bedrock principally consists of shale near the dock site, dolomite approximately 3 miles up the hills at the mine area and sandstone and quartzite several miles further up the hills towards a site selected for the airport with an extensive cover of talus everywhere, produced by the action of freeze-thaw. In several areas there are relic glaciers underground surfacing close to the ground level.

The dock area which consists of a beach and a body of water extending to approximately 350 ft. offshore is demarcated from the land area by a cliff varying from a few feet to approximately 25 feet in height. The beach slope is approximately 1:15. Beyond a depth of water of approximately 15 ft. below the low water datum the sea-bed slope suddenly steepens to 1:2.

The beach material consists of fine angular material, gravel and some shale chips. Below water the soil consists of fine sand mixed with fine gravel and shale and is generally uniform. On the land side the permafrost table is approximately 3 ft. below the ground level. Below water the permafrost table appears to dip at approximately 45° (see plate 3).

Sufficient wind, tidal and currents data now exist collected in recent years at Nanisivik. Much of the information is, however, still being analysed. The wind data required for the dock design were obtained from wind records of Arctic Bay. The tidal and currents data used in the design were based on short term observations at the dock site.
Arctic Bay wind records suggest that winds occur from the northwest 20%, from the north about 15%, west 12% and south 10% of the time. Personal observations made during the construction period indicate that most storms occur from the south and west of Nanisivik. In addition, while strong winds may be occurring inland on the hills little or no wind activity may be felt at the dock site.

Because of protection from hills against winds, short fetches varying from 3 miles from north to 30 miles from west, restricted width of the sound and calmer summer wind climate, the wave activity at the dock site is generally calm to small. The wave climate hindcast from the Arctic Bay winds however showed the occurrence of waves of up to 8 feet high from the west and this wave height was adopted in the design.

The tides occurring at Strathcona Sound are mixed semi diurnal. The highest tide observed prior and during construction was 9.5 feet above the low water datum.

The currents occurring in the sound are mainly tidal confined mainly to the upper 25 ft. layer of water. Their magnitudes are small, rarely exceeding 0.5 knot.

The most interesting phenomenon is the occurrence of ice on the sound between October to July giving complete access from one area to another separated in summer by water. The ice sheet simply rides up and down on the water with the tides like a huge tailor made pontoon except in shallow water along shore where hinges are formed with the non-floating or partially floating ice. The ice begins to form around October. By the end of November the ice thickness is 1.5 ft., adequate to carry truck traffic. By March the ice thickness reaches over 5 ft. when heavy machines can be deployed on the ice. The maximum thickness of ice measured was 6.5 ft. in May.

DESIGN

The discussion under design will be limited to functional and structural design.

The functional characteristics of the dock, such as the basic dimensions of the dock, were dictated by the expected shipping, physical conditions of the site, loading and unloading requirements and functions of the dock as a public dock.

The shipping requirements dictated that the dock have a minimum length of approximately 200 ft., a berth length of 1,200 ft., minimum depth of water of 40 ft. and 8 mooring points to secure 4 springlines, 2 breastlines and 2 fore and aft lines. Close proximity of deep water and mobilization costs of dredging plant ruled out dredging. Instead the dock was projected to the required depth of water. Loading and unloading considerations required a 15 ft. corridor for conveyor belt system and a 30 ft. access corridor. The access corridor was eventually widened to 60 ft. to allow for possible future other uses of the dock.

Feasible types of structures for the dock were reviewed and the three types of structures considered were: (1) a multiple point mooring arrangement with 2 floating buoys and 3 onshore bollards, together with a longboom shiploader to load ships, (2) a steel DeLong dock consisting of 10 steel caissons, supporting a steel barge 200 feet long by 60 feet wide by 10 feet deep, at a height above the influence of ice action, and, (3) circular steel pile cells.

Consideration of costs, the desirability of a solid dock to avoid double handling of cargo through barges and the existence of a similar structure at Deception Bay in somewhat similar environment led to the adoption of the alternative using circular steel pile cells. This alternative composed of 3 circular 70 feet diameter steel pile cells filled with angular material with an adequate backup area and a causeway protected against washout with
armour stone, together with associated facilities detailed earlier in this paper. The cells were to be spaced at 125 feet center to center aligned in one straight line and constructed using Arbed PBP 12.7 straight web steel piles 80 feet in length and 0.5 inch thick conforming to CSA G40.20 and G40.21, 38T. The minimum interlock strength in direct tension was to be 16800 pounds per linear inch.

The structural analyses made included overturning (factor of safety greater than 2), sliding (factor of safety 1.50) interlock tension (factor of safety greater than 2) and checks against rotational slip failure. The analysis to investigate rotational slip failure was carried out with ICES-LEASE I computer program which is based on Bishop Method. The minimum factor of safety found against slip failure was 1.50 and was considered to be acceptable subject to confirmation of the angle of friction of 35° used in the analysis, with further test boring and laboratory analysis, prior to commencement of construction.

The final design of the cells included provision of 1.5 ft. thick reinforced concrete deck on the outer two cells and guard rails along the inside periphery of all three cells.

The fendering required under the design composed of three sets of large diameter used rubber tyres on each cell, each set made up of two tyres bolted together to from a double fender. The fenders are suspended from the guardrail.

CONSTRUCTION

The first thought on construction, was the conventional technique with floating plant. The very short open water season of the Arctic coupled with cost and practicability of moving floating equipment to this remote site quickly dispelled this idea. Several ice experts were consulted and a scheme was devised to build the dock using the ice as the platform. This meant literally cutting the ice to build the dock. The initial advice was that the thickness of ice occurring at Strathcona Sound was not adequate. Plans were therefore drawn up to thicken the ice to the recommended thickness of 9 ft. Elaborate templates were designed to drive and protect the cells during the construction period while they remained unfilled. The materials required for the cells and templates were shipped during the preceding shipping season prior to the winter of 1974/75 when the construction of dock was scheduled. Except for pile driving and ice cutting equipment and certain lighter materials required for the construction, all equipment and material were brought on site prior to contract award. The only thing that remained was selection of a suitable contractor to carry out the construction work.

Invitations were issued to several contractors to tender for the construction work. The contractors were asked to submit an estimated cost for the total dock work.

All materials, equipment, consumable supplies, power, water, fuel warehousing, transportation between Resolute Bay and Nanisvik, room and board, first aid and communication services were to be made available to the contractor at no cost.

Response to the invitation to tender was good. Three contractors were selected from a list of several to discuss the methods proposed by them for the dock construction. All the three contractors displayed equal capabilities to carry out the dock work and the lowest bidder was selected for the work.

The discussions with the contractors and a further evaluation of the conditions existing at Nanisvik indicated that ice thickening would not be required. Also, the soil conditions occurring at the site appeared to be better suited to the use of a vibratory type pile driver compared to the impact pile driver originally envisaged. In consultation with the successful contractor, a Foster 2-40E electrical vibratory type pile driver/extractor was selected.
for the driving of the dock cells.

The dock construction work commenced on March 13, 1975, within only a few days of the award of contract. One of the initial tasks was to set out the dock lines on the ice using previously established land based survey points. Once the cell locations were marked out, holes were drilled through the ice, at the seaward face of the cells to verify the availability of the required depth of water. The soundings revealed certain inconsistencies in the original sounding survey. The sounding survey was therefore extended to cover the whole dock area resulting in on the spot adjustment of the dock location.

Observation of the ice did not reveal any measurable lateral movement of the ice. To install the cells it was considered that a ring of ice, approximately 5 ft. wide, could be cut out from the ice sheet for each cell. The circular slab of ice left as a result of removal of the ice ring was utilized to provide lateral support for the threading and driving of the cell. This simplified considerably the template requirement reducing it to a simple guide ring mounted on piles, as a second template above the circular slab of ice.

Once the basic decisions were made the work began on various aspects simultaneously. The contractor had only 6 men as the work team for the cell construction. Two men were employed on the fabrication of the guide ring using 5" x 5" steel angles. Other two men were employed on cutting ice for the installation of eight H piles for the guide ring, around the periphery of the cell. The remaining two men hauled the steel piles on the ice close to the cell location. While the dock contractor was proceeding at full speed on the dock work the mining company was preparing a landing strip, on the ice, 8000 ft. in length, for a Nord Air Boeing 737 chartered to transport the vibratory hammer from Pittsburg, U.S.A. Test drilling required to verify the soil conditions assumed in the design was also in progress at the same time.

The vibratory hammer arrived as soon as the holes for H piles were prepared. The actual pile driving time for each of the 8 piles, which were driven approximately 15 to 20 ft. into the sea bed, was no more than 15 to 30 seconds. The guide ring, which was fabricated to 70 ft. diameter, was cut into four arcs for installation on the H piles, and rewelded and fastened on brackets welded on the H piles (see plate 4). As the guide ring was being installed two concentric circles were being cut with the ice trencher to remove a ring of ice of approximately 5 ft. wide around the outside periphery of the cell. Radial cuts were made across the two concentric circles to cut up the ice into blocks of approximate 5 ft. x 5 ft. The cuts made in the ice were stopped short of the full depth of ice to avoid premature flooding and refreezing. When all the cuts were made the ice was broken up into blocks by an ice pick fabricated on site with steel sheet piles, and removed with a clam bucket mounted on a 60 ton crane. All ice removed from cuts was bulldozed away from the working area, and cuts cleared of ice were kept continuously clear of refrozen ice (see plate 4). The refreezing that took place in the cuts amounted to 1 to 1.5 inches per day.

The next step was to commence the threading of the steel piles to form the cell prior to driving. The first pile to be lowered was the center pile on the seaward face of the cell, using the 60 ton crane. This was the only crane available at the time with a long enough boom to handle the 80 ft. long piles. Prior to threading, the pile interlocks were thoroughly cleaned. The threading was accomplished by raising the pile to match its lower end with the top end of the preceding pile, manually threading the interlocks, allowing it to gently drop to the ice level to permit men at the ice level to install timber guides to hold the pile against the templates and then letting the pile drop freely to the sea bed. The threading was closely monitored to accomplish easy closure of the cell with the last pile. The first cell was fully threaded within a month of the start of the dock construction.
The vibratory hammer obtained for the pile driving proved to be very effective. The piles were driven in rapidly and with no major problem. The piles were driven in sequence such that tops of piles being driven were not lower than approximately 5 ft. below the top of the adjoining pile. Some problems were encountered in driving the landward piles of the west cell and tops of some of them had to be cut off to conform with the top elevation of the cells.

The front sections of the cells were overdriven in a staggered sequence to achieve a minimum penetration of over 20 ft. The gaps, which were arranged to be above the ice level, were made up with pieces of steel piles, welded on to the main piles with plates.

The ice cutting, the pile threading and the pile driving work was completed by around the second week of May 1975.

The next major operation already started on completion of the driving of the first cell, was the filling of the cells and the construction of the causeway and the backup area. Beach material was used for the cells. The material required for the causeway and the backup area was obtained from upshore areas. Two scooptrams, two D8 scrapers, two loaders and two dozers were employed round the clock to carry out the fill work. Over 100,000 cubic yards of material was moved to complete the fill work. The armour rock required to protect the fill areas was obtained by blasting from near the mine site.

All dock works except for concrete capping, bollard work, permanent fendering and electrical work were completed by mid August 1975 (See plate 5).

The first ship docked at the dock on July 16, 1975.

To allow for settlement of the cells and to integrate the dock concrete work with concrete works of other projects, all dock concrete work was deferred to the 1976 construction season.

Settlement of the cells was closely monitored. Lateral movements of the cells were also closely watched. By 1976 the vertical settlement that was recorded amounted to approximately 6". The maximum lateral movement measured was 16" seaward. This can be attributed to realignment of the cells due to consolidation of the fill inside and behind the cells.

A program to measure vertical and lateral forces imposed on the cells by the ice was also carried out during 1975/76, the details of which are to be discussed in a separate paper at this conference.

The concrete work was commenced in July 1976 when no further movements of the cells were recorded. The concrete mix used was as follows:

<table>
<thead>
<tr>
<th>Batch</th>
<th>8 cu. yds.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregates</td>
<td>11,400 lbs.</td>
</tr>
<tr>
<td>Fine aggregates</td>
<td>10,200 lbs.</td>
</tr>
<tr>
<td>Portland cement</td>
<td>4,800 lbs.</td>
</tr>
<tr>
<td>Water</td>
<td>215-240 gallons</td>
</tr>
</tbody>
</table>

The concrete work was begun with bollards and completed with the decking of the east cell and the provision of a ring beam in the center cell. The east cell deck and the center cell ring beam concrete work was carried out as a winter work. The aggregates and water were preheated. Up to 2% by weight of cement calcium chloride was added to the mix. To avoid freezing shelters were erected around the cells using 4 ft. x 8 ft. plywood boards.
A minimum temperature of 50°F was maintained in the shelters for at least 3 days after completion of concrete pouring.

**COSTS**

The estimated cost of the dock project was 3.8 million dollars. When all the final payments have been made the actual costs are not expected to exceed 2.75 million dollars.

**CONCLUSIONS**

The use of ice as the working platform provided unlimited working area and considerably simplified the dock construction work. Simplification of working methods and procedures, elimination of unnecessary requirements, good working relationships, immediate response to problems and an excellent work crew of the contractor helped in the speedy completion of the dock project. The severe climate of the North presented no major problem. In fact on this particular project, it provided a natural aid to a speedy and economical form of construction.
MOHR ENVELOPE

NORMAL EFFECTIVE STRESS (PSI)

LEGEND

LOOSE TO COMPACT SILTY SAND
DARK GREY
BLACK FINE SAND, VERY LOOSE
DARK GREY FINE TO COARSE SAND,
SOME GRAVEL, TRACE TO SOME SILT
LOOSE TO COMPACT
PERMAFROST

NOTE: FOR BOREHOLE LOCATION SEE PLATE 2
REMOVAL OF A RING OF ICE TO THREAD THE STEEL SHEET PILES

GROOVES OF EACH PILE WERE THOROUGHLY CLEANED PRIOR TO THREADING
A SIDE VIEW OF THE FULLY THREADED CELL. THE PILE TOPS DEPICT THE SLOPE OF THE SEA BOTTOM. THE PICTURE ALSO SHOWS ICE CUTTING IN PROGRESS FOR THE NEXT CELL.

AN AERIAL VIEW OF THE DOCK
INTRODUCTION

A desirable design criterion for an enclosed harbor is that the channel connecting it with navigable waters be self-maintaining. This condition may prevail where sediment movement is negligible, or, in the case of moving sediment, where tidal or river discharge is sufficient to maintain acceptable channel dimensions. A method to predict the stable configuration of a navigation channel connecting open tidal-waters with an enclosed harbor is presented in this paper. The stable cross-sectional area, cross-sectional shape, and bottom elevation of the channel are considered. A relationship between these variables and the water discharge through the channel is determined using the geometric characteristics of nearby natural channels and the hydraulic regimes that sustain the channels.

An example is given using data obtained from a navigation channel at the harbor of Dillingham, Alaska, and from natural drainage channels on a tidal flat and in rivers near Anchorage, Alaska. The resulting relationships may be used when sediments are like those on northern tidal flats, i.e., highly compacted glacial silt and mud-sized material generally lacking in clay minerals and organic constituents. However, using appropriate field data, the method may be applied to the design of a navigation channel in any region where natural tidewater drainage channels exist.

BACKGROUND

Two widely used procedures for estimating channel cross-section dimensions prior to construction are the tidal prism relationship, applied to sandy tidal inlets, and the regime theory, mostly applied to river channels. A tidal prism is defined as "the total amount of water that flows into a harbor or estuary or out again with movement of the tide, excluding any freshwater flow" (Allen, 1972). The relationship of cross-sectional area with tidal prism was reviewed by Jarrett (1976) who notes LeConte, in 1903, first used the relationship in the form

\[ A = CP \]

for Pacific Coast inlet gorges in which \( A \) = cross-sectional area, \( C \) = constant, and \( P \) = tidal prism. O'Brien (1931) first developed a power function form of the relationship

\[ A = CP^n \]
in which \( n \) = exponent of \( P \). Jarrett (1976) found \( A \) values for Atlantic coast inlets to be greater than those on the Pacific coast. Among the contributing factors, he cited a difference in wave climate, with the higher waves occurring along the Pacific coast. He hypothesized the amount of littoral material entering an inlet would be greater there, and consequently the average flow velocity required to maintain Pacific coast inlets would probably be greater. A conservation of water mass would dictate a smaller inlet cross-sectional area in this situation. In addition, Jarrett also cited differences in the characteristics of the astronomical tides, and errors introduced by computational procedures in determining the tidal prism, as possible causes of the difference in Atlantic coast and Pacific coast \( A/P \) ratios.

At least three different procedures have been used to determine the tidal prism. O'Brien (1931) and Johnson (1972) calculated \( P \) by multiplying the surface area of a bay by the tide range at the entrance to the bay. They assumed a uniform rise and fall of the tide over the entire bay. This procedure results in larger tidal prism values than the cubature method (Jarrett, 1976) in which variations in the bay tide range are considered. A third procedure is the NOS current data method (Jarrett, 1976). It consists of current measurements made at a vertical station near the inlet throat, with continuous observations during one or more tidal cycles, preferably spring tides. Velocities are reduced to an average velocity representative of the throat cross-section which is then converted to discharge and integrated over a flood or ebb portion of the tide to get the tidal prism.

The regime method, in which a channel in regime is defined as having no net erosion or deposition over a hydrological cycle, has been widely used since Lacey, in 1929, first proposed it as a basis for the design of irrigation canals. Regime equations relating to channel cross-sectional geometry are essentially of the form

\[
L = rq^s
\]  

(3)

in which \( L \) = some length characteristic of the cross-section, \( q \) = some characteristic unidirectional discharge, and \( r \) and \( s \) = constants. A number of investigators have applied regime concepts to tidal flow conditions and Chantler (1974) summarized and re-analyzed some of the data. The characteristic discharge used by the different investigators varied, but was usually some form of "dominant" or maximum discharge.

**APPROACH**

The objective of this paper is to present a method by which the stable geometry of a channel connecting an enclosed harbor and navigable waters may be predicted. Because of the complex nature of the problem resulting in part from varying tide ranges, quantities of sediment in transport, and channel bank and bed material from one location to another, theory does not exist to cover all circumstances of channel design. The alternative is some empirical procedure which uses data from existing nearby channels. Past studies indicate a relationship between channel geometry and some measure of the flow through the channel will be a useful approach.

Two problems arise in using such an approach. The first comes in choosing a characteristic cross-section. The other concerns the choice of a flow parameter. In areas of high tidal range the cross-sectional geometry will likely vary as the slope of the banks and bed of channels vary, especially for tidal flat channels. The procedure used should, therefore, be adaptable to any cross-section. Two flow characterizations have been used in the past. An instantaneous flow, i.e., fluid dis-
charge, is considered in the regime theory. The tidal prism uses the total volume of water which passes through the cross-section during a specified time period. In general, the latter flow parameter will be more useful because it can be calculated using data obtained from tide and river discharge records and topographic information such as may be available on natural channel dimensions. Data on the maximum channel discharge, especially for tidal channels is frequently unavailable.

Formulation of Method

Channels are assumed to be created and sustained by one or more of tidal discharge to or from a harbor basin, river discharge, tidal discharge through a natural channel from regions at a higher elevation, and tidal discharge downslope off a tidal flat. Channel geometry is only considered a function of currents created by the rise and fall of the tide, or by river discharge. Wave scour is not considered.

It is further assumed that a relation of cross-sectional channel area to water discharge through a harbor channel, during the ebbing portion of a tide, will fall on the curve of the relationship of ebb discharge through natural channels cut in the same material at nearby locations. The ebb portion of the tidal cycle is used because at that time river and tidal discharge are additive, and tidal-flat flow is downslope into the channels.

The total water volume moved through a harbor channel or a natural channel during the ebbing portion of a tide, Q, was found to produce the best relationship with channel cross-sectional area where

\[
Q = \int_{t_h}^{t_l} (q_t + q_r) \, dt
\]

in which \(q_t\) = ebb tide discharge of water through the channel cross-section resulting from the previous flood discharge up the channel and/or on a tidal flat, or from the rise in channel stage caused by the backwater effect of river discharge; and \(q_r\) = normal river discharge, if any, that would occur if the tide was not present, \(t_h\) = time of high water, and \(t_l\) = time of low water.

The determination of \(q_t\) and \(q_r\) in channels may be accomplished using existing tide gage and river stage data and natural channel dimensions. The harbor basin discharge is equal to the water volume difference in the harbor between high and low tides. It is calculated using the proposed harbor dimensions and the tide range at the harbor site.

In intertidal regions it is assumed water moves into the channels from the adjacent planar portions of the tidal flat only during an ebbing tide and only when the vertical distance between the water surface and the tidal flat is small. It is further assumed that tidal flat flow is downslope only, and thus conforms to the topography of the tidal flat (usually a combination of seaward and channel-directed slopes). Flow on the tidal flats thereby adds to the channel discharge. Because of the variable topography of tidal flats, and of tidal flat channels, the discharge must be obtained from cross-sections and contour maps of the tidal flats. The volume discharged through a channel at any elevation on the tidal flat is, therefore, assumed equal to the volume under a horizontal plane at an elevation of the boundaries of the tidal flat drainage at the channel location, and bounded by
downslope flow streamlines which intersect the channel above the cross-section being studied. This is illustrated in the example provided in the next section.

The tidal discharge volume for river channels is computed using the water volume measured between high water elevation and low water elevation throughout the tidal reach of the river above the cross-section. It is the emptying of the tidal prism upstream of the section, plus the river discharge, and is obtained using topographic data for the river from the site of observation to the head of tidewater plus river discharge and tide data.

**EXAMPLE**

A harbor navigation channel at Dillingham, Alaska; two streams, Eagle River and Ship Creek near Anchorage, Alaska; and a series of tidal-flat channels at Anchorage, were studied to determine whether the cross-sectional area/ebb discharge relationships are similar for these features. Sediments through which the channels flow are similar in each region, and composed of well-compacted silt and mud-sized glacial-source material with < 2% clay minerals. Channel and stream bank elevations were in all cases < 1 m above mean higher high water.

**Dillingham Harbor**

Dillingham Harbor is a "half-tide" type harbor (Everts, 1976). Because of currents resulting from the high tides, as well as severe winter ice conditions, "half-tide" harbors are constructed as enclosed basins sited adjacent to, rather than within, navigable estuaries. Harbor depths are generally specified near mean lower low water (MLLW) to reduce initial excavation costs. The unique feature of these land-contained harbors is a sill at an elevation above the harbor bottom and across the navigation channel where it enters the harbor. The sill provides flotation for vessels during low tide stages while it restricts navigation into or out of the harbor to times of higher tidal elevations.

Dillingham is located 500 km southwest of Anchorage, Alaska, in the upper reaches of Nushagak Bay (Fig. 1). The local tide range is 6 m and suspended sediment concentrations vary from 50-1500 mg/l of fine-grained material (1-100 microns). Local and transient fishing boats and commercial barges to 15-m long use the harbor. The harbor basin area is 21,500 m² at the project elevation of + 0.6 m (MLLW). The sill elevation is + 1.8 m (MLLW), and sidewall slopes are 1:5. Moorage is provided for 140 boats. Sedimentation has averaged almost 2 m/yr since the harbor was constructed in 1961. Nearly continuous dredging during the ice-free season is now required to keep it in use.

Flow into and out of the basin is almost entirely tide-dominated with negligible inflow from Scandinavian Creek. The channel is parabolic in shape (Fig 2) with a top width near the basin of 45 m. At low tide, as shown in the figure, the channel is nearly dry, while near the time of high tide, the channel is bankfull near the entrance to the basin (Fig 1). In this region an average 1.3 X 10⁵ m³ ebb tide prism passes through the channel cross-section, which has a bankfull area of about 160 m².

**Anchorage Tidal Flats**

Channels were studied on a tidal flat in Knik Arm, the estuary serving the Port of Anchorage at the northeastern end of Cook Inlet (Fig 3). Knik Arm at Anchorage carries an average suspended sediment concentration of about 1300 mg/l (Everts and
Moore, 1975), and has a median tide range of 8.8 m. Near Anchorage the tidal flats are shore-connected, composed primarily of silt-sized material, and variable in width with a maximum extension of 600 m into the Knik Arm channel.

Flow in the tidal-flat channels occurs primarily when the tide is ebbing. During the flood portion of the tide, water rises over the tidal flat as a sheet, and is not carried preferentially in the channels. Figure 4 is an aerial view of the study area at Anchorage. The arrow identifies the tidal flat drainage region shown in Figure 5. Contours in Figure 5 are given in feet rather than meters because the original topographic computations were from aerial photographs in feet with associated ground surveys also in feet. Figure 5 also shows cross-sections at 2-ft contour intervals extending from one side to the other of the drainage region.

For computation purposes, channel areas were calculated to be the area enclosed by the channel walls up to the obvious break in slope, and extending upward to the contour elevation at the width of the break. On the cross-section of the 22 ft contour (Fig 5) the break occurs at 20.5 ft elevation and the width is 35 ft. The cross-sectional area of the channel is 156 ft² (14.6 m²). This is the area through which the discharge is assumed to pass.

Water discharge through the channel becomes important when the water surface elevation declines to the elevation of the cross-section (Fig 5). The ebb-tide discharge through the channel is then calculated to be that volume of water which is upslope from the cross-section. It includes the volume in the channel and above the channel break, and the volume on the tidal-flat surface that may be expected to move to the channel at or landward of the channel section. The volume is computed by drawing orthogonals to the tidal flat contours at the cross-section which intercept the channel at its break (Fig 5). Generally the easiest way to get the volume is to sum the volumes of thin horizontal slices in the upslope volume. This volume for the channel at the 22 ft cross-section was 8,800 ft³ (250 m³). The area - ebb tidal discharge relationship is shown in Figure 6. The contour at 4 ft is omitted because of the bichannel condition.

Eagle River and Ship Creek

These rivers are tidal in their lower reaches. Eagle River (Fig 7), which originates at Eagle Glacier, carries a heavy suspended load at a mean summer discharge of 30 m³/sec. Ship Creek carries much less sediment at an average summer discharge of about 3.2 m³/sec. In calculating the water discharge from these rivers during an ebb cycle, the average river discharge was added to the discharge resulting from the water volume upstream of the cross-section caused by the previous flood tide. This volume, calculated using survey data, hydrographic charts, and 1:80,000 topographic maps, is equal to the difference in river channel volume at high tide, and the volume at low tide. The results are in Figure 6. In Eagle River the total ebb-tide discharge includes 70 percent from tidal causes and 30 percent from river discharge. The ratio for Ship Creek was 64 and 36 percent, respectively. Eagle River, as shown in Figure 7, has a particularly extensive tidal reach near its mouth.

Results

The cross-sectional area/discharge relationship of the channel at Dillingham Harbor (Fig 6) closely approximates that of natural channels of similar material in a region where the tidal hydrograph is similar in shape, if not in amplitude.
Harbor and river channel cross-section shapes appear to vary according to the hydraulic regime active within them. Channels, such as those on the tidal flat which carry less than $6 \times 10^2 \text{m}^3/\text{ebb cycle}$ were V-shaped, and the channel area is

$$A = \frac{d^2}{z}$$  \hspace{1cm} (5)

in which $z = \frac{d}{x}$ (sidewall slope), and $d =$ channel depth. Thus the depth of the channel is

$$d = \sqrt{Az}$$  \hspace{1cm} (6)

When $Q > 1.4 \times 10^3 \text{m}^3/\text{ebb cycle}$, the channels were parabolic (concave-up sidewalls) and as the discharge increased the channel bottom became slightly broader and flatter. The cross-sectional area of a parabolic channel is

$$A = \frac{2}{3} wd$$  \hspace{1cm} (7)

and the depth is

$$d = \frac{3}{2} \frac{A}{w}$$  \hspace{1cm} (8)

in which $w =$ channel width.

Channel bottom elevations can be approximated using discharge-dependent width/depth ratios ($w/d$). The $w/d$ ratio was < 10 (ave = 6) for discharges of > $80 \text{ m}^3/\text{ebb cycle}$, and > 10 (ave = 25) for discharges < $80 \text{ m}^3/\text{ebb cycle}$. Bottom elevation may be approximated using these values only for the region above mean lower low water. The depth to which a channel will be maintained by tide-caused flow, unless the river discharge and tide flow is larger than about $10^6 \text{ m}^3/\text{ebb cycle}$, is at or just slightly below MLLW. Ship Creek and the Dillingham Harbor Channel bottom depths, at the MLLW shoreline of Knik Arm and Nushagak Bay, respectively, are about -1 m (MLLW). The depth of Eagle River at its estuary mouth is unknown.

**DISCUSSION AND CONCLUSIONS**

The best-fit linear equation to the data on Fig 6 is

$$A = 0.56 Q^{0.52}$$  \hspace{1cm} (9)

in which the cross-sectional area, $A$, is given in $\text{m}^2$, and the total discharge, $Q$, through the channel during an ebb-tide cycle is given in $\text{m}^3$. The relationship shown in Fig 6 appears to be a useful tool in the design of self-maintaining navigation channels between an enclosed harbor and navigable waters. When the relationship is established for an area using data from natural channels (tidal-flat channels and the tidal portion of river channels), design tradeoffs between harbor dimensions (volume) and channel size are possible. Navigation channels should be short enough that appreciable phase lag and modification in tidal amplitude does not occur as the tidal discharge enters the harbor. Short channels will reduce construction costs, and will be less likely to become temporary sediment storage areas if suspended sediment is deposited during times of low current speed.

The method closely follows the area-tidal prism method used for tidal inlets (Jarrett, 1976). Figure 6 shows the difference in the area-discharge relationship.
between channels in sandy tidal inlets and channels in cohesive, fine-grained, glacial-source sediment in Alaska. For ebb-tide discharges of $< 10^7 \text{ m}^3$ the channel area for sandy tidal inlets carrying a given discharge is less than that for channels in Alaskan sediment. The difference probably arises because of sand brought into the inlets and deposited there by longshore currents. The sediment size difference is also probably a factor, as is the difference in channel depths relative to MLLW.

Channel dimensions should be approximated for the location where the channel meets the basin. This is where the channel depth will probably be smallest if longshore transport is not a problem.

A stream through the harbor may increase the discharge and allow a larger navigation channel. But, based on the river discharge/tidal discharge ratios of Eagle River and Ship Creek (about 3/7) this influence may be slight. Clear water inflow from a stream may also dilute the amount of suspended sediment in the harbor and decrease sedimentation within the basin.

ACKNOWLEDGEMENTS

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Figure 1. Location map of Dillingham Harbor, Alaska, showing the three elements which comprise a half-tide harbor: basin, sill, and navigation channel.

Figure 2. Navigation channel at Dillingham Harbor. View is toward Nushagak Bay from a location near the sill (Fig 1).
Figure 3. Location map of Anchorage, Alaska, study area.

Figure 4. Aerial view to the southwest of the tidal flat study area at Anchorage, Alaska. Arrow identifies channel shown in Figure 5.
Figure 5. Plan view of tidal flat drainage, and cross-section of the drainage region, at the study location shown in Figure 4.
Figure 6. Channel area–water discharge relationship for certain Alaskan tidal channels.

Figure 7. Mouth of Eagle River. The aerial view is east from Knik Arm.
CONSTRUCTION OF PIPELINES BETWEEN THE CANADIAN ARCTIC ISLANDS

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INTRODUCTION

Intensive exploration over the past decade has shown that the Sverdrup Basin in the Canadian Arctic Islands contains rich deposits of hydrocarbons. The most promising reserves that have been located so far are the large gas fields in the western part of the archipelago, on the Sabine Peninsula of Melville Island and on King Christian Island and Ellef Ringness Island further north. Parts of these fields lie offshore. Gas reserves are estimated at $3.7 \times 10^{11} \text{ m}^3$ (13 Tcf), which is enough to make a significant contribution to Canada's energy resources. Since 1973, Polar Gas has undertaken feasibility studies of various modes of transporting the Arctic Islands gas to markets in the South. It was concluded very early from these studies that the pipeline mode provided the most economical method of transportation. Subsequent work was primarily directed towards the engineering, environmental, financial and socio-economic aspects of a large diameter pipeline. The engineering, field investigations and construction methods related to installing the marine crossings for Polar Gas have been cooperative effort between Montreal Engineering Co. Ltd. and R.J. Brown and Associates. The construction methods and the factors influencing the design of the marine crossings are discussed in this paper.

A Polar Gas route will have to cross the channels between the islands. It is with the construction of these submarine pipeline crossings that this paper is concerned. The route and the environmental and physical conditions are considered first. We then examine the existing state of the art of submarine pipeline construction, and see how far the construction of an Arctic Islands pipeline can rely on known and tried techniques. Finally, we consider specific construction techniques in the Arctic context.
ENVIRONMENTAL CONDITIONS ON THE PIPELINE ROUTE

At the time when work on possible Arctic Islands pipelines began, there existed hardly any of the environmental data needed for the design of submarine pipeline crossings and for the assessment of construction techniques. Since then, a large amount of survey work has been done. Soundings have established the detailed topography of the sea bed, and recording current meters and tide gauges have been used to determine the oceanographic conditions. Sub-bottom profiling and deep borings have been used to study the rock beneath the sea bed, and core sampling, side-scan sonar and underwater photography have given information about the bottom itself. Ice thickness measurements have been made at different times of the year, and there have been sea ice surveys during the summer. All these activities have established an invaluable data base, essential for subsequent work.

A number of route alternatives were examined in detail, and, as a result of this comparison, the route from Melville Island, identified as a solid line in Figure 1, was selected for detailed evaluation of construction methods. Alternate and additional routes are plotted as dashed lines.

An average channel between the islands is 35 km wide and up to 250 m deep. The widest crossing on the main route is that of East Barrow Strait, at the western end of Lancaster Sound, which is 58 km wide, and the deepest is Crozier Strait, one of two water crossings between Bathurst Island and Cornwallis Island, which is more than 300 m deep. In the wider channels, the greatest tidal currents are of the order of 0.5 m/s, similar to those met with in comparable crossings elsewhere in the world. In the narrower channels, the tidal currents are sometimes higher. The bottom is generally silty sand, of relatively low cohesion, to a depth of the order of 10 m below the bottom, underlain by bedrock, which is generally limestone. In a few places, notably the northern half of East Barrow Channel, the bedrock is exposed. Sometimes there is sand, and occasionally soft clay. The bottom is fairly smooth almost everywhere. This is a factor in route selection, since a rough rocky bottom complicates pipeline construction, because of the difficulty of finding a route on which the pipe can rest smoothly and continuously, without long unsupported spans.

The more westerly and northerly channels, between Melville Island and Bathurst Island, and north of Bathurst Island, are covered by sea ice for most of the year. The ice breaks up for a short and unpredictable period in late summer, and reconsolidates and becomes landfast in October. The surface of the ice is generally smooth, but pressure ridges and pressure fields occur. The ice reaches its maximum thickness, about 1.8 m, in late spring, and thereafter becomes thinner and less reliable.

EXISTING TECHNOLOGY OF MARINE PIPELINE CONSTRUCTION

Marine pipelines are usually constructed either by a lowering method or by a pull method. The most frequently used lowering method uses a laybarge. Lengths of pipe are brought from shore by a supply vessel and are one-by-one lined up and welded together, on an anchored barge. The barge advances slowly forward, one pipe length at a time, and as it does so the pipe moves on rollers along a ramp on the barge, through welding stations, where the weld is completed in several passes, and through an inspection station where each weld is X-rayed. The pipe then leaves the barge.
over a buoyant pontoon, called a stinger, and curves down through the water until it reaches the bottom. Except in very shallow water, it is necessary to control the curvature within the suspended catenary span, so that the pipe shall not bend too far and buckle, and this is done by applying tension to the pipe, through a tensioning device on the barge. Several variants on the basis system exist: Figure 2 illustrates one modern lay barge which has been constructed to lay large pipelines in deep water and in rough seas. In good conditions a lay barge can install large diameter pipeline at approximately 3 km per day, but average production rates are generally lower - the decrease being dependent on the sea state and weather conditions.

The control and handling systems of modern lay barges are nowadays highly sophisticated. Because the barge is anchored, by up to 14 mooring lines, each several thousand meters long, accurate positioning and move-up of the barge requires precise control of each mooring line, and this can now be done by an on-board computer. As the barge advances, the anchors have to be relocated periodically, and this is done by anchor-handling tugs, which lift the anchors on pennant lines and carry them forward.

One version of the pull method is illustrated schematically in Figure 3, in the form in which it might be used to construct a line from shore to an offshore tanker mooring. Individual pipe lengths are welded together in a shore make-up site, to make 'strings', typically 500 m long. Winches are mounted on an anchored pull barge, and cables run from the winches to an equalizing sheave at the leading end of the first string. The winches pull the cables, and the first string moves forward until its trailing end is opposite the leading end of the second string. The second string is then rolled onto the launchway, and welded to the first string.

This method has several variants. The winches may be fixed on the opposite shore (as when a pipe is pulled across an arm of the sea), or the pipe can be made up joint by joint on the barge, and then pulled to shore, or the winch barge can itself be moved forward in steps, relocating its anchors with the help of anchor handling vessels. The pull force needed to move the pipe depends on its submerged weight per unit length, its length, and the properties of the bottom soil. Sometimes the effective weight of the pipe string during the pull can be reduced by buoys or pontoons. The maximum force that can be applied is limited by winch capacity, by the strength of the pull cables, and by the capacity of the anchoring system. Maximum pull forces of the order of 5 MN are not uncommon. The longest continuous length of pipe that has been installed in this manner is 30 km of 762 mm pipe off Kharg Island in 1960.

Instead of pulling from stationary winches onshore or on an anchored barge, the pipe can be pulled by a tug. This technique, called bottom tow, eliminates the need for anchor-handling, and the completed pipe string can be moved rapidly to its final position. In June 1977, a 2200 m length of 914.4 mm diameter pipe was made up at a site on the coast of Norway, and towed 400 km in two days to the Statfjord oilfield in the North Sea. There it was towed into its final position in a pre-plowed trench in 160 m of water. The tow vessel was a 22000 HP tug (Figure 4).

Oil and gas exploration and production in deep water have made severe demands on pipelaying technology, and the past ten years have seen a rapid development. The progress of this development is illustrated in Figure 5. Broadly speaking, the complexity and difficulty of a pipelaying project increase both with water depth and with pipe diameter (because a larger pipe is heavier and less flexible, and
cannot be bent so far without buckling, especially in deep water). Each point on
the Figure represents the diameter and maximum depth for a specific project. Recent
major projects are identified by date and name. It can be seen that there has been
rapid progress since 1970, and that the Polar Gas requirement, for a 914.4 mm pipe-
line in depths of the order of 250 m, is not beyond the limits of today's technology.
In temperate climates, certainly, pipeline crossings like those between the Arctic
Islands could be constructed without undue difficulty.

APPLICATION TO THE ARCTIC ISLAND MARINE CROSSINGS

It can be seen from the outline in the previous section of this paper, and from
Figure 5, that the pipeline engineer has at his disposal several possible construc­
tion techniques, but that some of them cannot be used in ice-covered seas, at least
not in their usual form. One of the key decisions to be made is how to relate the
construction task to the presence of sea ice. Should one use conventional water-
based construction techniques, and work as rapidly as possible in the short summer
open-water period, and perhaps in light to moderate ice cover? Alternatively,
should one adopt the opposite policy and work in the winter, using the ice surface
as a stable working platform, and completing the pipeline crossings in the three
or four months when ice is strong enough to carry construction equipment, and after
the continuous darkness of mid-winter? Ice-based construction of this kind follows
the principle that one should work with the Arctic environment rather than against
it. Polar Gas studies have shown that water-based construction is the preferred
technique for the East Barrow Strait crossing, from Cornwallis Island to Somerset
Island, because of the relatively long open-water period there. In the other
crossings, the open-water period is short and unpredictable, and therefore ice-
based construction is to be preferred.

Once this choice between water-based and ice-based construction has been made, the
precise method still has to be chosen, and the alternatives are examined below.
There is a third possibility, that of avoiding the need for a marine pipeline
altogether, by making a tunnel under each channel and installing the pipeline in
the tunnel. It is technically feasible to do this, using tunnelling machines in
the relatively soft rock that underlies the channels. Tunnelling has several
attractive advantages; the influence of the environment is minimized, and a
relatively small number of men can work continuously twelve months in the year,
in conditions little different from those in tunnelling projects elsewhere. The
pipeline itself is protected from the environment and can easily be inspected and
maintained. Against this must be set an uncertainty about the time and costs to
complete the tunnel, since experience with long underwater tunnels indicates that
an encounter with poor-quality rock or a water-filled fault would slow down progress.

MARINE CONSTRUCTION

Four options are illustrated schematically in Figure 6. The laybarge method has
been described earlier. The remaining three options are alternative realizations
of the pull method.

The pull barge (6b) pulls itself forward by cables attached to its anchors,
which are periodically relocated by anchor-handling vessels, and is connected
to the pipe pull-head by chains. The other two methods are the pull ship
(6c) and the tow ship (6d). The distinction between these two lies in the means
of pulling the pipe strings. The pull ship carries two pairs of cables and winches
on a turntable on the ship. Two of the cables lead forward to anchors, and the
other two lead back to the pipe, where they are connected through an equalizing
sheave. The ship pulls itself and the pipe forward, by hauling on the anchor cables.
The anchors are periodically relaid by anchor-handling vessels. The tow ship, on
the other hand, does not use winches and anchors to pull itself forward. It is
essentially a large tug, driven by propellors, and tows the pipe by two cables,
connected to towing winches. The tow vessel system is simpler than the pull ship
system, and is faster, because there is no necessity to relocate anchors. It is
possible to use an existing tug, or to rerig an existing large icebreaker as a tow
ship, or to construct a purpose-built tow vessel.

The first two methods in Figure 6 construct the crossing in a single length from
one side to the other, so that the only connections necessary are those between a
marine pipeline and the pipeline in the shore tunnel. The force necessary to pull
a single pipeline across East Barrow Channel is about 12 MN. This large force
can easily be applied by a pull barge, but not by a pull ship or a tow ship, because
of cable strength limits in the former case and propulsion power limits in the
latter. A pull ship or a tow ship would instead move individual pipe strings, each
of the order of 10 km in length, into their final locations, The strings would
then be joined by tie-ins. Two alternative methods of making the tie-in connections
are illustrated in Figure 7. The first is hyperbaric habitat welding. The ends to
be joined are first brought approximately into line. The pipe is flooded, the ends
are cut, and loose sealing plugs are inserted. A heavy alignment frame is then
lowered over the ends, It has moveable clamps, which grasp the ends and move them
into exact alignment. A 'habitat' is then lowered onto the pipe. It consists
essentially of a rectangular steel box, open at the bottom, with short slots at
either end through which the pipes pass. An umbilical line links the habitat with the
surface support system; divers and materials are transferred by a diving bell.
Seals are placed around the pipe ends, and an oxygen-helium mixture is pumped into
the habitat at local hydrostatic pressure until the water level falls below the
pipes. The pipe ends are next cut and bevelled to match a pre-cut spoolpiece,
already in the habitat, which is then welded into place. The welds can be inspected
and, if necessary, X-ray films can be brought to the surface. When the welds are
complete, the habitat and the alignment frame are recovered, and the plugs are
removed by pigging the line. This method is widely used in the North Sea, at depths
to 160 m.

In the second method, surface tie-in (Silvestri, 1975), the ends to be joined are
lifited to the surface, by a combination of pre-installed pontoons and lifting cables
from davits on a vessel. At the surface the pipe ends are horizontal and in-line,
and can be connected by a short, straight spoolpiece welded under the same condit-
ions as the other welds in the line. Figure 7(b) shows the configuration while the
spoolpiece is being connected. The lifted loop cannot simply be lowered vertically
back to the bottom, because if that were attempted the pipe would go into comp-
ression and buckle, but instead the loop can be lowered sideways (perpendicular to
the plane of the paper in the diagram). The lowered loop then forms a long gentle
curve on the sea bottom. At the end of the operation, the pontoons and lifting
cables are released. Tie-ins of this kind are often done in shallow water, and
their feasibility in deep water was confirmed in 1974 when the technique was used
to make a mid-line tie-in in the 812.8 mm diameter Forties pipeline in the North
Sea, at a depth of 100 m.

Because of the short open-water season, it is important to know whether construc-
tion by these methods can continue in the presence of floating ice. Drifting ice can
interfere with anchor location, anchor handling, and the movements of supply vessels can overload vessel mooring systems, and will affect vessels' freedom to manoeuvre. Detailed simulation studies have been used to test the effects of historical ice conditions on the time it takes to complete the crossings, under various assumptions about production rates and the effect of ice.

**ICE-BASED CONSTRUCTION**

Many different ways of using the ice as a working platform have been examined. Three of them will be described here. The first and simplest is to use the ice to lay cables across the channel, to place a make-up site and launchway on one shore and winches on the other shore, and to use the winches to pull the pipe right across the channel, in the way illustrated in Figure 4. The need for operations on the ice is then minimal. Cables can be lowered through a continuous slot in the ice, cut by a modified pipeline ditcher.

Simple 'shore-to-shore' pull is only applicable if the channel is narrow enough, and the necessary submerged weight of the pipeline low enough, for the pull cable not to be overloaded. The load capacity of available cables is quite high: an 83 mm diameter 6 x 41 IWRC wire rope has a minimum breaking load of 4.32 MN, and the safe working load in this application is about half that value. A cable of this kind can be obtained in continuous 4,000 m lengths, which can be joined by connectors. Although the connectors cannot pass around conventional winches, they can be passed under load through continuous pull machines (gripper jacks). The range of the pull method can be extended by placing the winches or continuous pull machines on the ice itself, and pulling the pipe in 10 km sections. Figure 8a illustrates one acceptable scheme. The winches are anchored to the ice, which can, if necessary, be artificially thickened to support their weight, and the pull cables run across the ice surface, to a moving sled which runs in a continuous slot through the ice. The sled in turn pulls a second pair of cables which are attached to the pipe pullhead through an equalizing sheave. The sled carries a pontoon under the ice. The buoyancy of the pontoon balances the vertical component of the tension in the second pair of pull cables, and thereby reduces the load exerted on the ice slot sides by the sled. Although this system appears more complicated than pulling the pipe directly, it has several advantages. There is no need to lower cables to the sea bottom, and the force needed to pull the cables themselves is much reduced because the friction between the cables and the ice is much less than that between the cables and the sea bottom.

If the pipe is pulled in lengths shorter than the complete crossing, it is again necessary to make tie-in connections. Surface tie-in between pipe lengths can be made from the ice, just as it can from the water, and the configuration in an ice-based surface tie-in is very like that shown in Figure 7b. Winches on the ice surface lift the pipe ends up through a slot in the ice, into an alignment frame, and a spoolpiece is welded between them. The completed tie-in loop is then lowered sideways, supported by cables attached to the lifting frames, which move sideways across the ice, the cables passing through lateral slots. Detailed stress analysis has shown that tie-ins of this kind are feasible in water several hundred meters deep, and that both the stresses that occur during the operation and those that are left locked-in in the final configuration are acceptably small.

A third method of construction, quite different from the other two, is illustrated in Figure 8b. It can be thought of as a combination of the pull method and the lay barge method. The pipe is made up in strings on shore, but instead of being pulled
across the sea bottom it is pulled out directly under the ice, supported by cradles attached to ice anchors. The friction is then very low, and the pipe can be pulled all the way across the channel in a single length, so that no mid-channel tie-ins are necessary. Once the complete string is in place under the ice, it is lowered smoothly to the bottom, by cables attached to lowering winches.

CONCLUSION

It was at one time thought that the construction of a pipeline between the Arctic Islands might present insurmountable technical problems. That fear has proved groundless. All the methods described here have turned out to be technically feasible, and within the reach both of technology available to us today, and of construction methods that have been proven in other areas of the world. The choices between methods depends on considerations of cost and schedule. After careful consideration of all the alternatives, Polar Gas has chosen a preferred method for all the proposed water crossings:

- Byam Channel - Ice island bottom pull
- Austin Channel - Ice island bottom pull
- Crozier Strait - Tunnel and shore to shore pull
- Pullen Strait - Tunnel
- East Barrow Strait - Laybarge

In addition, it is planned to tunnel all shore approaches to protect the pipeline against possible ice scour.

REFERENCE

FIGURE 2  THIRD GENERATION LAYBARGE

FIGURE 3  BOTTOM PULL : SCHEMATIC
FIGURE 4  BOTTOM TOW: LAUNCHWAY AND TOW VESSEL

- Laybarge
- Bottom tow

- Sicily 1976
- North Sea 1975 77
- Statfjord 1977
- Forties 1973

- HIOS 1976

Experience to 1970

Depth (m)

200
400
600

100
200

60
40
20

Diameter (m)

0.1
0.2
0.4
0.6
0.8
1
2

FIGURE 5
DEVELOPMENT OF DEEP-WATER PIPELINE CONSTRUCTION
FIGURE 6 ALTERNATIVE METHODS FOR MARINE CONSTRUCTION
(a) LAYBARGE  (b) PULL BARGE  (c) PULL SHIP  (d) TOW SHIP
FIGURE 7 TIE-IN METHODS
(a) HYPERSONIC WELDING  (b) SURFACE TIE-IN

FIGURE 8 ALTERNATIVE METHODS FOR ICE-BASED CONSTRUCTION
(a) BOTTOM PULL  (b) SURFACE PULL AND LOWERING-IN
CONNECTION OF SUBMARINE PIPELINES IN ARCTIC CONDITIONS

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INTRODUCTION

The discovery, in the early 1970s, of gas reserves in the Arctic Islands introduced several new dimensions to the already challenging list of problems involved in developing offshore finds. The most obvious of these are the low ambient temperature which frequently falls to below -40°C and the year-round presence of an ice cover to the sea. Both of these features contribute to a third difficulty, that of the logistics and costs entailed in transporting men, materials and equipment to the construction site. For Panarctic Oil's Drake Field off Melville Island, (which is referred to in this paper) all three of these critical aspects of Arctic operations are present in full measure.

The Panarctic Drake and Hecla gas fields are located on the Sabine Peninsula of Melville Island, and extend to considerable distances offshore. Preliminary studies of both fields indicated that of the various means available for development of multi-well offshore fields, the use of individual bundled lines from each well to shore would be the least expensive while also offering a high level of confidence. Major phases in the construction of these pipelines would be their installation in conditions of continuous ice cover, and their under-ice, subsea connection to wellheads at depths ranging down to 350 meters.

From marine surveys of this particular area, it is immediately apparent that conventional pipelay equipment would be unable to operate because of the ice cover. As part of an assessment of alternative means of installation, studies were made of the ice thickness and strength to determine whether land-based equipment could operate from the ice surface. These studies confirmed that the bottom pull installation method could be employed, and would be able to take advantage of the continuous ice-cover to provide a stable work platform.

Also, use of the bottom pull method offered the possibility of limiting the pipeline's period of exposure to low temperatures solely to the make-up and pre-launch operations prior to entry of the pipe into the water. Once submerged, the line would assume the approximately -2°C ambient temperature of the sea, which is relatively warm compared with the air temperature and is not greatly different from the sea bottom temperatures encountered, for example, in the North Sea.

Techniques for pulling the pipe to its subsea location, stabilizing the line, and trenching followed by backfilling, have all been developed. This paper therefore addresses itself specifically to the proposed method for connecting the pipelines.
to the wellheads once the lines have been positioned close to their final undersea locations.

SYSTEM OF PIPELINE CONSTRUCTION

Bottom pull installation of the Panarctic flowline bundles from the ice surface will generally follow a standard series of operations. The pipeline sections will be made up onshore, tested, and their pulling heads installed. A slot will be cut through the ice for stringing the pull cables on the sea bottom, pulling equipment complete with deadman will be placed on the ice, and the pipe will be pulled to a target area close to its final location. The correct positioning of the pipeline within its target area will then be confirmed. Finally, as a special feature of the installation process, the lines will be deflected sideways by lateral pull to make connection with the wellhead.

The normal procedure for bundled lines has been to make up the pipelines in long bundles, attach buoyancy pontoons, install the lines and recover the pontoons. Experience in the North Sea, however, has shown that this is a more expensive method than making up the bundles within a continuous conduit which can then both act as a pontoon and be used for additional purposes after the line has been installed. This concept was therefore adopted for the Panarctic lines, as illustrated in the sectional view in Figure 1.

With the flowline bundle pulled into location, the annulus area within the carrier pipe can be utilized to enhance stability in a variety of ways. These include filling the annulus either with grout to provide concrete reinforcement, or with water inhibited with an oxygen scavenger to give added weight stabilization. Alternatively, the annulus may be purged with nitrogen or filled with an insulating foam material.

To guard against the possibility of damage from ice scour over the upper shore zone, the lines will additionally be protected by trenching in from the high water level to approximately minus 20 meters. The ditch will be cut by underwater plow, and will be backfilled after installation of the pipe, as shown in Figure 2.

CONNECTION METHOD UNDER DEVELOPMENT

In assessing the most suitable method for connecting the flowline bundles to their individual wellheads, model analysis offered some unique advantages. The problems involved could be viewed three dimensionally; problem areas could be investigated which would not be readily amenable to theoretical analysis; the optimum sequence of construction events could be established; and, providing that the correct parameters have been built into the model, useful measurements can be taken. A simulation model was therefore prepared to determine the practical and technical aspects of the lateral pull rigging, and to establish step-by-step procedures for the connection process. For this, a wire of appropriate weight and stiffness was used to simulate the flowline bundle.

The first objective of the connection procedure is for the line to be pulled to within a prescribed target area. This ensures that the leading end can then be deflected into the wellhead for the connection to be made. Figure 3 illustrates this initial step. To ease the deflection process, the leading 300 meters of pipe are buoyed off the sea bottom by a series of pontoons, as shown in Figure 4. The pontoons, together with lengths of chain attached below the pipe, are spaced at approximately 45 meter intervals. The positive buoyancy provided by the pontoons becomes counter-balanced at a lift-off height where the collective weight of the length of pipe and chain raised off the seabed renders the system neutrally buoyant.
As one of the design considerations for the system, a performance specification has been prepared for the pipe stresses occurring (1) during the deflecting process to the wellhead, and (2) during the normal operating mode of the flowline bundle following connection. For the first of these conditions a maximum of 90% yield stress in the conductor pipe has been set, which is consistent with accepted offshore installation practice: for the second, a limit of 60% residual combined stress has been designated. With these as criteria, and assuming that the leading 300 meters of the flowline bundle have been buoyed in the manner described, it is possible to establish an optimum point at which to position the lateral pull winch which minimizes the risk of overstressing the lines. For the 6, 2 and 1 inch pipelines, in their operating life condition, this in fact corresponds to bending stresses lower than 15% yield.

Figure 5 shows the flowline bundle after being pulled to location and prior to being deflected toward the wellhead. The connector sled on the leading end of the pipe is within its target area, the lateral pull rigging is in position and the deflecting process is about to be initiated.

Because a long length of pipe is off bottom, only a small force is required to cause deflection. For the configuration shown in Figure 6, a lateral pull back of 0.1 ton is sufficient to deflect the sled end toward the wellhead by approximately 15 meters. In this condition, the maximum bending stress in the conductor pipe is 5%. When the pull back force is increased to 1.2 tons, as shown in Figure 7, the pipe sled moves over an arc of approximately 55 meters length. In this condition, the stresses in the outer conductor over the main sidebend curve rise to 90% of yield. In the minor reverse bend the stresses are approximately 45% of yield.

As the weight and stiffness of the wire in the model closely simulate these parameters in the fullscale bundle, the configurations illustrated represent close approximations to those of the actual installation process. The stresses in the pipe can therefore be readily determined at any phase of the deflection process.

At the stage shown in Figure 7, a pull-in cable is connected to the sled through the wellhead and a pulling force of approximately 0.2 tons is applied in the direction of the wellhead. This operation effectively increases the pull-back force to 1.4 tons, and the winch is adjusted to reduce the load back to 1.2 tons again. This situation is illustrated in Figure 8. By repeating this procedure the sled is progressively moved towards the wellhead until it eventually impacts and latching of the connector mechanisms is achieved. With the sled connected to the wellhead, as shown in Figure 9, the residual stress in the sidebend corresponds to 60% of yield.

At this juncture, the pontoons are released, leaving the flowline bundle in its final configuration as shown in Figure 10. During the deflection and connection processes, the pipe acts as a spring and absorbs a maximum force of some 1.2 tons.

The model rig can also be tilted to represent the effects of a lateral current. In this way, currents of 1, 2 and 3 feet per second have been simulated and have shown that the stress level and holdback force undergo only very small changes. However, the current causes very large deflection of the sled and pipe leading end. For example, a current of 2 feet per second results in a deflection of the sled of approximately 9 meters. Under these circumstances it is clear that deflection and connection of the sled should be performed during the period of slack tide. An investigation is being made to confirm that vortex shedding will not induce resonance in the pipe during the deflection process.
SPECIAL RIGGING

For successful deflection of the pipeline into the wellhead, it is vital that proper care and attention be paid to the rigging and installation procedures. To confirm the correct operation of the rigging, the model rig was employed using larger-scale details. Figure 11 shows the pulling head landed in the target area and the lateral pull-back cable being lifted by buoy to the surface after release from the connector sled. When the cable reaches the surface, it is moved back to a point above the intended position of a sheave on the sea bottom, representing the pull-back point, as illustrated in Figure 12. The pull-back sheave and its anchor are then lowered to the bottom followed next by the pull-back cable, as shown in Figure 13. The deflection process can then be initiated, using the lateral pull winch on the ice surface.

When deflection is completed and the sled is located opposite the wellhead, the pull-in rigging can be positioned. Figure 14 shows the sled in location and the pull-in cables from the wellhead being lifted to the surface by buoys. When the cables reach the ice surface, a pull-down sheave guide system is fitted. The pull-in cables and sheaves are then pulled down to the wellhead, as depicted in Figure 15, and the system is ready for connection.

CONCLUSIONS

Based on the engineering work completed to date, a number of conclusions can be made:

- Deflection offers a technically-feasible process by which a pipeline can be connected to a subsea wellhead.

- Rigging procedures are available to ensure a successful connection by this means.

- Modeling can be of great advantage to projects of this type, to establish the detailed procedures required for making the connection.

- Modeling also enables potential problem areas to be defined and solutions developed.

- Lateral currents do not have a critical influence on stress levels and deflection loads, provided that the deflection process is properly analyzed.

- Connections to wellheads can be made using a minimum size of surface spread and very low deflecting loads.
Figure 7

Figure 8

Figure 9

Step No. 4

Pull force = 1.2 ton

Step No. 5

Bending stress = 45% yield

Hold back force = 1.2 ton

Step No. 7

Bending stress = 60% yield

Bending stress = 60% yield

Step No. 8

Bending stress = 55% yield
FIGURE 10

STEP NO. 7 IN SERVICE

FIGURE 11

RELEASE LATERAL PULL CABLE

FIGURE 12
INTRODUCTION

In planning marine facilities in Canada, a major factor of uncertainty has been the time delay to shipping due to ice conditions. This paper outlines a general approach to the problem and discusses the application of the approach to several recent projects in Eastern Canada.

THE KINDS OF DELAY INFORMATION NEEDED

Ice conditions on a given route are usually variable, and information on "average" or "typical" conditions is of little value. Information on delays must be related to those actual incidents which affect the project under consideration.

Answers to the following questions are usually sought:

1 - Will vessels ever be completely stopped, and over a period of years, how many times?

2 - How long may standstill conditions persist?

3 - What delays can be expected when passage is not prevented but when vessels are forced to proceed at reduced speeds?

4 - What is the influence of vessel ice capability on these projected statistics?

The best basis for predicting delays is actual experience in the operation of vessels on the route in question. The need for analysis arises in cases where there is no comparable experience on the route concerned.

APPROACH

A common approach which has evolved in a number of studies consists of four basic steps:

1 - Compilation of a quantitative description of historical ice conditions.

2 - Definition of the ice navigation capabilities of the various vessels involved.

3 - Translation of historical ice conditions into a corresponding description of impediments to navigation.
Interpretation of statistical results.

Chronology Of Historical Ice Conditions

The sources of information for a chronology of ice conditions consist mainly of present users of the route, aerial reconnaissance records, satellite images, and custom surveys.

Owners and masters of vessels using the route, and pilots serving them, can usually give firsthand information on conditions they have experienced. However, long-term written records are not usually available, and the observers' perceptions relate to the capability of their own vessels. If traffic is intermittent many significant events may have escaped notice. Records or calls for assistance to Coast Guard ice breakers may be available, if sufficient use has been made of the route in the winter months.

The Ice Central Office of the Canadian Atmospheric Environmental Service, and its predecessor, the Meteorological Branch of the Department of Transport, have carried out a systematic surveillance of ice conditions in many of Canada's coastal waters since 1959. Ice observer reports from reconnaissance aircraft, ice breakers, other ships, and shore stations have been compiled on daily ice charts and weekly summary maps since that time. These records offer several advantages for analysis. They have continuity over a period exceeding 15 years, and can identify ice features important to navigation. They extend over many coastal waters where there are no other forms of observation. Their principal disadvantage lies in the fact that the coding system developed for use by airborne ice observers does not directly reveal the extent to which vessel movements may be impeded.

The Earth Resources Technology Satellites have produced images which show ice coverage and ice characteristics in impressive detail. However, the satellite passes are currently too infrequent to provide a worthwhile record of ice conditions through the winter season. As new and more frequent forms of imagery become available, satellite imagery promises to be a major source of ice information.

When major activities are about to begin at a particular location, it is often worthwhile to undertake detailed local surveys of ice conditions at the site. These surveys are expensive and do not cover an appreciable time span. They may, however, permit details of the local ice regime to be understood.

Historical ice conditions may also be related to longer records of temperature and wind conditions.

Definition Of Vessel Capability

The most important factor in the definition of the physical capability of a vessel is the severity of ice conditions which prevents it from making a passage. A number of studies have been carried out in the Gulf of St. Lawrence and in the Great Lakes in attempts to relate factors such as vessel size and power to the limiting ice conditions (Bradford, 1972 and a report for the Maritime Administration, 1972). The Bradford scale, Table 1, was developed from an analysis of vessels in the Gulf from 1964 to 1968. The principal parameter used to rate ship capability is the shaft horsepower divided by the ship length. The St. Clair-Detroit scale was developed for significant ice conditions in the Great Lakes, and is shown in Table 2. This scale employs both the power to length and power to beam ratios as classification parameters.
Efforts have also been made to estimate the rate of progress which vessels can make under more moderate ice conditions. Some reduction in speed is made whenever ice is present as a matter of prudence. If the vessel has adequate strength, it may use its power and momentum to force a passage at whatever speed it can manage. The effectiveness of vessels strengthened to navigate in ice depends largely on the rate of progress they are able to attain under prevailing conditions.

Vessels not specifically designed as ice breakers usually operate in loose ice fields or in first-year ice of moderate thickness. A relationship known as Kashtelyan's equation (Kashtelyan, 1968) which relates the resistance of a ship's hull passing through an ice field to the dimensions of the ship, the form of the hull and the character of the ice cover, has been employed in making quantitative estimates of passage times (Cowley, 1976).

**Development of a Synthetic Sequence of Navigation Delays**

By applying the ice navigation capabilities defined for the vessels under study, the composite description of historical ice conditions can be transformed into a description of the instances when ice would have interfered with navigation if the route had been in use during the whole of that period of time. This synthetic record of delays is an estimate which can be used to answer the fundamental questions about the frequency, severity, and persistence of delays to be expected.

**Interpretation of Results**

The estimates which are made by this process are not numerically precise. However, they do reveal the character of ice delays and provide a means of judging the influence of these delays in terms which have direct meaning to the proposed project. The conclusions which are drawn will depend upon the importance of maintaining strict schedules, the costs associated with inventories to guard against delays, and other features of the particular project under consideration.

**CASF STUDIES**

The following are descriptions of three brief case studies which illustrate the application of the approach.

**Case Study A - Marine Terminal, Grande Ile, Quebec**

This feasibility study involved a proposed terminal for vessels up to about 272 000 tonnes which would serve a pipeline to carry oil inland from the vicinity of Grande Ile, about 160 km northeast of Quebec City, in the St. Lawrence estuary (Figure 1). The study has been described fully in a previous paper (Cowley, 1976) and only a summary is presented here.

The design supertankers were six times larger than the largest ships previously using the navigation route, with lengths in excess of 300 m and depths of more than 24 m. A time delay estimate was required because of the unprecedented size of the vessels and the important influence of delays on storage requirements at the terminal.

Ice is present in various forms in the St. Lawrence river and estuary for about five winter months. Shore-fast ice sheets generally reach a maximum thickness of 0.6 m. The dominating ice conditions, however, are ice flows influenced by winds and tides.
Certain exceptional ice formations are caused by local conditions of pressure and movement. Sustained onshore winds bring ice to the coast causing ridged and rafted formations.

A large number of vessels successfully use the route through the Gulf of St. Lawrence to Quebec City on a year-round basis. In 1972, the number vessels moving past the study site exceeded 200 per month in every month of the year. The largest vessels on the route consisted of 45 000 tonne tankers serving an oil refinery in Quebec City.

The previous uncertainties of winter navigation in the area have been greatly reduced by ice traffic control and ice reconnaissance forecasting services. However, delays of up to several days were experienced, and in certain ice conditions vessels were unable to proceed.

The technique used in this study consisted first of dividing the approach route into seven regions. The ice summary charts available from Ice Central were examined and ice condition for every week of 13 years of record were tabulated. The best route through the estuary and gulf was established, as dictated by the prevailing ice conditions. The summary then presented the worst ice conditions on the best route during each time period.

In order to relate ice conditions to ship progress, the Bradford scale previously mentioned was used in the final form of the ice condition summary. Figure 2 shows part of the data, namely, the worst ice ranks along the entire route as a function of time. The complete set of data was assembled for sorting and analysis by computer.

The resistance function for movement of the design vessel through the ice was represented by Kashtelyan's equation. Speed loss calculations were carried out on three sets of assumptions which reflect different hull strengths, as seen in Figure 3. The graphs show an envelope of speeds due to a range of assumptions, but median values were used in the resulting predictions.

For each week of the available records, a delay time was calculated for each of the seven reference regions. The delay for the complete route in each week was then computed. From the information obtained, average and maximum ice delays per passage were calculated. A statistical analysis of the calculated delays was made for each week of the winter season, as shown in Figure 4. The extent to which significant delays tend to occur in consecutive weeks was also computed.

Case Study B - Oil Storage Project, Conception Bay, Newfoundland

A study was conducted on the possibility of using the iron-ore mines on Bell Island, Conception Bay, Newfoundland, as an oil storage and trans-shipment facility. (The site is shown in Figure 1.)

In general, Arctic ice is formed off the coast of Labrador in mid-December and is carried south to Newfoundland waters under the influence of winds and currents. Historically, in 15 years of ice-recording data, the ice pack has extended as far south as Conception Bay about 70 percent of the time. This penetration depends on the severity of the winter and the predominance of a north to northwest wind.

Again, the purpose of the ice study was to interpret ice data as they affect ship movements to and from the proposed marine terminal on Bell Island, and to provide a quantified assessment of potential delays to navigation.
At the time of the study, a car ferry which was not specially ice-strengthened operated to and from Bell Island. The largest ships in the bay served a local refinery, and only some were ice-strengthened.

The greatest concentration of ice in the bay is Arctic pack ice, with the heaviest conditions caused by a pressure build-up from a strong northeast wind. The duration of such an event is usually less than 2 weeks.

Historical data on ice conditions were again obtained from ice maps on file at Ice Central. A list was prepared of the occasions when ice was present in or near Conception Bay for the period 1961 to 1975. The ice events on this list were then ranked on the Bradford scale, for three different areas in the bay.

In figure 5, the occasions when ice occurred in Conception Bay or across the entrance are indicated graphically. The information obtained shows that presence of ice of any rank occurs about 65 percent of years, and that the total time of ice presence varies from 1 to 10 weeks, with an average of 5 weeks.

Two classes of ships are expected to serve the proposed terminal; Very Large Crude Carriers (VLCC's) of up to 450 000 tonnes displacement, and smaller tankers of up to 45 000 tonnes. The VLCC's will likely be handled with great caution and will avoid all but light ice conditions. The smaller tankers will likely operate in moderately severe ice conditions.

On the basis of the data tabulated, the following conclusions were reached:

1 - Ice will have no effect on navigation in 35 percent of years.

2 - Tankers of the 45 000 tonne class will experience ice conditions resulting in a maximum delay of less than a day in 40 percent of years and will be halted in 25 percent of years.

3 - VLCC traffic will be subjected to ice resulting in a maximum delay of less than a day in 15 percent of years and will be halted in 55 percent of years.

Cast Study C - Extension of Navigation Study, St. Clair-Detroit Rivers System

In 1974 a study was conducted in order to examine the various factors affecting the extension of the navigation season in the St. Clair-Detroit Rivers system, and to recommend specific measures to extend the navigation season for definite increments of time (Acres 1974).

The St. Clair delta area (Figure 1) has been highly susceptible to ice jamming in the past. The occurrence of ice jams is caused by the development of ice cover in Lake St. Clair and the delta channels, followed by the transport of large volumes of ice from Lake Huron through the St. Clair River.

The stability of the ice cover in the lakes is very sensitive to wind direction and velocity. Vessel tracks in the lakes can be closed by wind forces within a very short time of the passage of the vessel. The broken ice in the track during frequent passages can break down and become saturated by constant immersion to form a mush ice, to depths of 1 to 3 m.
The action of movement through the ice for lake vessels is quite different from that of ocean-going vessels. The bow configuration of lake bulk carriers is quite blunt, and the angle of contact with the ice sheet is almost 90 degrees. Hence, the ice must be broken primarily by crushing, rather than by bending.

A different ice ranking system was then required for this study, owing to the special nature of lake and river ice, and the vessel operating characteristics. Two ratios were used to relate vessel operating capability to ice rank. The ratio of shaft horsepower to beam is significant when dealing with movement through uniform ice cover, since this movement is primarily related to the beam dimension. The ratio of shaft horsepower to length is significant in situations where the vessel is moving through extensive mush ice. In this case friction along the length of the hull is important. Even this approach is very simplified, since other factors such as hull shape, bow shape, method of propulsion, and trim all have influence.

The representative vessels of the United States fleet for a particular year were assigned an ice rank capability. This factor defined the ice condition which would stop the forward movement of the vessel. Two cases were presented; first, where the hulls had sufficient hull strength to operate at 100 percent rated shaft horsepower and second, where the hull was restricted to 75 percent rated shaft horsepower because of hull strength limitations.

The most severe ice conditions in each reach for each year of an 11 year period were then summarized. These "design" ice conditions, which have previously occurred in the system were considered the best estimate of what might be expected in the future.

To verify this 11 year record, a comparison was made with temperature records extending from 1940 to 1972. This showed that the observation period was slightly more severe than the long-term average.

The next step was to use the ice rank capabilities to representative vessels to determine where the weakest vessel would be stopped by the ice. Certain problem areas in the study zone were then identified. This basic information was then used to develop design criteria for improvements to winter navigation, in the form of ice booms, pile clusters, air-bubbler systems and ice breakers.

**CONCLUSIONS**

In conclusion, there is adequate ice data to make analyses of delay worthwhile in most Canadian waters. The great variability of conditions from year-to-year is one of the most important factors to be recognized in the analysis.

Various methods of describing ice conditions and assigning ice navigation capability have been used. Their selection is dictated by the ice conditions which prevail and the hulls of vessels under consideration.
REFERENCES


Figure 1 - Location map of case studies in Eastern Canada.

A - Grande Ile, Quebec
B - Bell Island, Nfld.
C - St. Clair - Detroit Rivers

Figure 2 - Occurrence of ice packs in the overall approach route, Study A.
CASE 1

CASE 2

CASE 3

LEGEND

ENVELOPES OF SHIP'S SPEED, DETERMINED FROM A RANGE OF ASSUMPTIONS IN KASHTETTAINS EQUATIONS

SPEED LOSS RELATIONSHIP EMPLOYED IN DELAY CALCULATIONS

NOTES
CASE 1 HALF POWER IN ICE; MAXIMUM SPEED IN ICE 8 KNOTS
CASE 2 FULL POWER IN ICE; MAXIMUM SPEED IN ICE 8 KNOTS
CASE 3 FULL POWER IN ICE
ALL VESSELS 285,000 DWT 32,450 HORSEPOWER

Figure 3 - Calculated vessel speeds under ice conditions, Study A.

CASE 1

CASE 2

CASE 3

CALCULATED DELAY DUE TO ICE IN HOURS PER PASSAGE

CALCULATED DELAY DUE TO ICE IN HOURS PER PASSAGE

CALCULATED DELAY DUE TO ICE IN HOURS PER PASSAGE

Figure 4 - Statistical occurrence of calculated delays, Study A.
Figure 5 - Ice events in Conception Bay and bay entrance, from 1961 to 1975.
Table 1 - The Bradford Rank Scale

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Table 2 - The St. Clair-Detroit Ice Rank Scale

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Governments have been exploring further afield, into more remote areas, in search of raw materials and fossil fuels. One of these areas is Labrador and its offshore Continental Shelf. The latter, for a major part of the year, is covered with sea ice in addition to having an annual pilgrimage of icebergs down the Labrador Current.

The problem may be defined as: How to move a vessel to and from the Labrador Coast with safety and efficiency all year round. Bearing in mind that the ice cover only exists for part of the year, the economics of the permanent use of icebreakers and/or icebreaker cargo ships is therefore brought into question.

This paper will give an overview of the ice problem, examine past observations and in the light of experience, examine the possible approaches for future observations and research, bearing in mind that many have mooted the idea that a specifically constructed vessel can be built to deal with one kind of ice. Is this the case in Offshore Labrador?

INTRODUCTION

This paper represents an attempt to investigate the problem of marine transportation in offshore Labrador by analyzing those characteristics of the Labrador region which make it unique, assessing their impact on marine transportation, and outlining possible alternatives for marine transportation systems, together with their possible advantages and disadvantages. Due to the complexity of the problem and the lack of available data on many of the key variables, this study is of necessity, very general. Without more detailed information, it is not possible to make definitive statements regarding 'optimum' marine transportation systems for the area. Furthermore, any attempt at an analysis would be fraught with uncertain economic assumptions. It is not the intention to address such issues here. This paper is intended principally to demonstrate the degree to which environmental conditions in the Labrador region will affect the design and selection of marine transportation systems, and to provide a basis for further, detailed analysis of the kind mentioned above.

The problem is principally one of economics. To oversimplify, it may be said that a desirable transportation system is one which provides a required service at minimum cost under given external constraints. In this case, the required service depends upon the market for shipping in Labrador, and the principal constraints are imposed by the environment. Therefore these two issues will be addressed. However, the treatment of the market is very brief and does not include any definitive predictions.
It appears more important at present to investigate the effect on shipping if certain market conditions prevail, than attempt to predict the probability of them prevailing.

**LABRADOR ENVIRONMENT**

Very little hard data on the environment of the Labrador Coast is presently available. The following description is based upon Sailing Directions, together with first hand experience of the area.

Ice on the Labrador Coast consists mainly of three types which are of consequence to shipping:

a) first-year sea ice, formed locally along the coast each winter;

b) first-year and multi-year polar sea ice, carried south from the Foxe Channel and Davis Strait areas by the Labrador Current and the prevailing winds; and

c) icebergs, calved from the glaciers of Western Greenland, and carried south by wind and current.

The inner channels along the Labrador Coast begin to freeze in early November. Polar ice generally reaches Cape Chidley in the North about the same time. Inshore areas north of Nain can expect fast ice in November one year out of five. By early December, sea ice reaches as far south as Cape Harrison, and by late December the entire Labrador Coast is exposed to sea ice. Generally at this time the ice is first year to about 55°N with thicknesses measured at Hopedale during December, 1955 averaging 7 inches (Weeks, 1956).

In early January, sheltered inlets such as Hamilton Inlet have ten tenths coverage of grey-white ice (6"-12" thick), while coastal areas as far south as Spotted Island experience about six tenths young ice (3"-12" thick). By mid-January, young ice (3"-12" thick) averages eight tenths coverage north of Cartwright, while areas further south have four to six tenths grey-white ice. The offshore extension of sea ice during this period is approximately 100 to 150 miles.

During February ice growth continues to increase and maximum ice conditions are usually reached during March, when grey-white ice can extend as far as 250 miles offshore, averaging about nine tenths coverage.

The southwesterly extent of sea ice begins retreating in April but it is usually late May or early June before navigation can be resumed as far north as Goose Bay. At this time, observations indicate there is still heavy ice drifting along this section of the coast and even though it is generally about 25 miles offshore, communities north of Hopedale can expect to be isolated by sea ice one year out of three.

Navigation on the entire coast is generally not possible until late July. The amount of knowledge of ice conditions in the area, accumulated over many years by local fishermen and mariners and handed down from generation to generation is quite phenomenal. It is to be regretted that this knowledge is being lost before it can be recorded.

For instance, very little information is available as to whether any dependency can be placed on the amount of open water which can be expected to follow a particular set of meteorological conditions or even if any open water exists at all at certain times during the winter. Again, given certain meteorological conditions, do leads develop in any specific location, the direction of which could be utilized by a mariner in furthering passage through the pack? Given a repeat of the same meteorological
conditions is there a recurrence of a 'lead' in the same general area? This is the type of knowledge which is being lost and which will be extremely expensive to replace.

Newfoundland and Labrador are at the focus of all major storm tracks in North America. Winter depressions traversing the area are, in the main, relatively fast moving and in conjunction with the Labrador Current and ocean swells, tend to create an extremely dynamic situation in the ice pack on the Labrador Coast.

Wind diagrams for Hopedale and Cape Harrison, Figures 1 to 3, are self-explanatory. The readers attention is, however, directed to the percentage frequency of easterly winds which occur between December and April.

Fall conditions recorded in 1970 are similar to those of 1955-62. However during the ice season the dominant wind directions alter to more southwesterly components during the early part of the season and are replaced by more northwesterly components for the rest of the duration.

**COMPARISON WITH OTHER ICE-INFESTED WATERS**

The key features distinguishing the offshore Labrador environment from others affected by ice are:

1) the ice infestation occurs during part of the year only;
2) the Labrador coast is an open sea environment hence subject to long frequency swells from weather systems outside the immediate area; and
3) during the ice season, ice conditions are extraordinarily dynamic.

To illustrate the peculiarities of environmental conditions on the Labrador Coast more clearly, they will be compared with those found in some other ice infestation areas of the world.

**Canadian Arctic**

'Canadian Arctic' refers here to the broad area extending northwards from the Canadian mainland and Davis Strait to the northern extremities of Canadian territory - an area very roughly triangular in shape.

For the most part, the Canadian Arctic consists of stretches of sea, straits, and channel enclosed by land. In comparison with the Labrador Sea there is little fetch. The principal consequence of this feature of Arctic geography is that the area is less exposed to the action of ocean swells and therefore more conducive to the rapid accumulation and consolidation of sea ice.

An exception to this enclosed sea generalization is the northwest boundary of the Canadian Arctic (ie. the boundary between the Queen Elizabeth Islands and the Arctic Ocean, and between Banks Island and the Beaufort Sea). The coastline here is possibly protected from swell action by the presence of the permanent polar pack, making the concept of an 'open sea' questionable. On the other hand, the absence of land at this boundary means that no protection is provided from the wind. With a long fetch - right across the polar pack - the wind impact can be severe and can result for example, in heavy pressure in McClure Strait.

Another characteristic of the Arctic is that the climate is much more severe thus making ice accumulation greater. The same influence increases the time over which the ice is consolidated. In extremely high latitudes (the permanent polar pack) the
differences between winter and summer ice conditions are relatively minor - the pack is indeed permanent. Further south, this difference increases as the growing influence of milder climates allows longer periods of weakness in the ice. Eventually latitudes are reached where the influence of the climate is sufficiently strong for open water to prevail for a portion of the year.

In summary, the environment in the Arctic is, in general, much less dynamic than that off Labrador even though climate conditions are much more severe. This basic dissimilarity implies different problems for shipping in each area. In the Arctic, ships must contend with thick, consolidated ice in winter, a period of rapid break-up in spring, followed by a short summer with considerable open water. On the other hand, off Labrador the extent and severity of consolidated ice (in this case generally shore fast) is not such a major problem as the continual motion and variety of ice in the offshore pack.

One final difference which must be noted is that, despite the many gaps which still exist in our knowledge of Arctic ice conditions, the environment in the Labrador Sea is even less well understood.

The Gulf and River St. Lawrence
This area, almost surrounded by land and accessible from the Atlantic only by way of Cabot Strait or the Strait of Belle Isle, has a relatively 'quiet' ice environment. Its southerly latitude limits the length of the ice season and the extent of ice growth. The formation of ice begins about mid-December and floes up to three feet thick may occur late in the season. Break-up and dispersion of the ice cover extends from March to possibly as late as mid-June, depending upon particular weather conditions.

Compared with the Labrador Sea, ice conditions in the Gulf of St. Lawrence can be considered less dynamic. Naturally the pack is not static, particularly during the spring breakup. At this time the predominant westerly winds tend to compress the ice against the West Coast of Newfoundland and in the approaches to Cabot Strait and pressure ridging in these areas can be quite severe at times. In the River below Quebec, large tides and strong currents tend to create and relieve pressure almost on a cyclical basis though these conditions can be aggravated or improved by climatic conditions at the time.

It can be stated as a general observation however that the more settled weather conditions in the area and the protection provided by the land, create an area which is not subject to the continual and rapid change characteristic of the Labrador pack.

Finally, the experience base for shipping in the Gulf of St. Lawrence is immense in comparison with knowledge of the conditions off Labrador. The relative ease of observation of ice conditions over the entire area and the volume of traffic provides for the routing of vessels on tracks where the lightest ice conditions prevail with the result that navigation to Montreal is almost a routine matter throughout the entire winter.

Baltic
As with the St. Lawrence, the experience base for shipping operating in the Baltic ice season is considerable. Much of the pioneering work dealing with icebreakers and the operation of ships in ice was performed in the Baltic countries. The same countries are still highly active in the field today.

The characteristics of the Baltic which are of importance from the point of view of shipping during the ice season are that it is a relatively closed sea environment
(surrounded by land), generally shallow and subject to a fairly consistent meteorological pattern. The net result of these factors is that the ice conditions are much less dynamic, more readily predictable, and generally easier to deal with. An additional feature of the Baltic which simplifies shipping operations is the absence of a major source of icebergs or drifting old ice, such as that which is provided off Labrador by the Labrador Current.

**SHIPPING MARKET**

It is useless to try to devise an efficient transportation system without references to the market which it will service. The nature of the goods being shipped will affect the choice of size, speed and other characteristics of the vessels employed. Consequently, the market for shipping in Eastern Labrador will be briefly addressed here.

At this point, it is important to differentiate between Western and Eastern Labrador. The Western Labrador market is large and is based upon the extraction and exportation of mineral resources, mainly iron ore and the support of the mining population. It is served through the port of Seven Islands in the Province of Quebec. It is also important to note that there is no transportation link between West and East Labrador. East Labrador is therefore dependent upon service through the ports on the Labrador Coast. For convenience the market has been divided into two parts: imports into the region and exports from it.

The first category, imports, consists mostly of goods necessary to support the local population and local economic growth. These include: foodstuffs, construction materials, oil and oil products, vehicles and miscellaneous goods such as personal effects. The quantities being considered here are quite small, as is appropriate for the small market being served.

The second category, exports, consists almost entirely of pulpwood. In addition, however, coastal traffic within Labrador contributes a small amount of tonnage.

Tables 1, 2 and 3 show the total traffic (both coastal and international) passing through Goose Bay for the years 1973, 1974 and 1975. This traffic has been divided into seven major groups, and the tonnage loaded and unloaded at Goose Bay in each group has been recorded. Total traffic and net movement of goods (ie. in or out of the port) are also presented. Though complete statistics are unavailable, Goose Bay shipping probably constitutes about 90% of the total for Eastern Labrador.

Tables 1, 2 and 3 clearly show the predominance of pulpwood in the traffic exported from Goose Bay. The figures for tons loaded in Groups 1-5 and 7 mainly represent goods coming into Goose Bay for local distribution and for transhipment by sea to other parts of coastal Labrador. Thus at the present time, the only export by sea from Eastern Labrador is pulpwood. The vast quantities of iron ore from Labrador West - some 33 million tons in 1975 - proceed via rail for export through the ports of Seven Islands and Port Cartier in the Gulf of St. Lawrence. The remainder of the regions natural resources have yet to be exploited.

Among these resources are large deposits of various ores - which are likely to be among the first to become economically attractive. The potential market for shipping these commodities is large, and once development is undertaken, a rapid growth in general cargo shipping can be expected, as the influx of money will stimulate economic development in a manner similar to the opening up of Labrador West.
With respect to the future for pulpwood shipping, the situation is unclear. At present, the pulpwood - in the form of saw logs - is shipped to mill at Stephenville, Newfoundland. This mill, however, is presently considered uneconomic and due to close. A market for wood and wood products - from low quality wood for pulp or foundation pilings, to high quality construction timber - exists in Europe, and the marriage of this market with the timber supply in Labrador appears possible. The form in which the timber would be shipped out however is not certain.

A further area affecting the market for shipping is offshore oil and gas development and here the future is even more obscure. About the best that can be said is that when such development occurs, it will, to an unpredictable extent, induce economic development on the coast of Labrador. This will increase the demand for general cargo shipping, for petroleum product carriers and for crude carriers and/or LNG carriers.

To summarize the present market for shipping in East Labrador is small. The principal commodities are general cargo and refined petroleum products being shipped into the area and pulpwood being shipped out. For the future, the general cargo market may be expected to grow slowly. Major growth would result from the development of the natural resources of the region, particularly ores and offshore oil and gas. Ores from coastal areas would probably be shipped out while the general promotion of economic and industrial development resulting from resource exploitation would increase the market for general cargo and petroleum products shipping.

MARINE TRANSPORTATION ALTERNATIVES

For reasons previously discussed, it is not possible to make definitive statements about 'optimum' solutions to the marine transportation problem. Therefore, in this discussion, general comparisons will be drawn between the characteristics of suitable marine transportation systems and the characteristics of systems which would be suited to market conditions in the absence of the prevailing unusual environmental conditions. In this way an attempt will be made to isolate the effect the environment exerts on marine transportation from the influence of the existing market.

From previous sections, it is apparent that the following types of ships are involved:

- general cargo ships for the importation of all manner of general cargo required to support the local economy;
- small general cargo vessels for local coastal traffic between the principal ports and the outlying communities;
- product carriers for the importation of the petroleum products necessary to support the local economy and local military installations;
- vessels for the exportation of saw logs and/or other forest products;
- ore carriers for the exportation of large volumes of iron ore; and
- crude, product and/or LNG carriers for the exportation of large volumes of crude oil, refined petroleum products or natural gas.

For each of these types, operations are limited by the winter environment and major cost benefits could be associated with extending the shipping season.

The possibility of seasonal traffic exists for goods which can be stockpiled or for goods for which the demand is as seasonal as the traffic. For example, oil may be stored through a winter without deleterious consequence but at high cost while fresh vegetables on the other hand will deteriorate after a relatively short time even if kept in a controlled environment. Consequently, either a year round transportation capability must be available or the demand for fresh vegetables must match their seasonal availability.
Stockpiling costs money; generally fixed investment costs. Such costs are undesirable because they cannot be rapidly reduced nor can the facilities be rapidly expanded to meet fluctuations in demand. The basic trade-off therefore is between required storage capacity and the capacity of the transport units, subject to the constraint of overall average demand.

This trade-off is fundamental to any transportation system employing discrete transportation units (as opposed to a pipeline, for example). The benefits of the economies of scale, which normally accrue to transportation units of large individual capacity, must be balanced against the increased cost of the large storage capacity which is needed to serve such units. In the case of winter shipping operations in Labrador, however, the trade-off is more complicated, for shipping to proceed through the winter ice environment must also be paid. Therefore, offsetting the reduced costs associated with smaller inventories, consequent upon an extended shipping season, not only are there the decreased efficiencies of the individual ships, but also the increased costs of ships designed to deal with the ice environment.

The actual calculation of the economic trade-off for a particular shipping service can become very involved. Reference 1 is a fairly simple example of such a trade-off study, dealing with the economics of an extended shipping season for Great Lakes ore carriers. Without performing such a study however, it can be stated that the influence of the winter ice environment will tend to increase the storage capacity required (for commodities which can be stockpiled) relative to the economic capacity which would exist in the absence of the ice environment. This, in turn, implies either larger ships, faster ships or more ships (or some combination thereof) in order to reduce the inventory to zero during the seasonal shipping period.

To summarize, in the absence of winter ice, a balance based on economics would exist between size, speed, number of ships and the required storage capacity. The effect of the winter ice environment is to place a tax upon short times between shipments. Consequently, the economic optimum moves towards a position of longer times between shipment. This, in turn, requires greater storage capacity. The transport capacity of the ships, though still adequate on average, is insufficient to meet transient requirements and must be modified. This can be done by increasing the size of the ships; their speed; or their number. Of these, the most economic way is to increase size. Thus, the ships will now tend to be larger and either fewer or slower (since the overall average demand is unchanged). The net effect is that the system will move towards a position of fewer, larger shipments, and a new economic equilibrium will be established.

Whatever the economics of the individual trades, there are three basic alternatives for shipping in Labrador: unstrengthened ships; ice strengthened ships; and some combination of cargo ships and icebreaker support.

The advantages of unstrengthened ships are low cost and flexibility. Low cost results both from the lack of ice-strengthening and from the fact that the ships can economically be larger, taking advantage of economies of scale. The flexibility reflects the facts that the ships can be employed economically on other routes, owing to their lack of specialization. On the other hand, the use of unstrengthened ships rules out the possibility of their use in winter and thus they are only practical for cargo which can be stockpiled or for which demand can be made seasonal.

The second alternative, ice-strengthened ships, covers all ships capable of operating independently in ice and not just those which are capable of year-round operations in the Labrador Sea. Thus, it includes ships which are merely structurally
strengthened, as well as those having large propulsion systems in addition to being strengthened. This alternative is expensive if more than a few ships are involved, and the ice-working capability adds so much unnecessary extra weight during the summer season. The ships will also have a lower deadweight-displacement ratio than comparable unstrengthened ships. On the other hand, such ships extend the operating season and can be very flexible in employment during the winter, efficiently accommodating fluctuations in demand.

The third alternative employs a mixture of cargo ships and specialized icebreaking vessels. The benefit of this alternative is that the concentration of icebreaking capability in a few vessels allows the cargo ships to be designed more efficiently from the point of view of cargo carrying. There is a thin line between this and the previous alternative, since the cargo vessels may have some ice-working capability, while the icebreakers may carry some cargo. Further, the merits of specialized assistance in an environment as dynamic as the Labrador Sea in winter must be questioned. Disregarding ice pressure which may halt any vessel, a ship may be temporarily halted by ice conditions yet, particularly in the Labrador environment, be freed by a change in the ice long before she could be reached and broken out by a stronger vessel. In addition, there is the further question of what to do with the icebreaker during the summer. The simple answer is to say that the ship will go north. However the careful matching of icebreaker requirements in different geographical regions during different times of the year is a tricky issue.

A PORT FOR LABRADOR

At present, the only port of any significance in Eastern Labrador is Goose Bay. This port has some major limitations, including draft restrictions and poor accessibility. Various groups have been studying the possibility of developing a deep water port on the Labrador Coast where it would not only be more accessible to shipping, but also ice free for a longer season than Goose Bay. Some of the consequence which supporters of the concept anticipate include:

- natural resource developments in central and western Labrador would become economically more attractive;
- processing of such resources in central Labrador would become economically attractive;
- the economic condition of the Labrador Linerboard operation would be improved, through reduced cost of wood delivered to Stephenville;
- development of the offshore hydrocarbon resources of the Labrador Shelf would be assisted and accelerated;
- the restraining effects of high inventory requirements on the future exploitation of forest resources may be reduced or eliminated;
- an alternative port for shipment of Julian Lake iron ore would become available;
- a site for landing LNG from the Arctic would be established; and
- the social implications for the population would be attractive.

Obviously the development of a port on the coast of Labrador would reduce many of the problems which currently affect shipping in the area. The coastal location would reduce voyage times by about a day, while the lesser degree of ice-infestation would naturally extend the shipping season. The extra depth is perhaps less important at present but will be vital if any large scale movement of ores or oil by sea is to be undertaken in the future.

Exactly how a deep water port in Labrador would affect the desirability of winter
shipping is difficult to assess. In general, the longer ice free season tends to work in favor of unstrengthened, seasonal traffic, since the natural extension of the shipping season makes ships optimized for the open seas economically better adapted to the prevailing conditions. Paradoxically, however, it may work in precisely the opposite way. The natural extension of the season may place the whole system in a position where the marginal cost of extending the open season completely (ie. year-round) becomes small, relative to the benefits which may be obtained thereby.

At present, it is not possible to say for certain which of the above cases holds for offshore Labrador. However, bearing in mind the fact that much of the Labrador trade is or will be in commodities which are partly processed and relatively expensive to stockpile, it appears that the net affect of a port on the Labrador coast would be to increase the attractiveness of winter shipping and encourage the use of strengthened ships operating on a year round basis with icebreaker support during the winter.

One benefit which would be derived from the development of such a port, and which cannot be ignored or excluded from consideration, would be the social implications for the population. Any extension to the present navigation season, however small, would reduce the feeling of total isolation which currently exists. Service on a year-round basis would reduce the dependency of the population on expensive stockpiled consumer goods during the winter. As the area is developed and the population increases, the current methods of auxiliary supply by air will become increasingly more expensive.

Although the volume of goods involved would probably never be sufficient to justify a port development on their own, the social implications could be a major factor in assessing the need.

**SUMMARY AND CONCLUSIONS**

This paper has documented a general investigation into marine transportation in offshore Labrador with particular reference to the effect of the winter ice environment on shipping in the area. Owing to the seasonal nature of the ice environment, the economics of operating all year - either with icebreaking cargo vessels or with icebreaker support - are subject to question. Given the constraints of the Labrador environment and the probable market for shipping in Eastern Labrador, the investigation is intended to assess the general features of marine transportation systems suited to the needs of the Labrador region.

The general nature of this study should be emphasized. Owing to the large number of independent variables to be considered, and the lack of reliable information concerning most of them, it has simply not been possible to be as specific as it would be liked. Therefore, what has been attempted is simply to provide an overview of the situation, by considering the major factors which influence marine transportation in the Labrador region.

From this investigation, it is possible to conclude that the seasonal and many types of shipping is presently the 'best' response to the shipping market in East Labrador and is likely to remain so for some time. This is principally because the relatively small market coupled with the possible ease of stockpiling of the commodities presently involved and does not appear to warrant the extra cost associated with year-round shipping. However as the shipping market grows in response to the general economic development in the region, this situation can be expected to change and year round shipping could become economically viable. Such shipping will
start with a few, specialized trade, carrying cargo for which winter demand is sufficiently strong to warrant the extra cost of winter transportation and will gradually spread to general cargo shipping.

While shipping can thus be expected to remain seasonal for some time, it appears likely that partial extension of the navigation season will occur, utilizing ice-strengthened ships coupled with satellite assisted navigation. Such ships will be capable of limited operations in ice and will rely on satellite data to assist them in choosing a route avoiding local areas of ice beyond their capability. This combination offers a relatively cheap method of extending the navigation season to include periods of time when ice is present but only of a weak and patchy nature.

ACKNOWLEDGEMENTS

The author gratefully appreciates the help of Captain N.A. Baird and Mr. M.P. Dewhurst.

REFERENCES

Benford, Harry, et.al., Cost-Benefit Analysis Model for Great Lakes Bulk Carriers Operating During an Extended Season. Michigan University, Ann Arbor, Department of Naval Architecture and Marine Engineering, Report 114, September 1971.


Statistics Canada, Shipping Reports, Parts II and III (International Seaborne and Coastwise Shipping). Water Transport Section of Transportation and Communications Division, 1973-75.


WIND DIRECTIONS CAPE HARRISON 1955 to 1962 FIGURE III
### TABLE 1
Goose Bay Shipping for 1975

<table>
<thead>
<tr>
<th>Group</th>
<th>Loaded</th>
<th>Unloaded</th>
<th>Total</th>
<th>Net (+ = out)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Foodstuffs</td>
<td>34</td>
<td>2,398</td>
<td>2,432</td>
<td>2,364 (-)</td>
</tr>
<tr>
<td>2) Wood, Construction Mat'ls</td>
<td>243</td>
<td>3,241</td>
<td>3,484</td>
<td>2,998 (-)</td>
</tr>
<tr>
<td>3) Oil, Oil Products</td>
<td>6,383</td>
<td>67,050</td>
<td>73,433</td>
<td>60,667 (-)</td>
</tr>
<tr>
<td>4) Metal Products</td>
<td>2</td>
<td>653</td>
<td>655</td>
<td>651 (-)</td>
</tr>
<tr>
<td>5) Vehicles</td>
<td>200</td>
<td>1,782</td>
<td>1,982</td>
<td>1,582 (-)</td>
</tr>
<tr>
<td>6) Pulpwood</td>
<td>245,690</td>
<td>-</td>
<td>245,690</td>
<td>245,690 (+)</td>
</tr>
<tr>
<td>7) Misc.</td>
<td>593</td>
<td>5,131</td>
<td>5,724</td>
<td>4,538 (-)</td>
</tr>
<tr>
<td><strong>Total Excl. (6)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7,455</td>
<td>80,255</td>
<td>87,710</td>
<td>72,800 (-)</td>
</tr>
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</table>

1 ton = 2000 lbs.

### TABLE 2
Goose Bay Shipping for 1974

<table>
<thead>
<tr>
<th>Group</th>
<th>Loaded</th>
<th>Unloaded</th>
<th>Total</th>
<th>Net (+ = out)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Foodstuffs</td>
<td>7</td>
<td>1,276</td>
<td>1,283</td>
<td>1,269 (-)</td>
</tr>
<tr>
<td>2) Wood, Construction Mat'ls</td>
<td>336</td>
<td>1,019</td>
<td>1,355</td>
<td>683 (-)</td>
</tr>
<tr>
<td>3) Oil, Oil Products</td>
<td>14,865</td>
<td>107,594</td>
<td>122,459</td>
<td>92,729 (-)</td>
</tr>
<tr>
<td>4) Metal Products</td>
<td>7</td>
<td>61</td>
<td>68</td>
<td>54 (-)</td>
</tr>
<tr>
<td>5) Vehicles</td>
<td>255</td>
<td>728</td>
<td>983</td>
<td>473 (-)</td>
</tr>
<tr>
<td>6) Pulpwood</td>
<td>226,338</td>
<td>-</td>
<td>226,338</td>
<td>226,338 (+)</td>
</tr>
<tr>
<td>7) Misc.</td>
<td>789</td>
<td>3,017</td>
<td>3,806</td>
<td>2,228 (-)</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>242,597</td>
<td>113,695</td>
<td>356,292</td>
<td>128,902 (+)</td>
</tr>
</tbody>
</table>

### TABLE 3
Goose Bay Shipping for 1973

<table>
<thead>
<tr>
<th>Group</th>
<th>Loaded</th>
<th>Unloaded</th>
<th>Total</th>
<th>Net (+ = out)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Foodstuffs</td>
<td>65</td>
<td>4,287</td>
<td>4,352</td>
<td>4,222 (-)</td>
</tr>
<tr>
<td>2) Wood, Construction Mat'ls</td>
<td>617</td>
<td>1,318</td>
<td>1,935</td>
<td>701 (-)</td>
</tr>
<tr>
<td>3) Oil, Oil Products</td>
<td>14,127</td>
<td>75,427</td>
<td>89,554</td>
<td>61,300 (-)</td>
</tr>
<tr>
<td>4) Metal Products</td>
<td>63</td>
<td>909</td>
<td>972</td>
<td>846 (-)</td>
</tr>
<tr>
<td>5) Vehicles</td>
<td>370</td>
<td>2,220</td>
<td>2,590</td>
<td>1,850 (-)</td>
</tr>
<tr>
<td>6) Pulpwood</td>
<td>157,768</td>
<td>-</td>
<td>157,768</td>
<td>157,768 (+)</td>
</tr>
<tr>
<td>7) Misc.</td>
<td>877</td>
<td>4,636</td>
<td>5,513</td>
<td>3,759 (-)</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>173,887</td>
<td>88,797</td>
<td>262,684</td>
<td>85,090 (+)</td>
</tr>
</tbody>
</table>
RESULTS OF FULL SCALE ICE IMPACT LOAD STUDIES ABOARD C.C.G.S. NORMAN McLEOD ROGERS

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C. Pellegrin, Transport Canada, Ottawa, Canada

INTRODUCTION

Before 1960, the St. Lawrence River up to Montreal was virtually closed to commercial navigation three months per year. However, with the implementation by the Federal Department of Transport of more efficient ice control and icebreaking measures, winter navigation had become a fait accompli in the late sixties.

Several hundred vessels are sailing the St. Lawrence System from the Gulf to the Port of Montreal every winter and the large increase in traffic in ice is posing new safety problems. These are made more acute by the fact that, as a result of new vessel technology and channel improvements, more and more of the vessels entering the System in winter are fast or deep draft vessels which have to operate at high speed to maintain a rigid schedule (containerships) or take advantage of the tide in some critical reaches (large tankers). Since such vessels are generally not ice strengthened, they are more exposed to structural damage when they strike large ice floes than slower conventional vessels.

To keep up with the high safety conditions prevailing in the System, The Department of Transport has to consider the development of new standards and guidelines which would form the basis of regulatory measures governing the movement of vessels in ice infested waters. As available information on the interaction between moving ice floes and vessels was scarce, the Departments initiated in 1974, an overall study of ice problems affecting the movement of ships in the Gulf of St. Lawrence and the St. Lawrence River and Estuary. The data obtained during the study will also have a direct application in hull design for ships operating in ice not only in the St. Lawrence System but also in Arctic waters.

The study included the assessment of ice conditions in critical reaches, the development of an analytical predictive model, full scale ice impact load acquisition and physical modelling of ice impact loading.

The mathematical model was completed in April 1974 (Major et al, 1975). In general, the model predicted impact loads for two ships which had been field tested (Edwards et al, 1972) (Levine, 1973) and which were higher than the observed loads. It was assumed that this discrepancy was due to the fact that the model predicts worst case loads for "faultless" ice while the ship rarely encounters such conditions.

The next step in the program was to acquire full scale impact load data for St. Lawrence River ice conditions, preferable on a commercial vessel.
Because commercial ships normally avoid ice and actually spend very little of their total operational hours in ice, a decision was taken to use a Canadian Coast Guard Icebreaker as a surrogate for a commercial ship and to subject it to ice conditions similar to those which a commercial ship would encounter entering the St. Lawrence River. The intent was to install instrumentation in the ship which would permit the measurement of the external load encountered by the ship's hull during impact with an ice feature. This was accomplished by judiciously placing strain gauges on portions of the internal structure. During a calibration period, a known external force was to be applied to the hull to permit later estimation of external force by interpreting internal strains. Simultaneous with strain load measurements, initial ship speed, and ice characteristics were to be measured.

**INSTRUMENTATION AND TESTING**

The original intent was to investigate the interaction between the ship and isolated floes primarily in the Lower St. Lawrence River region. Plans were made to measure floe size, depth and compressive strength with support from a helicopter assigned to the ship.

The C.C.G.S. NORMAN McLEOD ROGERS was chosen as the test vehicle for the following reasons:

a) She is a powerful ship and therefore capable of entering relatively heavy ice.
b) She is Quebec based and readily accessible.
c) Her hull shape at her normal operating draft is characterized by steep buttock and section slopes providing the possibility of high loads.

The instrumentation system, which is described more fully in Appendix A, consisted of the following elements:

a) 12 gauge arrays of strain gauges P&S to measure hull strain on impact with ice.
b) Strain gauges on propeller shaft to measure torque.
c) Magnets and pick-up on tail shafts to measure shaft speed.
d) L.I. T.V. camera on mast to observe general ice conditions and to measure speed of ship.

The strain gauge outputs were recorded on an FM tape recorder and oscillograph recorder along with the shaft rpm. The T.V. output was fed to a video tape recorder.

Calibration of the hull to relate internally sensed strains to external loads was carried out at the end of the test program in late March. An unique pendulum system was designed to permit application of large loads to the hull without external support. The methods used to calibrate the hull are described in Appendix B.

Due to the severity of the winter, the C.C.G.S. NORMAN McLEOD ROGERS was never released from duty in the Montreal area nor permitted to undertake the test program planned for the Lower St. Lawrence. Upon completion of her work in Lake St. Peter, she proceeded directly to open up the St. Lawrence Seaway, starting by clearing out the South Shore Canal.

A test team was assigned to the ship for the first two weeks in March 1976 to acquire impact load data in the course of her icebreaking operations on a not-to-interfere basis. The two-man shipboard team operated the instrumentation continuously during this period while three technicians gathered ice thickness and compressive strength data along the South Shore Canal independent of the ship.
During operations in the South Shore Canal, the C.C.G.S. NORMAN McLEOD ROGERS backed up and rammed the ice repeatedly, progressing one half to several ship lengths at each ram. The ice thickness ranged from 51 cm to 69 cm. Impact speed ranged from 3 to 5 meters per second.

The test procedure during these operations was as follows:

1. As the ship accelerated toward the ice edge, the masthead television camera was started and switching circuits arranged to record either port or starboard strain gauge arrays on the FM tape recorder. The run number, time of day and name of panel to be recorded and FM recorder counter reading was read into the audio channel of the video tape recorder.

2. When the ice edge was in sight on the video monitor, the FM tape recorder and oscillograph were started. The appropriate side camera was then switched into the video tape recorder to provide coverage of the impact.

3. When the ship stopped forward motion, the FM recorder and oscillograph recorders were stopped and the time of the run, its consecutive number and the video and FM tape counter readings were read into the audio channel of the video recorder which was then stopped.

The low light level masthead camera worked well and proved to be an excellent tool for measuring speed during ramming.

A total of 84 ramming runs were made. One hundred and twenty-two of the impact incidents recorded during those runs were extracted from the records in the form of tables of peak strains and subjected to data reduction and analysis.

RESULTS AND ANALYSIS

Eighty-four ramming impact runs were made between March 4 and 12, 1976, in the South Shore Canal and Lake St. Louis area. Table 1 lists the runs, calendar dates and prevailing ice thickness. One hundred and twenty-two impact incidents were extracted from those runs which were considered suitable for analysis. Seventy-eight occurred on the starboard side and forty-four on the port side.

The results of the C.C.G.S. NORMAN McLEOD ROGERS' trials are summarized in Table 2. In that table, the data has been grouped into velocity ranges and thickness ranges. The maximum values of impact load have been tabulated for each speed and thickness category as well. It may be seen in this table that the impact loads encountered by the C.C.G.S. NORMAN McLEOD ROGERS exceed, on occasion, 1,000,000 newtons in ice approximately 65 cm thick.

The output of the mathematical model developed in 1974 (Major et al, 1974) is plotted in Figure 1 together with the plots of maximum values of impact forces observed for the categories of ice thickness and speed listed in Table 2. The mathematical model was run for a strength value of 960 kilopascals, thickness of 64 cm and for various speeds. The mathematical model results do not exhibit the same sensitivity to velocity as the full scale data. The higher set of math model results are for the port side. The lower line is for the starboard side which is slightly more inclined than the port side. The magnitude of the model predictions is greater than the field observations. Nonetheless, considering the paucity of full scale observations, and the fact that the "math model should predict the extreme case", the relative agreement between the model and the data does not seem unreasonable.
The impact loads, ship speed and ice thickness for the C.C.G.S. NORMAN McLEOD ROGERS have been converted to the dimensionless numbers used (Edwards et al, 1972). The results are plotted in Figure 2. Also shown on the plot are the results of the ice impact studies on S.S. LEON FRASER, a wall-sided ore carrier and on U.S.C.G.C. MACKINAW, an icebreaker with fine forebody shape.

In Table 3, the angles and direction cosines for the hulls in question are listed.

The influence of hull shape is quite clearly seen in Figure 2 where the data for C.C.G.S. NORMAN McLEOD ROGERS lies between that for the two other ships. The value of the ROGERS' direction cosines also lie between those of the MACKINAW and LEON FRASER.

CONCLUSIONS

Based upon the results of the ramming impact load tests, we may conclude the following:

1. Ice impact loads can be measured by suitably instrumenting a ship's hull. Furthermore, under circumstances where a team can operate independent of the ship, useful data can be obtained using techniques developed in this project, without interfering with ship's operations.

2. Ice impact loads in level ice are extremely sensitive to hull shape, particularly buttock slopes and section slopes at the water line.

3. Stresses in the frames of C.C.G.S. NORMAN McLEOD ROGERS reached levels as high as 117,000 kilopascals when ramming ice only 61-69 cm thick. There is no guarantee that the sampled portion of the frame was indeed the point of maximum stress.

4. Totally satisfactory agreement between the mathematical predictions of ice impact load and observations has not been achieved. However, the proximity of the predictions to the observations for the C.C.G.S.NORMAN McLEOD ROGERS is encouraging and suggests that further refinement of the math model will produce a useful tool for predicting the level of external ice force for commercial ships penetrating level ice floes. Such a situation can easily be envisioned for a large ship encountering large floes in the one to two foot depth range.

5. Based upon the dimensionless plot in Figure 2, we conclude that a ship, with hull parameters similar to an ore carrier (LEON FRASER) could encounter loads four to five times as high as those encountered by C.C.G.S. NORMAN McLEOD ROGERS in similar ice conditions at similar speeds. This implies that such a ship, if forced to penetrate into a large ice floe, 60 cm thick at say, 3 to 5 m/sec, might experience loads in excess of 6.6 million newtons over relatively concentrated areas. Conventional ships structures can be expected to fail under such loads.

6. The C.C.G.S. NORMAN McLEOD ROGERS proved to be an excellent vehicle for impact load tests oriented toward commercial ship vulnerability. Although the structure is not easy to analyze, it is light enough to allow high levels of strain for impact loads commonly encountered in St. Lawrence ice conditions. Furthermore, the hull shape represents an excellent intermediate point (from a testing point of view) between a good icebreaking form and a cargo ship. The decision to instrument this ship was well taken,
in spite of her heavy operational commitments.

7. Despite the fact that the initial goal of this project was not achieved, (i.e. floe impact model correlation), the ramming penetration data which was acquired has provided much needed ice impact test results for an intermediate hull shape.

ACKNOWLEDGEMENTS

The work described in this paper was commissioned and funded by Transport Canada, Waterways Development.

The experiment was carried out with a great deal of assistance from the Officers and Crew of the C.C.G.S. NORMAN McLEOD ROGERS. The two captains of the ship during the trials, period, Captain Vezina and Captain Toomey were extremely helpful as were the Regional Fleet Superintendents, Captain Pelletier and Captain Moreau.

REFERENCES


APPENDIX A

INSTRUMENTATION SYSTEM

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>MEASUREMENT SYSTEM</th>
<th>RECORDING SYSTEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hull Strains</td>
<td>12 gauge array on both starboard and port sides</td>
<td>Oscillograph recorder and FM tape recorder</td>
</tr>
<tr>
<td>Propulsion Torque</td>
<td>Propeller shaft strain gauged</td>
<td>Oscillograph recorder</td>
</tr>
<tr>
<td>Shaft rpm</td>
<td>Magnets and pick up on tail shaft</td>
<td>Oscillograph recorder</td>
</tr>
<tr>
<td>Ship Draft</td>
<td>Draft marks</td>
<td>Data sheet</td>
</tr>
<tr>
<td>Ship Speed</td>
<td>L.L. T.V. Canera</td>
<td>Video tape recorder</td>
</tr>
<tr>
<td>Ice Characteristics</td>
<td>Ice drill holes and hydraulic jacks</td>
<td>Field book</td>
</tr>
</tbody>
</table>

Instrumentation layout is shown in Figure A-1.

APPENDIX B

CALIBRATION OF STRAIN GAUGE PANELS

REVIEW OF CALIBRATION TECHNIQUE

The technique for measuring ice impact forces on the hulls of the icebreaker which was used in this study, involved strain gauging frames of the hull structure in selected locations and a method for converting such measurements into the corresponding magnitudes of the ice impact forces. This conversion of strain readings into forces may be accomplished by means of an external calibration. The calibration is achieved by loading the hull of the ship with known forces and measuring the response of the strain gauges.

When this calibration technique was employed for tests with the U.S.C.C.C. MACKINAW (Edwards et al, 1972), the ship was afloat and the calibration was performed by applying load with an hydraulic jack from a floating raft which was braced against a set of pilings. There was some difficulty in keeping the ship from moving when the load was applied by an hydraulic jack, whereas, when the calibration testing was done for SS LEON C. FRASER (Levine, 1973), the ship was in dry dock. Since dry-docking of the C.C.G.S. NORMAN McLEOD ROGERS was not envisioned, a technique was tried which was planned for implementation on the trials of the POLAR STAR.

For calibration strain gauges on the POLAR STAR Arctec Incorporated proposed a swinging pendulum to be used as a method of load application. This has the advantage of not requiring dry docking and avoids the problem of restraining the ship and providing an adequate foundation for an hydraulic jack.

EXPERIENCE WITH THE PENDULUM

The pendulum was tried during January while the C.C.G.S. NORMAN McLEOD ROGERS was moored in Three Rivers, Quebec. The impact times appeared to be very low and were
unreadable on the oscillograph recorder. It was concluded that the plate on the pendulum was striking the hull obliquely imparting a very sharp impulse. The ship's structure was then resonating at high frequency.

To correct this misalignment problem, a series of rubber squares were glued to the face of the pendulum. In a range of deformation which corresponds to the maximum achievable pressure on the pendulum (221,500N/.372m² = 596 kilopascals), the "spring constant" of the pads was 14085 N/cm. The spring constant for the sufficient pads in parallel to cover the face of the pendulum was 662,756 N/cm or 6.6x10⁷ N/m. This is twenty times the spring constant for the main springs in the pendulum. The addition of the rubber pads apparently eliminated the alignment problem because, the oscillograph deflections were readily readable and had typical impact times close to the predicted value of 0.035 seconds.

Another problem in using the pendulum was that the pivot point could not be restrained sufficiently to prevent it from being displaced by centripetal force as the pendulum accelerated. The seriousness of this problem became evident when the calibration data was plotted. The plots of observed strain versus the initial angle of the pendulum should be almost linear because:

\[ F_c = \sqrt{2wg\theta Z} \]

\[ Z = R (1 - \cos \theta) \]

\[ R = \text{pendulum arm length} \]

\[ W = \text{pendulum weight} \]

\[ \theta = \text{initial angle of the pendulum} \]

\[ F_c = \sqrt{2wgR} \cdot \sqrt{1 - \cos \theta} \]  (1)

It turns out that \( \sqrt{1 - \cos \theta} \) is almost linear in \( \theta \). However, the observed strain was very non-linear with \( \theta \) at large values of \( \theta \) (pendulum lifted very high). Consequently, it was necessary to use only the initial slope of the strain versus pendulum angle plots.

In the future, the pendulum must be securely restrained at the pivot to avoid this problem.

**STRAIN GAUGE CALIBRATIONS**

The first step in the calibration process was to employ the impact pendulum to impart a force to the hull. The actual ramming impact tests were conducted at one water line (5.5 m forward). Consequently, the pendulum was set up to place 90\% of its height (60 cm) below the 5.5 m water line to simulate ice impact.

The longitudinal location of the pendulum was varied. At each location, the pendulum was raised to four different angles. The strain at impact in each gauge was recorded. Before each calibration run, a calibration resistor was switched in parallel with each gauge to simulate a known strain. The peak values of oscillograph galvanometer deflection for each impact were extracted from the records as were the values of oscillograph deflection for the calibration resistor. The ratio of the peak deflection due to impact to the calibration deflection was plotted versus the pendulum angle.
The slopes of these lines are the values of the sensitivity of the individual gauges to external load. The value of the slope \( \frac{\varepsilon}{P} \) for each gauge depends upon the distance of the impact location from the particular gauge and upon the stiffness of the local structure. Since the calibration and the ice impacts were restricted to the 5.5 meter water line, the linear distance from the gauge location (parallel to the water line) to the impact location may be considered to be the only parameter which influences the value of \( \frac{\varepsilon}{P} \) for each gauge. For each gauge, a calibration function was obtained of the following form:

\[
\frac{\varepsilon_1}{P} = \delta_1(X_1)
\]  

(2)

where:

- \( \varepsilon_1 \) = measured strain in gauge 1
- \( P \) = pendulum impact pressure
- \( X_1 \) = linear distance from gauge 1 to impact point.

### TABLE 1

<table>
<thead>
<tr>
<th>RUN NUMBERS</th>
<th>DATE</th>
<th>LOCATION</th>
<th>RANGE OF MEASURED ICE THICKNESS (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-21</td>
<td>4 March</td>
<td>St. Lambert</td>
<td>50 est.</td>
</tr>
<tr>
<td>22-42</td>
<td>8 March</td>
<td>St. Lambert - Cote St. Catherine Lock</td>
<td>50 est.</td>
</tr>
<tr>
<td>43-56</td>
<td>9 March</td>
<td>Cote St. Catherine - CPR Bridge</td>
<td>60-66</td>
</tr>
<tr>
<td>57-74</td>
<td>10 March</td>
<td>CPR Bridge - Lake St. Louis</td>
<td>60-69</td>
</tr>
<tr>
<td>75-84</td>
<td>12 March</td>
<td>Downstream of Beauharnois</td>
<td>46-50 est.</td>
</tr>
</tbody>
</table>
### TABLE 2

**SUMMARY OF IMPACT DATA FOR CCGS "N.M. ROGERS"**

<table>
<thead>
<tr>
<th>SIDE</th>
<th>MEAN ICE DEPTH (cm)</th>
<th>VELOCITY RANGE (m/s)</th>
<th>MEAN IMPACT FORCE (N)</th>
<th>MAX (N)</th>
<th>( \frac{F_i}{\rho_w g^2 h^4} )</th>
<th>( \frac{i \alpha_1 G}{\sqrt{g h}} )</th>
<th>( \frac{1}{\rho_w (h v)^4} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port</td>
<td>65</td>
<td>0-1</td>
<td>309,391</td>
<td>558,180</td>
<td>.296</td>
<td>209.1</td>
<td>3.99</td>
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<tr>
<td></td>
<td>65</td>
<td>1-1.5</td>
<td>495,296</td>
<td>733,976</td>
<td>.79</td>
<td>274</td>
<td>10.63</td>
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<tr>
<td></td>
<td>65</td>
<td>1.5-2</td>
<td>331,355</td>
<td>524,291</td>
<td>1.18</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>2-2.7</td>
<td>229,983</td>
<td>319,208</td>
<td>1.58</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>2.7-3.3</td>
<td>175,862</td>
<td>319,208</td>
<td>1.97</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>3.3-4.6</td>
<td>283,343</td>
<td>913,944</td>
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<td>367.</td>
<td>34.5</td>
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<td></td>
<td>65</td>
<td>4.6-5.2</td>
<td>293,098</td>
<td>418,710</td>
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<tr>
<td></td>
<td>48</td>
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<td>63,916</td>
<td>84,392</td>
<td>.588</td>
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<td>-</td>
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<tr>
<td></td>
<td>51</td>
<td>2-2.7</td>
<td>115,760</td>
<td>234,334</td>
<td>1.24</td>
<td>-</td>
<td>-</td>
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<tr>
<td>STBD</td>
<td>51</td>
<td>2.7-3.3</td>
<td>386,562</td>
<td>481,191</td>
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<td>374.</td>
<td>68.6</td>
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<tr>
<td></td>
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<td>-</td>
<td>-</td>
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<td>532,109</td>
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<td>3.3-4.6</td>
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<td>2.32</td>
<td>-</td>
<td>-</td>
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<td>1-1.5</td>
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<td>261,370</td>
<td>0.77</td>
<td>-</td>
<td>-</td>
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<tr>
<td></td>
<td>63.5</td>
<td>2-2.7</td>
<td>572,250</td>
<td>675,216</td>
<td>1.55</td>
<td>-</td>
<td>-</td>
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<tr>
<td></td>
<td>48</td>
<td>2.7-3.3</td>
<td>314,752</td>
<td>718,936</td>
<td>1.47</td>
<td>658.</td>
<td>74.3</td>
</tr>
</tbody>
</table>

1. \( \sigma_f \) = 960 Kilopascals used for all runs

2. Only the data points which define the limits of the impact loads were converted to dimensionless terms

- \( F_i \) = maximum measured impact force
- \( \alpha_1 \) = direction cosine of hull (see figure 2)
- \( h \) = ice thickness
- \( G \) = flexural strength of ice
- \( \rho_w \) = mass density of water
- \( v \) = ship speed at impact
- \( g \) = gravitational constant
- \( h_v \) = units of m²/sec

448
<table>
<thead>
<tr>
<th>SHIP</th>
<th>FRAME</th>
<th>ANGLE IN DEGREES</th>
<th>DIRECTION COSINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>CCGS N.M. ROGERS</td>
<td>117</td>
<td>15 8.5 29</td>
<td>.263 .326</td>
</tr>
<tr>
<td>CCGS N.M. ROGERS</td>
<td>101.5</td>
<td>8 6.5 39</td>
<td>.14 .218</td>
</tr>
<tr>
<td>LEON FRASER</td>
<td>-</td>
<td>37.5 7.8 8.5</td>
<td>.606 .091</td>
</tr>
<tr>
<td>USCGS MACKINAW</td>
<td>30</td>
<td>20 26 53.3</td>
<td>.30 .416</td>
</tr>
</tbody>
</table>
FIGURE 1
IMPACT LOAD
versus
ICE THICKNESS x SHIP SPEED

△ PORT SIDE
□ STARBOARD SIDE

MATH MODEL PORT
MATH MODEL STARBOARD

ICE FORCE (MN)

ICE THICKNESS x VELOCITY
(m²/sec)

UPPER LIMIT OF EXPERIMENTAL RESULTS
FIGURE 2

PLOT OF DIMENSIONLESS IMPACT LOAD VERSUS DIMENSIONLESS VELOCITY

$\frac{F_i}{\rho vgh^{\frac{3}{2}}}$

Slope = 1.2

Data from LEON FRASER 1973
$\alpha_3 = 0.0915$

CCGS N. McLEOD ROGERS Test of 1976
$\alpha_3 = 0.216$ to 0.326

Extrapolated data line of USCG MACKINAW
$\alpha_3 = .416$

Data from CCGS NORMAN McLEOD ROGERS 1976

Data from USCG MACKINAW 1971

Data from LEON FRASER 1973

$\frac{\sigma_1}{\sqrt{gh}}$ $\frac{\sigma}{\rho vgh^{\frac{3}{2}}}$
FIGURE A-1
INSTRUMENTATION LAYOUT

HULL STRAIN
24 semi-conductor strain gauges
12 active
12 dummy

PORT

MAST
HEAD
TV
CAMERA

HULL STRAIN
24 semi-conductor strain gauges
12 active
12 dummy

STARBOARD

FM receive

XMTR 2.5v batt
4 active arm strain gauge

SHAFT TORQUE

FM tape recorder

ACCUDATA 118

O'graph recorder

O'graph recorder

PORT RPM

STBD RPM

SHAFT SPEED

SHAFT SPEED
HYDRAULIC MODELING OF ICE-COVERED WATERS

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9104 Red Branch Road
Columbia, Maryland 21045
United States

ABSTRACT

Hydraulic modeling can play an important role in solving problems associated with ice-covered waters if it is shown to scale prototype conditions accurately. With this in mind, the purpose of this paper is three-fold. The first is to present unified sets of scaling laws for both distorted and undistorted hydraulic models of ice-covered waters. Secondly, a unique modeling material, which exhibits similarity with prototype ice properties, and its use in hydraulic models of portions of the St. Lawrence Seaway are described. Finally, the role that hydraulic modeling can play in solving ice-related problems is examined.

BACKGROUND

In solving engineering design problems, several approaches are available as alternatives to the actual building and testing of a full-scale prototype which is often too large and expensive for practical experimentation. The least expensive approach and the one requiring a minimum of time and effort is to use an existing mathematical or empirical formulation which has been proven by past experience to yield accurate predictions. If the existing formulation is not adequate, or if its derivation is based upon simplifying assumptions which either restrict the prediction or question its accuracy or reliability, a new and more sophisticated formulation may have to be developed. Often, the problem is either too complex or not sufficiently researched for the mathematical model to predict real world conditions with confidence. In these cases, it may be necessary to construct and test a physical model in order to obtain the needed information or to test the feasibility of a proposed solution. However, it is important to note that physical and mathematical modeling should not be completely separated since no physical phenomenon should be modeled without a preliminary analysis to determine the physical laws which govern the phenomenon and to gain deeper insight into the problem. On the other hand, physical modeling can add immeasurably to the development and experimental validation of an unproven mathematical model.

Hydraulic models have long been used as a design tool by engineers to aid in the development of effective and efficient hydraulic structures and systems. As with any physical model, hydraulic models can give reliable data only if they are designed in accordance with the governing scaling laws and are shown to scale prototype conditions accurately. If the model does not obey these scaling laws, the model will give erroneous results and the use of the most sophisticated instrumentation and equipment can only increase the accuracy of incorrect predictions.
DEVELOPMENT OF SCALING LAWS

A summary of the scaling laws for both undistorted and distorted hydraulic models in which gravity is the predominant force causing motion (Froude Scaling) is listed in Table 1. To facilitate the development of these scaling laws, undistorted and distorted models are considered separately in the following two sections.

Undistorted Hydraulic Models

In comparing prototype and model, three types of similitude between the model and the prototype are generally considered:

GEOMETRIC SIMILITUDE: The ratios of all similar length dimensions are equal to a constant ($\lambda$).

KINEMATIC SIMILITUDE: All similar particles travel geometrically similar paths and the ratios of the required time periods are equal to a constant ($\lambda_t$).

DYNAMIC SIMILITUDE: In addition to kinematic similitude, the ratios of similar masses are equal to a constant ($\lambda_m$) and the ratios of similar accelerating forces are equal to a constant ($\lambda_f$).

While complete dynamic similitude is always a goal in physical modeling, it is seldom obtained in practice. This limitation, however, does not hinder the usefulness of the results, provided the deviations are of secondary importance.

Using Newton's Second Law of Motion ($F = M \cdot a$) and noting that the ratio of masses ($\lambda_m$) and accelerations in the prototype and model equal $\frac{\rho_p}{\rho_m} \cdot \lambda^3$ and $\lambda/\lambda_t^2$ respectively, $\lambda_f$ can be expressed in terms of $\lambda$ and $\lambda_t$:

$$\lambda_f = \frac{F_p}{F_m} = \frac{M_p}{M_m} \cdot \frac{\rho_p}{\rho_m} \cdot \lambda^3 = \lambda/\lambda_t^2$$

where $\rho_p$ and $\rho_m$ are the corresponding mass densities in the prototype and model. If water is the testing fluid, then $\rho_p$ and $\rho_m$ are equal and Equation (1) reduces to:

$$\lambda_f = \lambda^4/\lambda_t^2$$

For most hydraulic models of water systems, gravity is the predominant force causing motion. It is therefore desirable to have the ratio of gravitational forces ($\lambda^3$) equal to the ratio of inertial or accelerating forces given by Equation (2), thus,

$$\lambda_f = \lambda^3 = \frac{\lambda^4}{\lambda_t^2}$$

Solving Equation (3) for $\lambda_t$ yields

$$\lambda_t = \sqrt[3]{\lambda}$$
<table>
<thead>
<tr>
<th>VARIABLE</th>
<th>SYMBOL</th>
<th>UNDISTORTED MODEL*</th>
<th>DISTORTED MODEL*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal scale</td>
<td>$\lambda$</td>
<td>$\lambda \ (\geq 1)$</td>
<td>$\lambda \ (\geq 1)$</td>
</tr>
<tr>
<td>Vertical scale</td>
<td>$\beta$</td>
<td>$\lambda$</td>
<td>$\beta \ (\geq 1)$</td>
</tr>
<tr>
<td>Length, horizontal</td>
<td>$L_H$</td>
<td>$\lambda$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Length, vertical</td>
<td>$L_V$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Area, horizontal</td>
<td>$A_H$</td>
<td>$\lambda^2$</td>
<td>$\lambda^2$</td>
</tr>
<tr>
<td>Area, vertical</td>
<td>$A_V$</td>
<td>$\lambda^2$</td>
<td>$\lambda\beta$</td>
</tr>
<tr>
<td>Volume</td>
<td>$V$</td>
<td>$\lambda^3$</td>
<td>$\lambda^3\beta$</td>
</tr>
<tr>
<td>Time</td>
<td>$t$</td>
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<td>$\lambda/\beta^{1/2}$</td>
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<tr>
<td>Mass</td>
<td>$M$</td>
<td>$\lambda^3$</td>
<td>$\lambda^2\beta$</td>
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<tr>
<td>Density (all)</td>
<td>$\rho$</td>
<td>$1$</td>
<td>$1$</td>
</tr>
<tr>
<td>Water velocity</td>
<td>$V_w$</td>
<td>$\lambda^{1/2}$</td>
<td>$\beta^{1/2}$</td>
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<tr>
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<td>$Q$</td>
<td>$\lambda^{5/2}$</td>
<td>$\lambda\beta^{3/2}$</td>
</tr>
<tr>
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<td>$\lambda$</td>
<td>$\beta$</td>
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<td>River slope</td>
<td>$S$</td>
<td>$1$</td>
<td>$\beta/\lambda$</td>
</tr>
<tr>
<td>Bed friction factor</td>
<td>$f_d$</td>
<td>$1$</td>
<td>$\beta/\lambda$</td>
</tr>
<tr>
<td>Ship breadth</td>
<td>$B_s$</td>
<td>$\lambda$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Ship length</td>
<td>$L_s$</td>
<td>$\lambda$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Ship draft</td>
<td>$H$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Ship velocity</td>
<td>$V_s$</td>
<td>$\lambda^{1/2}$</td>
<td>$\beta^{1/2}$</td>
</tr>
<tr>
<td>Ice cover thickness</td>
<td>$h$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Ice cover length</td>
<td>$L_i$</td>
<td>$\lambda$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Ice cover breadth</td>
<td>$B_i$</td>
<td>$\lambda$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Ice cover friction factor</td>
<td>$f_t$</td>
<td>$1$</td>
<td>$\beta/\lambda$</td>
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<tr>
<td>Ice cover flexural strength</td>
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<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Ice cover crushing strength</td>
<td>$\sigma_c$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Ice cover shearing strength</td>
<td>$\sigma_s$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
</tr>
<tr>
<td>Ice cover elastic modulus</td>
<td>$E$</td>
<td>$\lambda$</td>
<td>$\beta$</td>
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* Values are the ratio of prototype value to model value.
<table>
<thead>
<tr>
<th>VARIABLE</th>
<th>SYMBOL</th>
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<td>\lambda</td>
<td>\beta</td>
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<td>Water drag force ((~f_{L_B}V^2))</td>
<td>(F_W)</td>
<td>\lambda^3</td>
<td>\lambda \beta^2</td>
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<tr>
<td>Ship resistance (B h^2)</td>
<td>(R_{\text{ship}})</td>
<td>\lambda^3</td>
<td>\lambda \beta^2</td>
</tr>
<tr>
<td>Crushing force (B h)</td>
<td>(F_C)</td>
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<td>\lambda \beta^2</td>
</tr>
<tr>
<td>Shearing force (B h)</td>
<td>(F_S)</td>
<td>\lambda^3</td>
<td>\lambda \beta^2</td>
</tr>
<tr>
<td>Buckling force (\rho g B L h^{3/2})</td>
<td>(F_B)</td>
<td>\lambda^3</td>
<td>\lambda \beta^2</td>
</tr>
<tr>
<td>Gravity force (\rho g L B h S)</td>
<td>(F_G)</td>
<td>\lambda^3</td>
<td>\lambda \beta^2</td>
</tr>
</tbody>
</table>

Using these scale relationships for time, length, and force, the scaling laws for prototype and model characteristics, such as area, volume, river slope, velocity, discharge, acceleration, friction factor, energy, and momentum, can be readily derived.

In order to properly model the total system, all the parts must also be properly scaled so that the total system obeys a unified set of scaling laws. More specifically, the prototype and model of the ice cover and ship or marine structure should also possess geometric, kinematic, and dynamic similitude. As a result, all length dimensions, such as thickness, breadth, and length of the ice cover and length, breadth, and depth of the ship and marine structures, should scale by \(\lambda\) and the velocities and accelerations, by \(\sqrt{\lambda}\) and unity respectively. For dynamic similitude to be obeyed, all forces must also scale by \(\lambda^3\). Imposing this condition establishes the necessary scaling laws for mechanical properties of ice. To illustrate this, consider the force required to crush a cylinder of diameter \(d\) through an ice sheet of thickness \(h\) and crushing strength \(\sigma\). The ratio of crushing forces in the prototype and model can be expressed as:

\[
\frac{F_{\text{crush}_p}}{F_{\text{crush}_m}} = \left(\frac{\sigma_p}{\sigma_m}\right) \cdot \left(\frac{d_p}{d_m}\right) \cdot \left(\frac{h_p}{h_m}\right) = \lambda^2 \cdot \left(\frac{\sigma_p}{\sigma_m}\right)
\]

Thus, for the ratio of crushing forces in the prototype and model to be equal to \(\lambda^3\), the ratio of crushing strengths \((\sigma_p / \sigma_m)\) must be equal to \(\lambda\). By employing similar procedures for shearing forces, buckling forces, buoyancy forces, water drag forces, and ship resistance forces, scaling laws for all the mechanical ice properties can be derived.

Symbol, \(\sim\), means "proportional to".
Distorted Hydraulic Models

It is frequently necessary to design a distorted hydraulic model with a smaller vertical scale ($\beta$) than horizontal scale ($\lambda$) as a result of either area or economic constraints. In these hydraulic models, all horizontal length dimensions, such as length and breadth of river, ship, and ice, scale by $\lambda$ and all vertical length dimensions, such as water depth, ice thickness, and ship depth, scale by $\beta$.

In any distorted or undistorted hydraulic model in which gravity is the predominant force causing motion, it is important that the model reproduce the conversion of potential energy to kinetic energy, and vice versa, as they occur in the prototype. For this condition to exist:

\[
\frac{(PE)_P}{(PE)_m} = \frac{(KE)_P}{(KE)_m}
\]

where $PE = $ Potential Energy = $M \cdot g \cdot H$

$KE = $ Kinetic Energy = $1/2 \cdot M \cdot V^2$

Substituting these expressions for $PE$ and $KE$ into Equation (6) yields the following relationship for the ratio of the water velocities:

\[
\frac{V_P}{V_m} = \sqrt{\beta}
\]

Using this scaling law for water velocity, the ratio of discharge and time for the prototype and model can be expressed as follows:

\[
\frac{Q_P}{Q_m} = \lambda \beta^{3/2}
\]

\[
\frac{t_P}{t_m} = \lambda / \sqrt{\beta}
\]

As with the undistorted hydraulic models, it is necessary for all primary governing forces to obey the same scaling laws if quantitative data, as well as the qualitative data, is to be obtained from the hydraulic model. If this cannot be achieved, it becomes extremely difficult, if not impossible, to predict prototype forces. With this in mind, consider first the resistance of a ship in ice. From our experience of full-scale and model-scale testing of ships in ice, the ice resistance of a ship is proportional to the beam of the ship times ice thickness squared as the first approximation. Using this relationship, the ratio of the prototype ship resistance to the model ship resistance can be expressed as:

\[
\frac{R_{ship_P}}{R_{ship_m}} = \left(\frac{B_{ship_P}}{B_{ship_m}}\right) \cdot \left(\frac{h_{P}}{h_{m}}\right)^2 = \lambda \beta^2
\]
With this relationship of $\lambda B^2$ for the ratio of prototype and model forces as a basis, scaling laws can be developed for the remaining ice-ship-water variables. To illustrate this procedure, consider again the force required to crush a cylinder of diameter $d$ through an ice cover of thickness $h$ and crushing strength $\sigma$. The ratio for the prototype and model crushing forces can be expressed as:

$$\frac{P_{\text{crush}}^p}{P_{\text{crush}}^m} = \left(\frac{\sigma_\text{p}}{\sigma_\text{m}}\right) \cdot \left(\frac{d_\text{p}}{d_\text{m}}\right) \cdot \left(\frac{h_\text{p}}{h_\text{m}}\right) = \lambda B \cdot \left(\frac{\sigma_\text{p}}{\sigma_\text{m}}\right)$$

(11)

For this ratio to equal $\lambda B^2$, the ratio of the crushing strengths must scale as $B$. Similar procedures can be employed for the shearing forces, buckling forces, buoyancy forces, and water drag forces to derive scaling laws for all the ice mechanical properties as was done for the undistorted models.

**ICE MODELING MATERIAL**

For hydraulic models of ice-covered waters to accurately simulate the interaction between ice, water, and ships or marine structures, a modeling material must be used which obeys the scaling laws listed in Table 1. If such a material is used, the forces exerted by the ice on ships and marine structures and the resulting ice behavior (failure and movement) will be true scaled representations of real world conditions.

Materials, such as polyethylene, wood, and paraffin, have been used to simulate floating ice pieces. While the density and size of these materials can reproduce floating ice pieces in model-scale, these materials do not simulate the mechanical properties of ice such as flexural strength, crushing strength, and elastic modulus. As a result, these materials can only be used to study ice as discrete rigid bodies and cannot be used to study the internal failure or collapse of ice pieces or consolidated ice fields.

To our knowledge, only one material, MOD-ICE, closely models both the physical and mechanical properties for ambient temperature models. Saline ice could be used as a modeling material but its inability to permit testing at room temperature and its inability to hold its properties constant over long testing times are major disadvantages. In addition, we know of no method by which the Darcy-Weisbach friction factor for saline ice can be varied. MOD-ICE is a synthetic material, developed by Dr. Bernard Michel [1,2]*, which has been used successfully by him, ARCTEC CANADA Limited, and ARCTEC, Incorporated to study the interaction of ice with ships and marine structures. MOD-ICE is comprised of five (5) separate components whose individual percentage concentrations can be varied in order to obtain the following desired scaled properties of flexural strength, crushing strength, elastic modulus, density, and roughness (Darcy-Weisbach friction factor). In addition, the coefficient of sliding friction can be controlled by choosing the proper surface preparation or material for the model ship or marine structure. It is important to note that MOD-ICE, as with any modeling material, is not a perfect modeling material in that all the properties are not completely modeled correctly at any given time. MOD-ICE does, however, represent the current state-of-the-art in modeling consolidated ice covers. To illustrate this, a comparison of scaled MOD-ICE properties with fresh water ice properties and sea ice properties is shown in Table 2.

* Numbers in brackets indicate references listed at end of paper.
TABLE 2

COMPARISON OF SCALED MOD-ICE PROPERTIES WITH PROTOTYPE ICE PROPERTIES

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>FRESH WATER ICE</th>
<th>SEA ICE</th>
<th>MOD-ICE* (Scale=60)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ice Thickness (m)</td>
<td>0.0 - 0.6</td>
<td>0.0 - 3.5</td>
<td>0.1 - 6.0</td>
</tr>
<tr>
<td>Ice Flexural Strength (kPa)</td>
<td>500 - 1000</td>
<td>250 - 750</td>
<td>400 - 1000</td>
</tr>
<tr>
<td>Crushing Strength/Flexural Strength</td>
<td>2.0 - 5.0</td>
<td>2.0 - 5.0</td>
<td>0.7 - 7.0</td>
</tr>
<tr>
<td>Elastic Modulus/Flexural Strength</td>
<td>1000 - 5000</td>
<td>1000 - 8000</td>
<td>500 - 11000</td>
</tr>
<tr>
<td>Darcy-Weisbach Friction Factor</td>
<td>0.3 - 0.7</td>
<td>---</td>
<td>0.3 - 0.8</td>
</tr>
<tr>
<td>Coefficient of Sliding Friction</td>
<td>0.1 - 0.4</td>
<td>0.1 - 0.4</td>
<td>0.1 - 0.55</td>
</tr>
</tbody>
</table>

* Measured properties obtained during actual laboratory test program.

ST. LAWRENCE SEAWAY HYDRAULIC MODELS

Over the past several years, the Saint Lawrence Seaway Development Corporation (SLSDC) has been engaged in an effort to remove the constraints to winter navigation on the Seaway with the ultimate goal of year-round navigation [3]. One of the major impediments is the installation of ice booms each year in the Galop Island and Ogdensburg-Prescott areas. The purpose of the booms is to maintain a stable ice cover, thereby reducing the possibility of ice jamming which can destroy the hydraulic integrity of the river. To demonstrate that navigable ice booms can be installed which will allow commercial vessels to pass while preserving the stability of the ice cover, ARCTEC, Incorporated, under contract to the SLSDC, designed and instrumented a full-scale navigable ice boom in a navigation channel, known as the Copeland Cut, in Lake St. Lawrence near Long Sault Island. This project clearly demonstrated the feasibility of the navigable ice boom concept [4]. However, since the flow velocities through Copeland Cut are relatively low and the width of the boom opening was smaller than normal navigation channel widths, the applicability of the test results to other areas of the river could not be demonstrated from this test alone. It was, therefore, decided to use the Copeland Cut Test Ice Boom (CCTIB) primarily for collecting full-scale data which could then be used to prove the adequacy of physical hydraulic modeling. If such modeling techniques could be proven accurate, it was reasoned that hydraulic ice models could be used as an effective design tool for developing new navigable ice control structures for the Ogdensburg-Prescott and Galop Island areas.

In 1976, the SLSDC contracted with ARCTEC to design, construct, and instrument a working hydraulic model of the CCTIB which would demonstrate the applicability of hydraulic modeling to the problems of navigation season extension. The objective of the project was to determine the degree of correlation which can be achieved
between data obtained from a physical hydraulic model and that obtained from the full-scale tests of the CCTIB. A high degree of correlation between the model test results and the full-scale data was achieved in that: (1) boom forces compared favorably with measured full-scale boom forces, (2) shore cracks occurred with changes in water elevation as in full-scale, (3) shorefast ice remained shorefast in the model as it did in full-scale, and (4) ice bridging in the ship navigation channel after a ship passage occurred as it did in full-scale. A final report on the results of the model study is currently being prepared. Photographs illustrating this correlation are shown in Figures 1 and 2.

As a result of the success of this model, an 11 mile stretch of the St. Lawrence River from Stillwells Point, N. Y. to the middle of Galop Island will be modeled to develop a navigable ice control system for the Ogdensburg-Prescott and Galop Island areas. This model will be operational this winter.

ROLE OF HYDRAULIC MODELING IN SOLVING ICE-RELATED PROBLEMS

Hydraulic modeling has played an important role in solving ice-related problems [5 - 12]. To illustrate the role that hydraulic modeling can play, consider the ice problems in the St. Lawrence River. First, there is the problem of determining if a stable ice cover will form behind an ice retaining structure. For the most part, ice forms by juxtaposition of floes which break off from near the shore and flow down river until they form a natural arch across the river or encounter a man-made barrier such as an ice boom. If the current velocity is not too fast (subcritical), the floes will build upstream until the river reach is covered with ice. Once this occurs, the ice consolidates and grows in place forming a solid, somewhat uniform or monolithic ice cover. In reaches where the current is somewhat faster (transitional), the same thing may occur; however, the ice cover is very unstable and the least disturbance, such as a strong wind, can cause the ice to move periodically downstream in what is referred to as a "shove." These shoves cause the ice to thicken, sometimes to great thicknesses, and form what is termed an "ice jam." In reaches where the current is even faster (supercritical), floes moving downstream may "topple" under a natural or man-made barrier and then be carried further downstream where they eventually come to rest and form what is termed a "hanging dam."

The current velocities at which these phenomena occur can be computed from existing theories; however, the theories are far from being exact and, at best, give only a range in which one phenomenon or another is most likely to occur. The difficulty in mathematically predicting the transitional velocities lies in the fact that the formation of equations to describe ice jams and hanging dams is complex with numerous variables entering into the equations.

If predicting whether a stable ice cover will exist in a particular reach appears difficult, predicting the forces that a given type of ice cover can exert against a structure and the physical characteristics of the ice cover is even more difficult. Prediction of the forces that ice can exert against a structure is obviously important from the point of view of the designer of the structure, and prediction of the physical characteristics of the ice field is equally important from the point of view of the marine engineer who must determine if a ship of given icebreaking capabilities can transit the ice field. Once again, some mathematical theories exist, but the state-of-the-art is not at a very high level and many of these theories have not been proven in the field or in the laboratory. From this discussion, it is readily apparent why many researchers active in the hydraulic/
FIGURE 1a  PROTOTYPE BOOM BEFORE ICE FORMATION

FIGURE 1b  MODEL BOOM BEFORE ICE FORMATION
FIGURE 2a  PROTOTYPE BOOM WITH AN ICE COVER AFTER SHIP PASSAGE

FIGURE 2b  MODEL BOOM WITH AN ICE COVER AFTER SHIP PASSAGE
ice/marine engineering fields have turned to physical model experiments to obtain answers rather than relying solely on mathematical theories. As a result, one can expect the role of hydraulic modeling in solving ice related problems to increase in the future.

REFERENCES

ABSTRACT

Based on the projected developments in the Arctic regions and the corresponding lack of adequate oceanographic data in this area, the need for a research ship capable of operating in ice-covered waters has recently been established. The performance requirements for a research ship which would fulfill these needs includes the capability for breaking up to 0.46 m of level ice at a speed of 3 knots. As ice conditions in the polar regions vary as a function of time, it was decided to determine the operational limits of the ship for different geographic areas as a function of calendar month. This paper presents the results of that operational study for Arctic East, Arctic West, and Antarctica.

INTRODUCTION

A capable Arctic Research Ship has been a matter of long standing need in the United States. For the past half-century, and even earlier, the National Academy of Sciences and the National Research Council have posed the problem of polar research and the particular difficulties involved with developing adequate facilities to cope with these problems. Recently, the need to develop Arctic resources has lent increased emphasis to the requirement of suitable ships to work in the Arctic as well as the sub-Arctic waters. More recently, the recommendations of a workshop in Bering Sea Research, held under the auspices of the National Science Foundation, cited the need for an iceworthy research ship capable of operating in high-latitude seas (Probes, 1973).

Operational requirements for the ship included the capability for withstanding occasional besetment in ice, the capability to operate at a three-knot speed in 0.46 m level ice and 0.91 m broken ice, and the ability to withstand severe superstructure icing. Weather design criteria included: air temperature of -40°C at 50 knots wind, sea temperature of -2°C, and sea state 5 ("very rough" 2.4-3.7 m waves).
PRINCIPLE CHARACTERISTICS

A design was performed [Elsner, 1977] which resulted in the following characteristics for the Arctic and Antarctic Research Ships shown in Table 1. The Antarctic ship is similar to the Arctic ship but has an additional 10.67 m of midbody to accommodate the need for greater endurance and range required in southern deployments.

TABLE 1. PRINCIPAL CHARACTERISTICS OF ARCTIC AND ANTARCTIC SHIPS

<table>
<thead>
<tr>
<th></th>
<th>Arctic Ship</th>
<th>Antarctic Ship</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length, Overall</td>
<td>58.17 m</td>
<td>68.83 m</td>
</tr>
<tr>
<td>Length, Waterline</td>
<td>53.34 m</td>
<td>64.01 m</td>
</tr>
<tr>
<td>Beam</td>
<td>12.19 m</td>
<td>12.19 m</td>
</tr>
<tr>
<td>Depth</td>
<td>6.40 m</td>
<td>6.40 m</td>
</tr>
<tr>
<td>Draft (normal)</td>
<td>4.57 m</td>
<td>4.57 m</td>
</tr>
<tr>
<td>Draft (maximum)</td>
<td>5.00 m</td>
<td>5.00 m</td>
</tr>
<tr>
<td>Block Coefficient</td>
<td>0.53</td>
<td>0.61</td>
</tr>
<tr>
<td>Displacement</td>
<td>1814 m ton</td>
<td>2409 m ton</td>
</tr>
<tr>
<td>Shaft Horsepower</td>
<td>2.76 MW</td>
<td>2.76 MW</td>
</tr>
<tr>
<td>Number of Propellers</td>
<td>2, Controllable Pitch</td>
<td>2, Controllable Pitch</td>
</tr>
<tr>
<td>Number of Rudders</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Bow Thruster</td>
<td>1 Water Jet Type</td>
<td>1 Water Jet Type</td>
</tr>
<tr>
<td>Type of Propulsion</td>
<td>2 Diesel Engines with Reduction Gears</td>
<td>2 Diesel Engines with Reduction Gears</td>
</tr>
<tr>
<td>Range</td>
<td>21,319 km @ 13 knots</td>
<td>32,180 km @ 13 Knots</td>
</tr>
<tr>
<td>Endurance</td>
<td>60 Days</td>
<td>60 Days</td>
</tr>
<tr>
<td>Accommodations</td>
<td>17 Crew (12 required plus 5)</td>
<td>17 Crew (12 required plus 5)</td>
</tr>
<tr>
<td></td>
<td>17 Scientists</td>
<td>19 Scientists</td>
</tr>
<tr>
<td>ABS Classification</td>
<td>1AA</td>
<td>1AA</td>
</tr>
<tr>
<td>Laboratory Area</td>
<td>193.79 sq m</td>
<td>327.10 sq m</td>
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</table>

HULL DESIGN FEATURES

The bow shape includes a 30° stem angle for good icebreaking ability. This angle is extended 0.61 m above the design waterline for efficient operation through rafted or hummocky ice fields, and is carried 3.05 m below the design waterline for operation through small pressure ridges. At this depth, the stem angle changes to a near vertical orientation. The bow form is similar to the Maierform shape used on fishing vessels and other seagoing ships to improve seakindliness; therefore, the final bow form is a compromise between ice transiting and seakindliness.

The stern of the ship was designed to be structurally strong to withstand ice encounter while backing. In addition, the maximum beam is sufficiently forward of the propellers so that the stern shape at the waterline gives a continuously decreasing beam which permits submerged ice pieces to rise to the water surface, thereby reducing propeller-ice impacts. In addition, allowance has been made for large propeller tip-hull clearances.
No restrictions were placed on midbody shape for a ship having the icebreaking ability specified and the shape at the waterline could be sloped or vertical. Although the sloped surface would give lower levels of ice force, a properly designed wall-sided vessel can withstand expected ice forces and can become beset in ice without damage.

SHIP PERFORMANCE

Icebreaking capability of the Polar Research Ship and the open water speeds are shown in Figure 1. Ship performance in small broken ice pieces with 100% ice coverage is at least twice that of level ice thickness. In addition, the ship can break through a pressure ridge of 2.13 m thickness with impact speed of 6 knots in the ramming mode.

![Figure 1. Icebreaking and Open Water Performance of Polar Research Ship](image-url)
OPERATIONAL CAPABILITY

In the conceptual design of the Polar Research Ship, an ice transiting capability of over 0.91 m in broken ice fields and an icebreaking ability of 0.46 m in level ice were provided. As a result, the ship can not operate year-round in certain ice-covered geographic areas. In an attempt to define the capability of the ship as a function of geographic area and time, a set of figures and tables was developed.

Arctic East operating zones are shown in Table 2 and Figure 2. They show that operations along the Greenland West Coast will, in general, provide easy access to the northern latitudes. Operations along the Eastern Coast of Baffin Island are not recommended for at least seven months of the year. The area between Canada and Greenland will abound with icebergs and iceberg fragments in addition to the usual assortment of pressure ridges and level ice.

<table>
<thead>
<tr>
<th>TABLE 2. OPERATING ZONES BY CALENDAR MONTH FOR ARCTIC EAST</th>
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<tbody>
<tr>
<td>CALENDAR MONTH</td>
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<tr>
<td>Apr</td>
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<th>OPERATING ZONES</th>
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KEY:
- Ice Conditions Exceed Ship Capability
- Ice Conditions Exceed Ship Capability for Part of Zone During Ice Break-up
- Ship Can Safely Operate

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FIGURE 2. ARCTIC EAST OPERATING ZONES FOR A POLAR RESEARCH SHIP AS DEFINED IN TABLE 2.
Arctic West operating zones are shown in Table 3 and Figure 3. The Polar Research Ship will be able to operate successfully year-round south of St. Matthew Island and Nunivak Island, and three months of the year in the vicinity of Wrangel Island and Point Barrow. The operating zones and months depicted are representative and variations can be expected. There are also some typical characteristics associated with the movement of the ice edge that should be noted. More specifically, the ice edge tends to remain at the extreme northern and southern latitudes for several months at a time. Conversely, during ice growth and decay, the ice edge moves fairly rapidly in north-south and south-north directions. The Arctic West areas will include level ice, pressure ridges, and hummock ice fields, but will not include icebergs.

### Table 3. Operating Zones by Calendar Month for Arctic West

<table>
<thead>
<tr>
<th>Calendar Month</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
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</tbody>
</table>

**KEY:**
- ✗ Ice Conditions Exceed Ship Capability
- ✔ Ship Can Operate Safely
FIGURE 3. ARCTIC WEST OPERATING ZONES FOR A POLAR RESEARCH SHIP AS DEFINED IN TABLE 3.
The Antarctic operating zones are shown in Table 4 and Figure 4. Ice conditions in the Antarctic will differ from the Arctic West as only a few pressure ridges will be found and icebergs can be located in select areas.

### Table 4. Operating Zones by Calendar Month for Antarctica

<table>
<thead>
<tr>
<th></th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Jan</th>
<th>Feb</th>
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**KEY:**
- Ice Conditions Exceed Ship Capability
- Ship Can Safely Operate
FIGURE 4. ANTARCTIC OPERATING ZONES FOR A POLAR RESEARCH SHIP AS DEFINED IN TABLE 4.
The need for the Polar Research Ship to operate in both Eastern and Western Arctic regions gave rise to an analysis of the ship capability to transit above the North American continent. Figure 5 shows a typical east-west passage by the ship with an east coast departure on August 1 and a west coast (Seward, Alaska) arrival on November 1. The ship can successfully navigate the waters shown as sufficient water depth exists and ice conditions are sufficiently mild to permit unescorted ship transit. It should be noted that the region just west of the point on the route marked September 10 may prove difficult during severe ice years, however, during typical ice years, transit will not be difficult. Finally, it should be noted that a one week port call at Thule, Greenland is shown as it is approximately midway in distance between east coast and west coast ports.

FIGURE 5. 90 DAY VOYAGE ABOVE THE NORTH AMERICAN CONTINENT (EAST-WEST VOYAGE) - 11,600 KM
ACKNOWLEDGEMENT

The work described in this paper was sponsored by the National Science Foundation Grant No. OCE76-10089, "The Conceptual Design of a Polar Research Ship." The study was administered by the Institute of Marine Science of the University of Alaska on behalf of the University National Oceanographic Laboratory System. A special note of appreciation and thanks is extended to co-workers on the project, Dr. Bob Elsner and Mr. John Dermody of the University of Alaska; Mr. Jonathon Leiby, of Woods Hole Oceanographic Institution; and Mr. John Gilbert, of John Gilbert Associates, Inc. The help and assistance of Mss. Debbie Burrows, Shirley Brown, Linda Trent and Diane O'Grady in the preparation of this paper are gratefully acknowledged.

REFERENCES


ESTIMATION OF ICE FORCES ON THE HULL OF M.V. ARCTIC EXPLORER BY STRAIN GAUGE MEASUREMENTS

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ABSTRACT

The ice forces on the hull of M.V. Arctic Explorer have been estimated by measuring strains in the frames of the ship structure when the ship was chartered by NORDCO to conduct a range of scientific experiments. The recorded strain gauge data is interpreted with the help of theoretical calibration using finite element analysis of the hull structure. The estimated ice force is tabulated for the most severe cases of loading during the project.

1. Introduction and Review of Literature

Over the years, many semi-empirical and analytical expressions have been developed to describe ship resistance to continuous motion in solid field ice. The complexity of ship-ice-water interactions makes the development of purely analytical solutions to the problem more difficult. Lewis and Edwards (1970) developed methods for determining the resistance that uniformly thick ice cover offers to a moving ship. Their experimental solutions were based on experimental data collected in the field on full-size icebreakers and in the laboratory on model-scale icebreakers. Johansson (1967) calculated the ultimate pressure which various portions of a ship's hull can withstand and plotted these against the square root of the product of the icebreaker's shaft horsepower and displacement. Popov, Faddeyer, Kheysin and Yakovlev computed the hull impact loads using expected values of ice properties, ship size and shape, and ship speed. The equations for the impact between a ship and ice floes of various sizes and shapes were solved by energy methods. Milano (1975) established an energy balance and computed the ship resistance to continuous motion in ice fields based on a characteristic distance which gave rise to that energy level. Milano (1975) discussed the ship resistance in solid ice. White (1965) showed that the vertical force developed at the bow of the icebreaker as a result of climbing onto the ice was most affected by variations in bow angle, coefficient of friction, displacement and impact velocity. Edwards and Lewis (1970) reviewed the model test techniques employed by the U.S. Coast Guard at the Naval Undersea Research and Development Centre. Correlation between model and full scale was demonstrated by comparing the results of full scale and model trials of the Wind Class icebreaker.

2. Objectives of the Project

In order to obtain optimal design of the structure of an icebreaker or a commercial
ship, it is essential to know the expected loads to which the ship may be subjected. With this objective in mind, a project was undertaken to measure the strains in the frames of the M.V. Arctic Explorer, which was chartered by NORDCO to conduct a range of scientific experiments in the pack ice off the Labrador coast. In this paper, a description is given of the strain gauge installation, summary of data collection and interpretation of data.

3. The Strain Gauge Installation

The choice of the ship was based on its suitability for the overall project and there were characteristics of the vessel which were not ideal from the standpoint of a strain gauge installation. Bearing in mind that the watertight integrity of the vessel had to be maintained at all times and since all areas in the region of the waterline was of double skin construction, it was obvious that the gauges would have to be installed in a watertight compartment which was not easily accessible. The location chosen for strain gauge installation was on the starboard side, shown in Figure 1.

The bonded resistance type strain gauges were attached to the hull by an adhesive agent. The surface preparation was carried out according to the manufacturer's specifications. One significant problem with installing the strain gauges in a sub-zero temperature environment lies in keeping the gauge installation area warm enough to allow the bonding agents to cure to maximum efficiency. A combination of electric fan heaters and infrared lights were found to be adequate in heating the surface areas and maintaining the air temperature in the compartment at above 20°C. The original plan called for the installation of gauges onto the shell as well as on the flanges of the built frames. This idea had to be abandoned when it was found that moisture kept forming on the gauge area even when continuous heat was applied from infrared lamps and electric heaters.

The gauges were left to cure overnight after which lead wires were soldered on and the gauge installation was covered with a waterproof sealant. A covering of neoprene rubber was added to afford mechanical protection. The installation was then covered with aluminum foil tape, the seams of which were sealed with a rubberized liquid compound. The electrical leads were run out of the watertight compartment by employing two rubber gaskets of the type normally used to seal manholes with a layer of silicone sealant between. The strain gauges were calibrated by shunt resistance before the experiments.

Of the twenty gauges installed before leaving port, seventeen functioned well throughout the trip. The other gauges were found to be defective and they were replaced except for one gauge which was not accessible. Upon arrival back in port, an inspection of the forward tanks revealed that the protective coating suffered no damage during the voyage.

4. Data Collection

Only ten strain gauges were monitored and their signals were simultaneously recorded on a UV-recorder. Recordings were done on a random selection basis depending mainly on ice condition. The duration of recording ranged from a few minutes to one hour. A total of twenty-eight observations were carried out during the project. Each observation consisted of three aspects: signal recording from gauges, visual observations from bow, and photography on 16 mm movie film. A summary sheet was prepared during each observation. Correlation of data from these sources has resulted in the information presented in the final section of this report.
5. **Interpretation of Strain Gauge Data**

The experimental calibration of strain readings to applied external loads could not be carried out due to logistic problems. Hence the ship's hull structure was analyzed using finite element analysis. The schematic arrangement of a portion of the hull with the plating and transverse frames is shown in Figure 2. It consists of steel plates and frames of three different sizes varying in depths from 180 mm to 435 mm supported between two decks. The location of the eleven strain gauges installed are shown in Figure 2. The analysis was carried out idealizing the structure as an assemblage of (i) uniform beam elements and three dimensional elements and (ii) only three dimensional elements. SAP-IV structural analysis computer program (Bathe, Wilson and Peterson, 1973) was used in the analysis.

From the results of analysis in the first idealization, it was observed that the displacement is large only in that panel which is loaded and it decreases to very small magnitude in the adjacent panels due to the rigidity provided by the other frames. This has also been observed in the strain gauge recorded data, shown in Figure 3. It can be seen from Figure 3 that while a particular load causes high strain on a frame, the strains in the other frames are negligible. The shift in the records of individual strain gauges is due to the movement of load with respect to the ship and this shift has been approximately correlated to the speed of the ship.

In the second idealization, a typical panel bounded between the frames of type A and B shown in Figure 2 was discretized and analyzed in detail. The finite element discretization of the panel using three dimensional elements is shown in Figure 4, and a static analysis is carried out for different load cases. It has 720 degrees of freedom with 338 nodal points; computation time for a typical analysis was 108.12 seconds on an IBM 370/158 computer. The results of the analysis are presented in Figure 5, which shows the predicted strains in the flange of the frame with respect to the vertical distance along the frame for different load cases.

It is evident from Figure 5 that the strain response is narrow with respect to vertical distance. Hence, only those strain data can be interpreted, which indicate very high strain at any one of the two gauges on a single frame with negligible reading on the others. The maximum response due to ice impact based on calibration is 0.9961 microstrain per kN force on the frame. The corresponding value is 1.5054 based on the full scale experiments of the Great Lakes icebreakers reported by Edwards, et al (1970). Using the individual load cases and the strain calibration obtained by the finite element analysis, the interpretation of the strain data is given in Table 1, which shows some of the high strains recorded during the entire project.

6. **Conclusion**

The strains in the frames of the M.V. Arctic Explorer are measured and the strain data is interpreted with the help of the finite element analysis. Some of the most severe cases of loading are tabulated. The interpretation is only possible with the help of the SAP-IV structural analysis computer program. From the results of analysis (Figure 5), it is evident that more strain gauges should be installed on a frame so that better prediction of the ice force can be made.

**ACKNOWLEDGEMENT**

The authors gratefully appreciate the computing facilities provided by the Memorial University of Newfoundland.
REFERENCES


M/V ARCTIC EXPLORER

main particulars

- l.o.a. 49.6m
- lbp. 42.4m
- breadth 11.5m
- depth 4.8m
- draft 4.1m

Figure 1. Sketch of the ship showing location of strain gauge installation
Frames
Type A - 435 x 50 mm
Type B - 220 x 50 mm
Type C - 180 x 50 mm

Figure 2. Sketch showing frames and location of gauges 1-11
Figure 3. Sample of recorded strain data (all showing tensile strain)
Figure 4. Discretisation of a panel showing plate and frame.
Figure 5. Plot of strain in the flange of a frame due to an applied external force of 1 kN at 4 different positions.
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TRANSPORTATION OF PERSONNEL, INSTRUMENTS AND EQUIPMENT ON FIRST - YEAR SEA ICE FOR OCEANOGRAPHIC SURVEY AND RESEARCH PURPOSES

Hermann A.R. Steltner
Arctic Research Establishment
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INTRODUCTION

This report represents the experience of more than 21,000 vehicle kilometers driven on first-year sea ice during arctic winter conditions without accident, injury, mishap or loss. This experience was accumulated during the years 1972 (March) to 1977 (July) on the Eclipse Sound and adjacent waterways. Basis of operations was and still is in the Hamlet of Pond Inlet at the Eclipse Sound, latitude 72°40'15" N, longitude 077°50'00" W. The waters in this region are ice-covered from 8 to 10 months per year; however, there have been years when the Eclipse Sound ice stayed into a second year (last time in 1972). The Eclipse Sound is connected to the Baffin Sea via Pond Inlet and to the Lancaster Sound by the Navy Board Inlet; and the waters around Bylot Island indicate extensive mixing, thus supporting abundant sea life which, by historical evidence, has for millennia supported local people and still is today the backbone of this "area community" centering in the Hamlet of Pond Inlet with a population of now about 600.

The apparent marine ecological importance of the waters in this area extends along Baffin Island to the Labrador current, and physical and biological knowledge from the Bylot marine region contributes significantly to the understanding of the Baffin Island eastern seaboard ecology, possibly extending as far as Labrador and Newfoundland.

In order to satisfy the need to know, shipborne and ship-based oceanographic surveys, with their operational limitations by sea ice, needed to be extended to surveys performed from the ice plate, in this effort benefitting from the advantage of a stable platform.

TRANSPORTATION ON SEA ICE

1. First Period of Experience

1.1. Details -- The first period of experience was from March to July 1972, and the first oceanographic survey here in our series was made as part of the EOS project. Oceanographic measurements were taken along a traverse from Pond Inlet to Sermilik glacier on Bylot Island, a station every 1 km, and to a depth of 250 m on the Eclipse Sound. Daily excursions were made from Pond Inlet and, as it was relatively warm then, we rigged a 2 x 2 m trailer with balloon tires, 20 x 11.00-8, and mounted an A-frame over the 12 V elec-
trically driven C-5 winch. A small generator and battery charger were placed on the trailer as well, balancing the weight. The trailer was connected with a ball hitch to an air-cooled diesel-driven Argo-8 Amphibious All-Terrain Vehicle (AATV). The travelling speed averaged 8 km/h and the limitation was the driving comfort as well as the bouncing of the trailer at higher speeds over rafted ice. Although during that period (1972) the snow cover was hard-packed, an occasional loose drift became difficult to traverse as the wheels lost traction when the vehicle bottom compacted the snow.

--The conversion of the Argo-8 AATV to diesel power (weight 410 kg) was a modification and not a standard. The diesel engine, a single-cylinder 10 HP at 2000 RPM, made by HATZ and sold in Canada, was selected because of the vertical cylinder-cooling arrangement, and this is an engine type that has not given us any problems during the entire period until today.

--The author designed for the scientific tasks a 1.5 m x 3.6 m modular sled of welded aluminum construction. The runners, clad by 3 mm mild steel sheeting and flanged, with a runner width of 10 cm and 30 cm high (freeboard), are bolted at the flanges to the aluminum sled frame. These sleds (4) weigh 135 kg each, and it takes 24 bolts to assemble one sled. These sleds were used for the under-ice video recording, for the seismic instrument housing and for general transport on the ice. Onto these sleds assembled in Pond Inlet, shelters for various other purposes were mounted. The shelters consisted of a framework of perforated 50 mm light-weight angle iron, nut-and-bolt assembled, covered with 8 mm plywood and painted black outside, with a tar/felt cover over the roof part.

--As towing vehicles we had four diesel-driven Argo-8's described above, one 2 cycle gas-engine driven Argo-8, and two Otaco diesel-driven 6-wheelers, all of which are AATV's. We also engaged locally a varying number of Bombardier snowmobiles. The bulk of the transportation, 5888 km out of 8000 km during the period May 10 to July 31, 1972, was carried out with the Argo-8's.

1.2. Experience -- The experience learned from this period was as follows:
The aluminum sleds and their usefulness met all requirements.
Driving wheeled vehicles is limited by the degree of compactness of the snow cover on the ice.
On the diesel modification, although technically practical and break-down free, the positioning of the comparatively heavy engine in the front made steering difficult (differential brakes).
Skill in operating must be accumulated before a maximum utility can be expected out of these AATV's.
On wet surfaces with large or deep puddles and/or cracks or leads, amphibious vehicles, such as we had and still have, showed to be the only vehicles with which one can safely operate, as well as pull loads over any distance under wet-ice conditions.
For fast transport of persons and loads up to 700 kg, snowmobiles with a 440 cc engine, pulling locally built sleds (kamotiks) offer the greatest utility on dry sea-ice surfaces, but without comfort and, under certain conditions, a serious hazard to health on two counts, not considering the noise-effect: maximum stress of the back (spinal column) and monoxide poisoning from the engine exhaust.
2. Second Period of Experience

2.1. Details -- The second period of experience was from December 1974 to May 1975, with regular scheduled measurements of sea ice, snow on sea ice, and sea-ice temperatures at a number of stations along the 1972 traverse. This time-series data-collecting effort differed in many ways from the research and investigative efforts of 1972, and regularity of repetitive attendance to measurements on the sea ice was the task over several months.

-- During the cold and dry period, snowmobiles were used pulling kamotiks, 3.6 m long, with loads up to 250 kg. Sled runners were shod with 6.4 cm wide and 6 mm thick steel bar. The kamotik weight varied with the type of wood used, between 100 and 150 kg. The freeboard between the steel-clad runners and the wooden cross bars was 23 cm. The average speed during this cold period, traversing areas of rafted ice, was 18 km/h.

-- Navigation during the dark period and periods of drifting snow was aided by suspending amber strobe lights at a height of 2.5 m on top of the tetraheders spaced over the traverse, a strobe light every 5 km. The flash cycle was set at 1.5 seconds, which appeared compatible with temperatures in the $-40^\circ C$ range, and the alkaline battery power supply was suspended in a watertight bag underneath the ice plate.

-- During the following wet period, the Argo-8, but with a 440 cc two-cycle engine drive, was used until the end of the measuring period (Argo-8 gas version weighed 350 kg). The average speed was 12 km/h.

-- As certain instruments need a prescribed ambient temperature to function properly within their calibrated ranges, these were transported in well insulated boxes and resting on 85 W battery-heater blankets. Alongside the instrument we inserted a dial thermometer for reading the ambient temperature in the box. On a $-40^\circ C$ day, starting out with an ambient temperature of 22°C, as an example, we still read 19°C ambient after 75 minutes. To replenish the lost heat, we carried a Kohler 3000 W, 120 V, 4 cycle generator in a wooden box on the kamotik, in order to heat up the battery blanket whilst readings were in progress, thus maintaining the required temperature. The generators, properly tuned, never gave us any starting problems during the last three years.

2.2. Experience -- The experience learned in this period was as follows:

Repair and maintenance efforts for equal milage, terrain and ambient conditions were greater for snowmobiles compared to the AATV's. The lack of protection, particularly from the wind, made measuring "out-of-the-box" from the open kamotik uncomfortable.

On a tight schedule, preventive maintenance is difficult and lacking spare parts supply for equipment older than the current year adds to the time-loss problem.

With proper planning, it is possible to keep to daily and weekly schedules during the winter season under arctic conditions.

Winds in excess of 25 knots effectively prevent travel if they are accompanied by an overcast sky condition. Winds in excess of 36 knots effectively prevent travel under any conditions.

Navigation markers, such as flags or balloons, at an elevation of 4.5 m or more above the surface were visible from a distance up to
2 km at 32 knots wind, and up to 1 km at 36 knots of wind. This was over a relatively smooth ice surface. For frequently travelled directions navigation markers (tetrahedrons) spaced at 1 km intervals provide optimum aid.

3. Third Period of Experience

3.1. Details — The third period of experience started in December 1975 and ended in the middle of June 1976. For this period, two of the aluminum sleds were converted; one to accommodate a diesel-hydraulic oceanographic winch, and the other was insulated as a personnel comfort accommodation. Both sleds were towed out about 17.5 km on Eclipse Sound and set up as oceanographic station. At the front of the oceanographic sled, a "porch" made of 2 x 4's and 7 mm plywood was placed over the hole in the ice through which oceanographic casts were made weekly.

— Heating was provided from a 30,000 BTU oil-fired portable heater, thermostat-controlled. The warm-up time varied from 30 minutes to one hour, depending on the ambient temperature and the time of the year. From April on, we did not need a warm-up period, as the black coating of the oceanographic sled in the sunlight converted enough energy to warm up the place. In addition, the air-cooled diesel also provided heat.

— The weight of the oceanographic sled, fully equipped, was about 480 kg, and was towed to location with a 650 cc dual-track snowmobile in 1.5 hrs. during January 1976 (18.0 km total distance).

— The personnel comfort sled was towed with a 440 cc single-tracked snowmobile in the same time (weight of P.C.S. 250 kg).

— For ice coring and snow sampling the generator, electric motor for the corer, the coring equipment, and the core and snow sample box were transported on a kamotik towed by a 440 cc single-tracked snowmobile.

— Ice temperature measurements, weekly at each of the four stations, were made using the Argo-8 gas version to carry the galvanometer box and to make the readings and to record inside the nylon-covered cabin top of the vehicle. The Argo was always accompanied by a snowmobile towing a kamotik on which the generator and vehicle fuel was carried. On arrival at the measuring station, the air-cooled engine heating the inside of the Argo-8 during the driving was shut off and a 1500 W electric heating fan kept the inside of the Argo comfortable. By placing an additional canvas over the nylon cover of the vehicle we prevented the interior from frosting at temperatures below -25°C.

— For chart changes at the climatic station on the ice, we used a snowmobile.

— The navigation markers were towed with a 440 cc snowmobile on one of the aluminum sleds, fully assembled and nested.

— Reconnaissance for cracks and ridges was carried out with a 440cc snowmobile towing a kamotik carrying fuel and the food box with Coleman stove and utensils.

— Communication with parties in the field was by schedule, and the equipment is Spilsbury and Tindall, SBX-11, operating on the PCSP frequency of 4,982 MHZ. Both, dipole as well as whip antenna were tried and the dipole antenna is now preferred.
3.2. Experience -- The experience learned in this period is as follows:

The winter period, coincidental with a relatively high salinity of the snow cover, requires more ball-bearing replacements on the snowmobiles than the following, usually much colder, period in the early spring when the snow cover has a lower salinity. The geometry of the surface travelled, that is, the gradients of sastrugi's and drifts, the extent and configuration of rafted refrozen ice fields combined with either absence or degree of presence of snow cover, and characteristics of the snow cover are directly related to the degree of vehicle maintenance required. Absence of snow is generally rough on the undercarriage of any snowmobile, and when such equipment-breaking conditions exist, no degree of preventive maintenance can reduce break-downs, and for such conditions we have learned to carry along undercarriage sub-assemblies for the snowmobiles.

For the Argo-8 wheel shear pins and main drive chain replacements are more frequently needed whilst travelling through rafted refrozen ice fields.

Enlarging of windshields and attaching wind deflectors at the sides of the snowmobiles added measurably to the driving comfort and actually reduced fatigue. The steeper the windshields are mounted, the lesser the icing up; however, vapour conditions may exist at which, no matter what one does, the windshields will ice up.

Back supports on the snowmobiles definitely reduce fatigue. Particular attention must be paid to the relation between dew point temperature, wind direction, wind speed on one side, and direction and speed of the snowmobile on the other side, to prevent CO poisoning or a lesser degree of intoxication. In an unfavourable combination of the aforementioned conditions, direction and travel-speed must be changed to suit. This may entail travelling a longer distance by frequently changing course, which adds then to the difficulties of navigation and consumes more fuel. Unfortunately, all snowmobiles have an exhaust system which the author considers a potential health hazard under arctic conditions.

On the Argo-8 AATV, we modified the exhaust by extending the exhaust pipe to about 2.0 m above ground level. Although prior to this modification the hazard of exhaust-gas inhalation was the same as it still is with the snowmobiles, we experienced during the last two years, this is since we modified the Argo exhaust, a drastic improvement in the fatigue-level of the various drivers during and after trips. No case of drowsiness, headaches, vomiting or painful lungs occurred after the exhaust was modified.

The frequent design changes in snowmobiles with inherent year-to-year differences in common hardware used, even within one model-series, complicate unduly maintenance and servicing. Even considering buying new snowmobiles every year would not overcome this operational problem as the mechanical differences appeared to be considerable within a model series throughout the last few years' time.

4. Fourth Period of Experience

The fourth period of experience started at the end of October 1976 and had ended with the completion of this report on July 10, 1977.
4.1. Details -- During this period we undertook to test the real mobility and utility of the oceanographic station. For this a third aluminum sled was built up in the manner described for the first oceanographic sled (during the third period). The roof was sectionally extended to accommodate the head sheave for the oceanographic wire, replacing the "porch". Through the floor was fitted an opening for oceanographic casts and large enough for a diver with a set of twin-bottles on his back to descend down a ladder for under-ice sampling. The mobile oceanographic station now comprises three sleds with the sled parts being identical. On each end of the sleds, front and back, fittings were attached to rig towing bridles to provide for towing all three sleds at once with one more powerful vehicle.

--A J-5 Bombardier tracked vehicle was modified by removing the standard superstructure and replacing it with a structure designed by the author and suitable for cold-weather operation. Particular attention was paid to the exhaust system of this vehicle. This vehicle is powered by a Chrysler 241, six-cylinder, industrial gas engine with no automatic adjustment on either distributor or carburetor. Trailing skirts were made from canvas and weighted at the trailing edge. Artificial rubber sheets of a cold-resisting composition were placed alongside the tracks, covering the rear half of the tracks and reaching down to the center of the idler wheels.

--The whole of the train was tested and set up in March 1977. The actual setup of the oceanographic station was completed in four hours in the following manner:

After site determination by hodometer measurement and sextant bearing, a T-shaped area on the ice was cleared of snow. The oceanographic sled #1 was then towed into the vertical part of the T. Then the hole was cut through the ice and oceanographic sled #2 was towed over the hole, forming the horizontal part of the T. As the towing end of the #1 sled has an open front and #2 sled is provided with a matching opening in its left side, the oceanographic working area is totally enclosed. The access to it is through a door in the rear of #2 sled. Electric lighting is provided with a closed, grounded connection to the generator, the common ground being the sea water.

--The mobility of the oceanographic station was one point (1) of the field programme; the other points were: (2) physical and biological oceanography, taking samples of sea-water plankton and zoobenthos, as well as sea-ice samples and, for biology, diving to extract samples undisturbed from underneath the ice plate; (3) weekly ice and snow cores; (4) daily visits to the ice-climatic station; (5) periodic ice column temperature measurements; (6) ground-truth measurements for remote sensing overflight; (7) reconnaissance for cracks and ridges and other phenomena; (8) snow and sea-ice salinity experiments.

Significant for this period was the fact that we had to be continuously mobile for a period of over 90 consecutive days, which — as it turned out — was a tall order in view of the ambient conditions as well as the lamentable absence of conscious and responsible air carriers for any emergency supplies; even, if mailed First Class, taking up to six weeks from Hamilton, Toronto, Montreal to reach Pond Inlet; air cargo taking just as long. Nevertheless, mobility was maintained throughout the entire period, and not a day was mis-
sed on account of vehicle problems, and only one day was missed by reason of snowfall.

--Abnormal for this region was the snowfall on April 10th, depositing an average of 48 cm of snow on the sea ice on Eclipse Sound in a 12-hour period. As a consequence, this snowfall - combined with the high ice temperature compared to the other years - prevented the occupation of one oceanographic station in Tay Sound. On the day following this snowfall, a double- and a single-tracked snowmobile were required to attend to the climatic station about 8 km out on the ice. Before the snowfall, the average travel time to reach this station was 35 minutes; on April 11th, it took the two snowmobiles three hours to reach this station, thrusting forward alternately to make a path. During the month following this snowfall, gasoline consumption went up by 20% for same distances travelled as before the snowfall, and consequently, travel time increased. But, worst of all, additional wear and tear on the vehicles was evidenced by a fourfold increase in parts replacement compared to all previously experienced relation of distance travelled to replacement parts needed.

4.2. Experience -- The experience learned during this period was as follows: Extremely heavy snowfall on sea ice as experienced this spring takes a long time to compact and, in fact, this year never did reach the degree of compactness of wind-placed snow. This may have been due to the fact that there was no wind during this heavy snowfall and only low winds during the 8 days thereafter, which resulted in the snow surface crusting sufficiently to resist transport by following medium speed winds (up to gust level). Wheeled vehicles became useless on sea ice under the above described snow conditions.

The towing ability of tracked vehicles was effectively reduced, not due to resistance of the snow by its height, but by the lack of compaction and lack of frictional resistance. Where the tracks reached the sea-ice surface and the ice-surface temperature was warmer than -12°C, towing anything became almost impossible and even steel bars across the tracks did not provide much of an improvement. The dual-tracked snowmobiles performed best during this time.

In future, sleds built for the transport of equipment, supplies and instruments should not be less than 5 m long at the runners. We had borrowed a 5.5 m long kamotik and compared towing it with our kamotiks which have a length of about 4 m. We loaded both sleds equally with 50 kg per meter for the comparison, and the longer sled was an easier tow in spite of the additional 75 kg load.

Prior to the heavy snowfall, we tested the towing of the oceanographic station train with the J-5 Bombardier and experienced no difficulties. A comfortable towing speed of 12 km/h was reached and sustained.

The cooling air intake of the J-5 required to be extended for operations in low temperatures to above the superstructure to prevent snow crystals, set in motion by the movement of the tracks, from accumulating at the radiator air intake.

The sample transport back to base did not give any problems. Ice cores are placed, after frozen dry, in a bed of rags into the core boxes which for this purpose are adequately dimensioned. Water and biological samples are transported in 2"-styrofoam-lined field
transport cases. Any instruments were packed in foam rubber sheeting and placed at the rear end of the sleds only, in order to minimize shocks.

Sample transport to the South, especially frozen or live samples, by official air carrier was found to be unreliable as to treatment of shipment and the time it took to transport.

5. Summary

Throughout all experience periods, a number of prerequisite requirements were observed and are summarized as follows:

5.1. Prerequisites -- We have always worked with checklists identifying all items required for any particular trip out to the ice. The safety check list portion is always the same, whereas the programme check list varies with what is intended to be measured, observed or sampled.

Safety check list (extract) as example:
-- Fuel for vehicles - distance/quantity plus 50%.
-- Spare spark plugs, drive belts, shear pins, coils and other assorted hardware, plus fully equipped tool box and, for trips in excess of 75 km, under-carriage assemblies.
-- First Aid kit and, during certain periods, spare sunglasses.
-- Food for two days on any trip exceeding 30 km, also Coleman stove and fuel, matches and pots, pans, cups, bowls and cutlery.
-- Extra clothing for trips in excess of 75 km and sleeping bags, also one tarpaulin per person.
-- Portable radio on any trip exceeding 30 km, spare batteries, small caliber rifle and ammunition, one box of flares.

Rules of travel are simple; always two persons or party travel within others sight, stop every 30 minutes for a short walk, and every 90 minutes for tea during the cold period.

During the break-up period, the towline must be secured to the vehicle in such a manner and be of such dimension that a vehicle breaking through the ice can be retrieved. For each sled two planks 25 cm wide, 5 cm thick and 4 m long are carried along for crossing leads.

5.2. Improvements -- An arctic base-line data collection must by necessity go on year-round to have any significant and real value for the community-at-large and properly planned transportation of personnel, instruments and equipment, as well as supplies on the sea ice then provides an opportunity and solution for the problem of how to cover about 80% of the Canadian arctic waterways for about 75% of the year.

There do not appear to be major or complex problems in maintaining transport on first-year sea ice in most channels, sounds, inlets, bays or straits in the Canadian Arctic; however, a number of items should be considered in order to improve such operations.
-- An effort should be made to describe, measure and categorize the physical characteristics of the sea-ice and snow surface, and to standardize the relevant terminology.
-- There is a public need for the prediction of sea-ice surface travel conditions, as it must be recognized that there are periods during the ice-covered season when more than 80% of the Inuit population travel on the ice and hunt on the ice, in addition to us few base-line data collectors.
Arctic weather forecasting and climate description is on the whole aviation-oriented, but the sea-level climatic regime is basically different, at least for this region here, from the land-based regime during the ice-covered period. Sea-level climatic regimes must be established.

Manufacturers and designers of anything sold in and for the Arctic to be used outside in the "field" as we say, should utilize available technology to prevent hazards. Perhaps the Canadian Standards Association with government support and directive should set mechanical and electrical standards for items to be used in arctic climate.

Of particular concern to all should be the problem and solution of the CO poisoning or intoxication while operating snowmobiles under arctic conditions on sea ice.

Although the manufacturer of the Argo vehicles seemed to be interested during 1973/74 in the arctic application of his vehicle and corresponding modifications, none of his plans for proper engineering development materialized, leaving us to our own devices to make modifications for arctic use of this basically sound concept of an AATV.

Approaches to the manufacturers of Bombardier vehicles regarding modifications for arctic use have yielded no results. Here again, available technology should find application.

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With the sustained moral support by many individuals in the South the author, in residence in Pond Inlet, was able to carry a difficult task and he has ample evidence that the support received so far is gratefully acknowledged by the Inuit who look forward to participation with a long-term continuation of the base-line data-collecting programme.

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ABSTRACT

Results of several simulations of sea ice dynamics in the southern Beaufort Sea are examined to determine how sensitive velocity, deformation and stress fields are to variations in large-scale strength of the ice cover. An elastic-plastic material model developed as part of the AIDJEX program is used. Both the compressive and shear strengths are adjusted to provide a reasonable simulation of the velocity, deformation and stress fields simultaneously. When compressive strength is an order of magnitude lower or higher than this value, then velocity of the ice cover satisfies either free-drift or is driven almost entirely by the boundary motion. When the shear strength is increased by changing the yield surface from a squished teardrop to a triangle, the stress and deformation that occur as a large-scale lead system opens uniaxially are found to satisfy physical intuition. From the simulations studied, compressive strength during spring 1975 and winter 1976 are estimated to be on the order of $0.4-1.0 \times 10^5$ Nm$^{-1}$.

INTRODUCTION

One of the main goals of the AIDJEX program has been to develop a mathematical model of the behavior of sea ice that can simulate observed motions, deformations and stresses accurately. Progress in this task has been achieved through a sequence of basic physical assumptions followed by testing of the ideas using observations from the AIDJEX experiment and other sources (Coon et al., 1974). The model uses as input the atmospheric barometric pressure field and can simulate the behavior of any chosen region of the Arctic Ocean. In addition to modeling the ice, the equations also represent the atmospheric and oceanic boundary layers.

Several simulations of ice dynamics observed during the AIDJEX main experiment from March 1975–May 1976 have been performed (Coon et al., 1976; Pritchard, Coon and McPhee, 1976; Coon, Hall and Pritchard, 1976; Pritchard et al., 1977). In each of these calculations we are able to show either how the ice model responds under given conditions or how selected parameters in the model affect solutions.

The purpose of the present study is to show how yield strength in the elastic-plastic ice model affects model response. By selecting data from several calculations, we show that yield strength plays an important role in determining ice behavior and we estimate what values are correct for the conditions simulated.

The model used in this study is presented. It is shown analytically that the magnitude of the ice stress divergence averaged over a large area is limited by the yield
strength. This result allows us to estimate the maximum effect of ice stress and strength on the average velocity of the region. The results of several simulations are discussed to ascertain the effect on dynamic response due to using different values of yield strength. Finally, in one simulation the shape of the yield surface (the shear strength) is varied to show how this variable affects velocity, deformation and stress fields.

**MODEL**

According to the AIDJEX model the large-scale motion of sea ice is affected by air stress $\tau_a$, water drag $\tau_w$, Coriolis force $m f_c k \times v_g$, sea surface tilt $m f_c k \times g$ where $v_g$ is geostrophic ocean current and by the ice stress divergence $\nabla \cdot \sigma$ [e.g., Coon et al., 1974]. Momentum balance is given by

$$ m \frac{\partial \mathbf{v}}{\partial t} = \tau_a + \tau_w + \nabla \cdot \sigma - m f_c k \times (v - v_g) \tag{1} $$

where $m$ is the mass per unit area and $f_c$ is the Coriolis parameter. The symbol $(')$ denotes material rate of change of $( )$.

The air stress is assumed to be specified independently—determined from the surface barometric pressure field and a model of the atmospheric boundary layer. Water drag satisfies a relationship that is quadratic in $v - v_g$ (McPhee, 1976).

$$ \tau_w = \rho_w C_w \|v - v\| B(v - v_g) \tag{2} $$

where $\rho_w$ is water density, $C_w$ is a dimensionless drag coefficient and

$$ B = \begin{pmatrix} \cos (\pi + \beta) & -\sin (\pi + \beta) \\ \sin (\pi + \beta) & \cos (\pi + \beta) \end{pmatrix} \tag{3} $$

gives a rotation through angle $\pi + \beta$, counterclockwise. The geostrophic current $v_g$ is presently specified as the long-term mean. The stress-deformation law is assumed to be elastic-plastic (Coon et al., 1974; Coon and Pritchard, 1974; Pritchard, 1975). We now describe the elements of the constitutive law used in this work.

An isotropic yield surface of the form

$$ \phi(\sigma_I, \sigma_{II}, p^*) = 0 \tag{4} $$

is used. The stress invariants are $\sigma_I = \frac{1}{2} tr \sigma$ and $\sigma_{II} = (\frac{1}{2} tr \sigma' \sigma')^{\frac{1}{2}}$ where $\sigma'$ is the stress deviator. The parameter $p^*$ is the compressive yield strength and is to be determined as a function of the ice thickness distribution (Coon et al., 1974; Thorndike et al., 1975; Rothrock, 1975). However, evidence shows that strengths obtained by these relationships are incorrect (Pritchard, 1976). Therefore, we have not considered the thickness distribution in this study. Instead, we have specified $p^*$ as an independent parameter, keeping it constant in each simulation (perfect plasticity), but using different values. From these results we learn whether or not velocity, deformation or stress fields are sensitive to the value of $p^*$ and learn what values are needed to simulate ice dynamics accurately. The function $\phi$ defines the shape of the yield surface in stress invariant space. During the course of this work we have considered the three different yield surfaces shown in Figure 1.

Plastic stretching $\mathbf{D}_p$ satisfies the associated flow rule

$$ \mathbf{D}_p = \lambda \frac{\partial \mathbf{\phi}}{\partial \sigma} \tag{5} $$

where $\lambda$ is a positive scalar. The elastic response is linear

$$ \sigma = (M_1 - M_2) \frac{1}{2} tr \varepsilon + 2M_2 \varepsilon \tag{6} $$
where $\varepsilon$ is elastic strain. The elastic response is to be stiff enough to approximate rigidity. We use $M_1/p^* = 500$ and $M_2/p^* = 250$. These values limit elastic strains to about 0.25 percent and satisfy this constraint. The elastic strain is related to stretching $\varepsilon$ and spin $\omega$ by

$$\dot{\varepsilon} - \omega \times \varepsilon = \varepsilon - \varepsilon_p$$  \hspace{1cm} (7)

a linearized kinematic relationship that assumes small elastic strains (Pritchard, 1975).

There are conditions when stress plays a dominant role in controlling ice response, and other conditions when it is unimportant. The relative importance of stress divergence in equation (1) may depend on problem geometry, boundary conditions, air stress, the ice conditions and on the length scale under consideration. We may understand this last effect by averaging forces acting over an arbitrary region $R$ bounded by a curve $L$. The momentum balance equation appears identical to equation (1), but each term is an average over $R$. Thus, we find

$$m \ddot{\mathbf{u}} = \mathbf{T}_a + \mathbf{T}_w + \mathbf{f}_\sigma - m \mathbf{f}_c \times (\ddot{\mathbf{u}} - \ddot{\mathbf{u}}_g)$$  \hspace{1cm} (8)

where

$$\ddot{\mathbf{u}} = \frac{1}{A} \int_{R} \mathbf{u} \, da \quad \mathbf{T} = \frac{1}{A} \int_{R} \mathbf{T} \, da \quad \mathbf{f}_\sigma = \frac{1}{A} \int_{R} \nabla \cdot \mathbf{q} \, da$$  \hspace{1cm} (9)

are the average velocity, body traction and ice stress divergence. While spatial variations in the driving forces and ice motion cause the average values to be less than local maxima, they are often the same order of magnitude. However, we shall now show that $\mathbf{f}_\sigma$ is bounded by material properties and the bound is decreasing function of the size of $R$. This is a material property that is independent of problem geometry or driving forces and depends only on the state of the ice cover (yield strength) and on the size of region $R$.

By the Green-Gauss theorem equation (9) is

$$\mathbf{f}_\sigma = \frac{1}{A} \oint_{L} \phi \, \mathbf{n} \, dl$$  \hspace{1cm} (10)

where $\mathbf{n}$ is the outward unit normal from $R$ on $L$. The magnitude of $\mathbf{f}_\sigma$ is

$$||\mathbf{f}_\sigma|| = \left( |f_x|^2 + |f_y|^2 \right)^{1/2}$$  \hspace{1cm} (11)

where $f_x$ and $f_y$ are the Cartesian components

$$f_x = \frac{1}{A} \oint_{L} (\sigma_{xx} n_x + \sigma_{xy} n_y) \, dl \quad f_y = \frac{1}{A} \oint_{L} (\sigma_{xy} n_x + \sigma_{yy} n_y) \, dl$$  \hspace{1cm} (12)

We seek upper and lower bounds on the components $f_x$ and $f_y$. Following Pritchard (1976) we relate Cartesian stress components to invariants and direction of principal stress and then introduce limitations on the stress invariants imposed by the yield constraint. Using the cylinder and cone yield surface (Figure 1b) we find

$$-p^* \leq \sigma_{xx}, \sigma_{yy} \leq 0$$  \hspace{1cm} (13)

$$-\tau^* \leq \sigma_{xy} \leq \tau^*$$  \hspace{1cm} (14)

where $\tau^* < \frac{1}{2} p^*$ and similar limits may be found for other functions $\phi$. We assume $p^*$ and $\tau^*$ are the largest values throughout region $R$.

To determine our best upper and lower bounds on $f_x$ and $f_y$ we decompose the boundary curve $L$ into two segments—and we decompose it two ways (see Figure 2). Let $L^+_X$ and $L^-_X$ be the portions on which $n_x$ is positive and negative, respectively. Similarly,
let $L^+_y$ and $L^-_y$ be the two parts on which $n_y$ is positive and negative, respectively. Then rewrite (12) as

$$A f_x = \int_{y_a}^{y_b} \left\{ \sigma_{xx} \mid_{L^+_x} - \sigma_{xx} \mid_{L^-_x} \right\} \, dy + \int_{x_a}^{x_b} \left\{ \sigma_{xy} \mid_{L^+_y} - \sigma_{xy} \mid_{L^-_y} \right\} \, dx \quad (15)$$

Introducing the algebraically largest (or smallest) values allowed by equations (13) and (14) for the integrands in (15) provides the maximum (or minimum) value of $f_x$.

We find

$$|f_x| \leq p^* (y_b - y_a) + 2 \tau^* (x_b - x_a) \quad (16)$$

A similar argument provides

$$|f_y| \leq 2 \tau^* (y_b - y_a) + p^* (x_b - x_a) \quad (17)$$

The geometry of $R$ is arbitrary but if we choose a circle of diameter $\lambda$ then $x_b - x_a = y_b - y_a = \lambda$ and $A = \pi \lambda^2 / 4$. Substituting into (16) and (17) and using the norm introduced in (11) provides

$$\|f_0\| \leq \frac{4\sqrt{2}}{\pi} p^* + 2 \tau^* \quad (18)$$

If we replace the circle defining $R$ by the circumscribed square with horizontal and vertical sides then the coefficient $4\sqrt{2}/\pi$ is replaced by $\sqrt{2}$. Similarly, if we replace the circle with the inscribed square with diagonal sides then the coefficient becomes $2\sqrt{2}$, changing $\|f_0\|$ by -22 percent and 57 percent, respectively. The shape of the yield surface also affects the magnitude of the upper bound on $\|f_0\|$ but it still varies inversely with $\lambda$. For the squished teardrop yield surface (Figure la) and for the triangle yield surface (Figure lc) the expression $p^* + 2 \tau^*$ is replaced by $1.6 \ p^*$ and by $4 \ p^*$, respectively. Considering the diamond yield surface (the cylinder and cone with $\tau^* = \frac{1}{2} \ p^*$) as a standard, we see that the other yield surfaces change $\|f_0\|$ by -20 and +100 percent.

The numerical values given by equation (18) are presented in Figure 3. When strength is small, say $p^* + 2 \tau^* = 2 \times 10^3 \ \text{Nm}^{-1}$ we see that the ice stress can contribute a force no larger than 0.04 Pa in magnitude even at the fundamental gauge length of 100 km. This force is reduced to less than 0.01 Pa when averaged over a gauge length of 500 km. The maximum effect of ice stress on the average velocity is then comparable to a modest error in air stress of the same magnitude. Therefore, for a material this weak, the velocity may be simulated accurately by a local free-drift model. As the strength increases the magnitude of stress divergence increases linearly. When $p^* + 2 \tau^* = 2 \times 10^5 \ \text{Nm}^{-1}$ we require a gauge length of 1200 km to be certain that stress divergence has a magnitude less than 0.04 Pa. In this case the winds over a large area affect the behavior of the ice cover.

**COMPRRESSIVE STRENGTH EFFECTS**

In a recent paper Pritchard et al. (1977) have presented results of a simulation of winter ice dynamics in the nearshore Beaufort Sea. The time period is 27 January - 3 February 1976 and the domain is a rectangular region from the coast of Alaska north to about 75° latitude and from Banks Island westward to Barrow, Alaska. The ice behavior at this time is ideal for testing an ice model because it is measured accurately and because several important features are present. During the first two days the ice is motionless even though modest winds blow. Then as winds increase to larger values over the entire region the ice begins to move westward. The motion begins on the western boundary and propagates eastward during the next three days until reaching Banks Island. The dominant mode of deformation is uniaxial opening across a line extending orthogonally from the Alaskan shore. This opening of a lead
system is verified by NOAA-4 satellite imagery. Furthermore, there is a fast ice region nearly 80-km wide along the Alaskan coast. The velocity field is discontinuous along the line separating this fast ice from the moving pack.

The perfect plasticity ice model with a squished teardrop yield surface is used and it is shown that the model with a compressive strength of \( \sigma^* = 10^5 \) Nm\(^{-1} \) produces daily displacement fields for each of the eight days that represent observed motions very accurately. In addition, a simulation made with the same model but with \( \sigma^* = 10^6 \) Nm\(^{-1} \) produces daily displacement fields that are not accurate representations of observed motions but are more nearly like free-drift velocities. During the first two days the latter run shows ice motions as large as 15 km per day when the ice is observed to be motionless. With this low strength there is no fast ice region and no development of a flaw lead. On the other hand, when behavior is computed with \( \sigma^* = 10^5 \) Nm\(^{-1} \), the fast ice appears but the flaw lead is too far from shore. In this case motions are dominated by boundary conditions.

Another simulation of ice dynamics during the AIDJEX observation program is reported by Pritchard, Coon and McPhee (1976). The time period is 15-25 May 1975 and the region considered is a roughly circular region of about 800 km diameter centered around the manned array. The ice model used in that simulation (75 Run 1E) includes the squished teardrop yield surface and compressive strength is evaluated from a hardening law that depends on the thickness distribution. For the initial ice thickness distribution chosen, we obtained a strength of \( \sigma^* = 1 \times 10^3 \) Nm\(^{-1} \). The results of this simulation show that the daily displacement field is reasonable and displacement of the manned camps is modeled accurately. The simulated displacement field (75 Run 1E) during 18 May is shown in Figure 4. The spatial variations are typical of those observed throughout the ten-day interval. The velocity discontinuity at the boundary also is frequently observed. The strain occurring during 18 May is shown in Figure 5. The fact that strains are concentrated at the boundary is unreasonable. The large amount of shear at the boundary causes the formation of open water through mechanical redistribution. The compressive strength is then reduced and this softening then reduces the resistance to further deformation. As a consequence, we have feedback that is unstable (although for some thickness distribution it could become stable).

We suspect that the strength provided by the hardening law is too low. Therefore, we have performed a simulation of the same conditions with a compressive strength of \( \sigma^* = 4 \times 10^5 \) Nm\(^{-1} \) (75 Run 1F). Since we have not yet been able to modify any parameter in the hardening law to increase the strength of this value sensibly, we have used a perfectly plastic model. It is felt that the observed clockwise gyral motion would tend to induce convergence in the domain. This deformation would then strengthen the material. We feel that the chosen strength is high enough so that convergence would be small, making the perfectly plastic model realistic. The squished teardrop yield surface has been retained. The displacement field computed for 18 May (75 Run 1F) is presented in Figure 6. The spatial variability is reduced in the interior and also the velocity jump at the boundary is reduced in magnitude. However, even though the jump is smaller it still exists and this fact is typical of results throughout the simulation. We have found that the jump that remains is caused by accelerations of the boundary points that cannot be matched by the internal response of the material model. This fact is observed during the simulation of winter ice dynamics (Run 3C). In that calculation a sequence of steady state simulations is performed for each day and there are no jumps at the boundary. A comparison simulation (Run 3F) in which smoothly varying (a running 24-hour mean) air stress and boundary motions are input produces jumps in the ice velocity at the boundary. From this pair of calculations it is found that model response time is also a cause of the boundary discontinuity.
The strain field during 18 May obtained with the stronger, perfectly plastic model is presented in Figure 7. It is seen that strains in the interior are insensitive to the change in strength. However, strain around the boundary is reduced dramatically, which is consistent with the reduction in the jump in velocity. Only at the western edge are strains still large and they have been halved.

To test the accuracy of the simulated response we have compared modeled motion of the appropriate node with the observed motion of the manned camp Big Bear (see Pritchard, Coon and McPhee (1976) for the location of manned camps and data buoys). The velocity magnitude and direction (filtered with a 24-hour running mean) are shown in Figure 8. It is difficult to choose which model simulates speed better. During early times the strong perfectly plastic model (75 Run 1F) is better, but during the last four days when speeds are lower, the opposite is true (Figure 8a). By this time the hardening law has provided a strength of about $2 \times 10^3$ Nm$^{-1}$. However, the direction of flow is consistently better for the stronger model (Figure 8b).

As a final test of the accuracy with which we represent observed behavior we present the deformation relative to the configuration on 15 May (see Pritchard 1974 for a description of the strain tensor). We have chosen to present the strain in terms of the sum and difference of principal values ($\varepsilon_I$ and $\varepsilon_{II}$, respectively) and the direction is also presented to define the deformation completely. In Figure 9a, it is seen that the first invariant $\varepsilon_I$ is observed to be positive (indicating opening) but small. The stronger model (75 Run 1F) simulates the general trend well while the weaker model (75 Run 1E) shows negative values. It should be pointed out that the magnitude of these strains is small (0.1-0.3 percent per day) and it has been shown by Thorndike and Colony (1977) that observed strains may have an error of about 0.5 percent per day calculated from the motion of four stations. However, the fact that the trend continues in the opposite direction is significant and the stronger model is better. In Figure 9b the shear is presented. The magnitude of shearing $\varepsilon_{II}$ averages 0.4 percent per day. Both modeled results show larger shearing than observed during the first five days, although the stronger model halves the error. During the last five days the stronger model shows no shearing as observed, while the weaker model shows a reduction from the early excess. Again, the stronger model is more accurate, but we have not simulated $\varepsilon_{II}$ accurately during the first five days using either model. It is likely that increasing the shear strength would be more effective at reducing $\varepsilon_{II}$.

The direction of the larger principal strain is shown in Figure 9c. Both models tend to represent the observed direction, but only the stronger model is accurate. The weaker model shows the principal direction to be about 30° clockwise from the observed direction. Finally, in Figure 9d we present the rotation. The stronger model simulates rotation accurately while the weaker model shows rotation rates from two to three times as large as observed. We at first suspected that the stronger model simulates rotation well simply because the entire domain is rotating rigidly. If such were the case, a model that is stronger than reality would perform well in simulating rotation. However, we have learned that the rotation of the manned array is different from the average rotation of the domain (Pritchard and Reimer, to appear in a future AIDJEX Bulletin) and so we must conclude that the improvement is meaningful.

**Shear Strength Effects**

In this work we use the term shear strength when we refer to the shape of the yield surface. It is not one parameter as is the compressive strength $p^*$, but it is some measure of maximum shear stress that can be reached for a given yield surface. In this section we compare the simulated ice behavior during 27 January - 3 February 1976 for the squished teardrop (Figure 1a) and the triangle (Figure 1c) yield surfaces. The triangle gives the maximum ratio of shear to compressive strength while requiring a concave
surface. The compressive strength $p^* = 10^5 \text{ Nm}^{-1}$ is used in both cases. We have discussed results earlier for the squished teardrop at this compressive strength (Run 3C).

We have chosen to simulate conditions during 30 January using the triangle yield surface (Run 3E). The daily displacement field is presented in Figure 10. It is seen that a fast ice region about 150-km wide occurs off the Alaska north slope. This is nearly twice as wide as obtained with the squished teardrop, and does not match observed motions as well near the flaw lead or the manned array. However, the region near Banks Island is at rest (as observed by drifting buoys) rather than moving as calculated using the squished teardrop (Run 3C). The displacement field calculated with the triangle and $p^* = 10^5 \text{ Nm}^{-1}$ is somewhat similar to the velocity field calculated with the squished teardrop at $p^* = 10^6 \text{ Nm}^{-1}$. However, consideration of the daily strain fields points out important differences clearly. The strain calculated using the squished teardrop at $p^* = 10^6 \text{ Nm}^{-1}$ is unreasonably small and there is no realistic representation of either the flaw lead or the opening of north-south leads near the manned array. On the hand, the strain field calculated with the triangle at $p^* = 10^5 \text{ Nm}^{-1}$ (shown in Figure 11) does represent these important details accurately. The maximum shear across the flaw lead is about 16 percent per day (as also computed with the squished teardrop at $p^* = 10^5 \text{ Nm}^{-1}$). In the interior there is uniaxial opening of about 5 percent per day in a northwest-southeast direction as observed. Furthermore, this opening occurs across a line that is nearly orthogonal to the Alaskan coast. This uniaxial opening describes the opening of leads in this location that are observed in NOAA satellite imagery.

The stress field accompanying this behavior is presented in Figure 12. It is seen that a uniaxial stress state occurs. There is no stress acting across the opening leads. This is an important improvement in results compared with the squished teardrop at $p^* = 10^5 \text{ Nm}^{-1}$. In that case both principal stress components are similar in magnitude and there is a large traction acting on the opening lead, a physically unrealistic result. The triangle yield surface allows uniaxial stress and stretching states to occur simultaneously to describe the opening of a lead system. This is physically realistic and a desirable property of the ice model that is not represented accurately by the squished teardrop. However, the increase in shear strength gives larger stress divergence contributions to momentum balance. To approximate the velocity field accurately requires that the compressive strength be lowered accordingly. It is not known what value of $p^*$ would give the best approximation of the velocity field, but the present value gives a good approximation of both velocity and deformation. It would also be reasonable to use a diamond (the cylinder and cone shown in Figure 1b where $\tau^* = \frac{1}{2} p^*$). The compressive strength could be approximately $2 \times 10^5 \text{ Nm}^{-1}$ when using the diamond to get the same response.

CONCLUSION

Yield strength has been shown to be an important parameter in accurately simulating sea ice dynamics. This is true for both compressive and shear strength—or for both the size and shape of the yield surface. Motions may be similar to free-drift if strength is an order of magnitude too small and dominated by boundary motions if an order of magnitude too large.

If the opening (uniaxial) of a large-scale lead system is to be modeled without having large tractions acting across the leads, then part of the yield surface must lie along the tensile cutoff line so that one principal stress may be zero when the other is in compression.

Although the perfect plasticity model used in these simulations allows accurate simulations, it is important in the future to learn how hardening and softening affect the ice behavior. There are conditions where the perfectly plastic model must fail and a hardening/softening model is necessary to simulate these conditions accurately.

It has been shown that when the yield surface is chosen properly the AIDJEX model simulates motion and deformation of the ice cover accurately. Furthermore, the stress fields are realistic and felt to be accurate when yield strength is accurate.
When ice conditions are comparable to those observed during spring 1975 (15-25 March), a compressive strength on the order of $4 \times 10^4$ Nm$^{-1}$ is reasonable if a squished teardrop yield surface is used. When ice conditions are comparable to those observed during winter of 1976 (27 January - 3 February) a compressive strength on the same order is anticipated if the triangle yield surface is used or twice as large if the diamond yield surface is used. Furthermore, it is conjectured that use of the triangle (or diamond) yield surface for the spring 1975 simulation would improve the simulation of shear strains.

ACKNOWLEDGMENTS

I thank Max Coon who, in addition to his many other contributions to the AIDJEX program, first suggested that the triangle yield surface would possess properties desirable for simulating ice dynamics accurately. This work was supported by the National Science Foundation through Grant OPP71-03041 to the University of Washington for the Arctic Sea Ice Study.

REFERENCES


Fig. 1. Yield surface in stress invariant space.

Fig. 2. Arbitrary region $R$ showing boundary notation. Segment $L_y$ shown for clarity.

Fig. 3. Decay of magnitude of maximum average stress divergence.
Fig. 4. Displacement during 18 May (00-24 GMT) using AIDJEX hardening law (75 Run 1E). Displacement is proportional to scale vector.

Fig. 5. Strain during 18 May (00-24 GMT) using AIDJEX hardening law (75 Run 1E). Orthogonal lines show principal strain proportional to scale vector. Dotted lines show opening and solid lines closing.

Fig. 6. Displacement during 18 May (00-24 GMT) using perfectly plastic model and a strength of $4 \times 10^7 \text{Nm}^{-1}$ (75 Run 1F). Displacement is proportional to scale vector.

Fig. 7. Strain during 18 May (00-24 GMT) using perfectly plastic model and a strength of $4 \times 10^7 \text{Nm}^{-1}$ (75 Run 1F). Orthogonal lines show principal strains proportional to scale vector. Dotted lines show opening and solid closing.
Fig. 8. Comparison of displacement history of manned array.

Fig. 9a,b,c,d. Comparison of strain history of manned array.
Fig. 10. Daily displacement during 30 January (00-24 GMT) from Run 3E.

Fig. 11. Daily strain during 30 January (00-24 GMT) from Run 3E. Orthogonal lines show principal strain proportional to scale vectors. Dotted lines show opening and solid closing.

Fig. 12. Stress on 30 January from Run 3E. One principal stress is zero and the other is shown proportional to scale vector.
ESTIMATING THE DEFORMATION OF SEA ICE

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INTRODUCTION

In current treatments of the large-scale interaction between sea ice and its environment, an important role is assigned to the deformation of the ice pack. Deformation occurs by the three mechanisms of pressure ridging, the formation of open leads, and shearing along existing leads.

Pressure ridges are important, from a dynamical point of view, primarily because of the work required to form them. For a given deformation, the work done in ridging can be used to estimate the state of stress in the ice pack. Ridges are also important because of the large proportion of mass tied up in them. They affect the roughness characteristics of the top and bottom surfaces of the ice, and are thought to have anomalous rates of ice growth and melting.

The presence of leads has two consequences. The strength of the ice pack decreases as the area of leads increases. If the fractional area of leads is as large as 10%-20%, it is thought that the pack offers no resistance to applied forces. From a thermodynamic view, a vigorous exchange of heat and moisture occurs over leads, and the ice grows rapidly as a lead begins to freeze over.

Some shearing displacement along a wide lead can occur with no effect, but as the displacement increases, the irregular lead edges will come into contact and locally large stresses will cause the ice to break. Long, narrow regions in which floes appear pulverized are thought to be regions of concentrated shear deformation.

Having defined a probability density function to describe the relative abundance of ice of different thicknesses, Thorndike et al. [1975] developed a model which parameterizes the three deformation mechanisms in terms of a given strain rate tensor. This model has been coupled with a mechanical constitutive law [see Coon et al., 1974] to calculate the state of stress in a differential element of sea ice. The input data for these calculations are estimates of strain rate components. In what follows we discuss one measurement set from which strain rate estimates can be made, the procedure for making the estimates, and how the estimated quantities should be interpreted.

The deformation mechanisms are active at the boundaries separating nearly rigid floes which have length scales of up to tens of kilometers. The motion on such a scale is piecewise rigid with discontinuities between the floes. Along the lines argued by Maykut et al. [1972], an element of sea ice with a length scale of 100 km can be expected to contain many floe-to-floe discontinuities and therefore to have a well-defined average deformation. Such an element is still small compared with the wavelength of the Nye and Thomas curve [1974; also Nye, 1975], shown in Figure 1,
which is presumably related to the scale of atmospheric systems, or approximately 1000 km.

One of the objectives of AIDJEX was to measure the deformation of a single 100 km element. To this end, positions were measured about 30 times a day at each of four stations using satellite positioning systems accurate to about ±60 m. Each station was established on a single floe, so the raw measurements define the path followed by four distinct floes. The measurement set discussed here begins on 1 May 1975 and ends on 1 October 1975, when the residents of one station observed at close hand the mechanisms of deformation discussed above. (Measurements from the other three camps continued until 1 May 1976.) The configuration of the measurement array at several times is displayed in Figure 2.

ESTIMATING THE DEFORMATION

The motion of a point defines its position $x$ at some time $t_b$ in terms of its initial position $X$, at time $t_a$:

$$x = x(X, t_b, t_a).$$

(1)

The function $x$ can be approximated locally (at nonsingular points $X$) by the linear expression

$$x = c(t_b, t_a) + F(t_b, t_a)X,$$

(2)

where $F$ is called the deformation gradient. The vector $c$ is constant in space; it represents the rigid body translation of the system. Note that

$$c(t_a, t_a) = F(t_a, t_a) = I,$$

(3)

the identity.

The deformation gradient contains information about the strain and the rotation. It can be decomposed as

$$F = DS\theta,$$

where $D = \sqrt{e} I$ and $R$ is a rotation through a counterclockwise angle $\theta$. The symmetric matrix $S$ has principal direction $\psi$ and determinant 1. Thus, in principal coordinates, $S$ has the form

$$\begin{pmatrix}
\alpha & 0 \\
0 & 1/\alpha
\end{pmatrix}.$$ 

(4)

Under this decomposition, $e = \text{det} F$ is the ratio of the area of an element at $t_b$ to its area at $t_a$. $S$ is related to the shear. For small deformations, $\alpha$ is approximately 1 and $S - I$ will have the form

$$\begin{pmatrix}
\beta & 0 \\
0 & -\beta
\end{pmatrix},$$

which is the familiar pure shear representation.

This decomposition of $F$ differs from that of Pritchard [1974]. He defines a strain tensor $E = DS - I$. Invariants related to area change and to shear can then be defined in terms of principal values of $E$. The present decomposition is preferred here to illustrate in a later section that $F$ can be estimated in several steps related to the kinematic quantities of rotation, divergence, and shear.
From Equation 2, \( F \) can be written as \( \partial \mathbf{x} / \partial \mathbf{x} \). Suppose we require an estimate of \( F \) for some region \( A \). The Green-Gauss theorem relates the average of \( F \) inside \( A \) to an integral around the boundary of \( A \):

\[
\iint_{\text{region } A} F = \int_{\text{boundary of } A} \mathbf{x} \times \mathbf{n},
\]

where \( \mathbf{n} \) is the outward-directed unit vector normal to the boundary and \( \times \) is the tensor product. If \( \mathbf{x} \) is known at enough points \( \mathbf{X} \) along the boundary of \( A \) to allow a good interpolation, then the right-hand side of Equation 5 can be estimated. Values of \( \mathbf{x} \) inside \( A \) do not affect the estimate.

Another technique to estimate \( F \) uses least squares theory. If pairs of values \( (x_i^N, X_i) \) for \( i = 1, \ldots , N \) are known, then estimates \( \hat{\mathbf{c}} \) and \( \hat{F} \) for \( \mathbf{c} \) and \( F \) (6 parameters) can be found to minimize

\[
\sum_{i=1}^{N} [x_i^N - (\hat{\mathbf{c}} + \hat{\mathbf{F}} X_i)]^2.
\]

In this procedure, points near the vector mean make only small contributions to \( F \). (In the measurements discussed here an array of four points was used, one of which was close to the vector average of the triangle defined by the other three.)

The least squares approach is followed here because of the well-developed theory for constructing confidence intervals for the estimated quantities. For our data set the boundary integral method and the least squares method give similar results for \( F \).

We make the hypothesis that the measurement region defined with length scale 100 km can be viewed as a differential element \( A \) of the ice pack over which Equation 2 holds, i.e., \( \mathbf{x} \) is a linear function of \( \mathbf{X} \). Since any three points in \( A \) determine \( F \), there are several possible ways to group the data to calculate \( F \), using data from the four AIDJEX camps shown in Figure 2. Here we have calculated \( F \) for each of the three inner triangles.

Figure 3 shows the time series of area relative to area on 1 May 1975 for each of the three inner triangles and for the least squares approximation. A favorable comparison between the curves would suggest that Equation 2 was a good representation of the motion of the array. Of course, the three results are not independent--each pair of interior triangles share a common side--so no particular importance should be attached to a similarity between the results from any two triangles. The results are striking for their poor agreement; they show few sustained periods in which the changes of area of all interior triangles have the same sign. On a shorter time scale, the daily change of area was calculated and correlations between triangles were made. The correlation coefficients for daily area change are: between BB-CA-\( \mathbf{X} \) and BB-CA-SB, \( \rho = 0.38 \); between BB-CA-BF and BB-BF-SB, \( \rho = -0.35 \); and between BB-CA-SB and BB-BF-SB, \( \rho = 0.03 \). One must conclude that the comparison is unfavorable and that Equation 2 is not a very close approximation to the actual motion.

**ESTIMATING THE NONLINEAR PART OF THE MOTION**

We wish to quantify the notion that over finite regions the linear Equation 2 is not a close representation of the motion. Our approach is to resolve the actual motion into linear and nonlinear parts and then to estimate the magnitude of each part.

Define a velocity

\[
\mathbf{v} = v(X,t,\Delta) = \frac{v(X,t,t+\Delta) - X}{\Delta}.
\]

Then, motivated by the Nye and Thomas [1974] curve in Figure 1, we decompose the true velocity field \( v(X,t,\Delta) \) into a smooth field and a perturbation:
where \( v_L \) varies nearly linearly over distances \( L \) or less. We have in mind \( L = 100 \) km. The spatial derivatives of \( v_L \) are continuous and nearly constant over \( L \).

Nye [1973] constructs a smooth velocity (roughly analogous to \( v_L \)) by convolving \( v \) with a boxcar kernel of length \( L \) and shows, in one dimension, that the derivative of the smooth velocity at \( x \) is identical to the difference of the true velocities measured at \( x \pm L/2 \). Note, however, that the two-dimensional analogy requires that measurements be made at all points on the boundary of the element. A second comment with respect to Nye's definition is that his strain on a length \( L \) involves smoothing the velocity with a kernel of length \( L \). For \( v_L \) to vary linearly over a length \( L \) in our definition, the kernel must have a length which is long compared with \( L \).

We would like to estimate \( v_L \) and its first derivatives, given measurements of \( v \) at a few discrete points. To be precise, we have measurements (denoted by a tilde) of the positions of several points (subscript \( j \)) at evenly spaced times (subscript \( i \)). Then the measured velocities are

\[
\tilde{v}_{i,j} = (\tilde{x}_{i+1,j} - \tilde{x}_{i,j})/\Delta,
\]

where \( t_{i+1} - t_i = \Delta; j = 1, \ldots, 4 \) stations; and \( i = 1, \ldots, N \) times. The measurements satisfy

\[
\tilde{v}_{i,j} = v(x_{i,j}, t_i, \Delta) + \varepsilon_{i,j},
\]

where \( \varepsilon_{i,j} \) is a measurement error. The position measurements in Equation 8 have errors of only a few tens of meters; velocity errors are about 50 m/\( \Delta \) < 0.05 cm sec\(^{-1}\). Estimates of \( v_L \) and its derivatives will be functions of the measurements, and the uncertainty in these estimates will depend on the number of measurements and on the variances of \( w \) and \( \varepsilon \). The goal now is to estimate the variance of \( w \). The observed velocity is partitioned into the linear velocity, the nonlinear velocity, and the measurement errors:

\[
\tilde{v}_{i,j} = v_L x_{i,j} + (w_{i,j} + \varepsilon_{i,j}),
\]

and, since \( v_L \) has linear variation,

\[
v_L x_{i,j} = B_i + A_i x_{i,j}.
\]

We want to find estimates \( \hat{B}_i \) and \( \hat{A}_i \) which minimize

\[
\frac{1}{4} \sum_{j=1}^{4} r_{i,j}^2
\]

where

\[
r_{i,j} = \tilde{v}_{i,j} - (\hat{B}_i + \hat{A}_i \tilde{x}_{i,j}).
\]

For each interval \( (t_i, t_{i+1}) \) we can use standard least squares techniques to estimate \( \hat{B}_i \) and \( \hat{A}_i \) and the variance \( \sigma_w^2 \) of \( w + \varepsilon \). One estimator of \( \sigma_w^2 \) is

\[
\sigma_w^2 = \frac{1}{4 - 3} \sum_{j=1}^{4} r_{i,j}^2
\]

which has the chi-squared distribution with 4 - 3 degrees of freedom: 4 measurements, 3 parameters (1 in \( B_i \) and 2 in \( A_i \)) [see Mood and Graybill, 1963, p. 351]. The estimates of \( \sigma_w^2 \) for each time are plotted in Figure 4. Since estimates made this way have but one degree of freedom, very little confidence can be placed in
them. A small value of \( \sigma_w^2 \), for example, is very weak evidence that \( \sigma_w^2 \) was actually small at time \( t_i' \). Even at a time when the four measurements fit some linear field very closely, it would be a mistake to think that the parameters of that field gave particularly good estimates of the true velocity derivatives. A far safer course is to believe that \( \sigma_w^2 \) is constant over some time period, and to estimate it with many degrees of freedom as follows.

Successive intervals \( (t_i', t_{i+1}) \), \( (t_{i+1}' , t_{i+2}) \) appear to be independent (Fig. 4), so that we can lump together \( N \) points in time and take

\[
s^2 = \frac{1}{4N - 3N} \sum_{i=1}^{N} \sum_{j' = 1}^{4} \nu_{i',j}^2
\]

(13)

to be an estimator of \( \sigma_w^2 \) with \( N \) degrees of freedom. Ninety percent confidence intervals for \( \sigma_w \) are found as

\[
90\% \text{ confidence interval: } \sqrt{\frac{s^2 N}{\chi^2_{0.95}}} < \sigma_w < \sqrt{\frac{s^2 N}{\chi^2_{0.05}}}
\]

(14)

These confidence intervals are given in Table 1. Several time intervals \( \Delta \) have been used to define the velocity.

**TABLE 1. Estimated standard deviation \( \sigma_w \) of nonlinear contribution to 100 km velocity field, with 90% confidence intervals.**

<table>
<thead>
<tr>
<th>Time</th>
<th>No. of Points</th>
<th>( \Delta ) Days</th>
<th>( \sigma_w ), in cm sec(^{-1})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( \nu )-component</td>
<td>( \psi )-component</td>
</tr>
<tr>
<td>1-30 May 75</td>
<td>30</td>
<td>1</td>
<td>0.44 ( \pm ) 0.12</td>
</tr>
<tr>
<td>31 May-29 Jun 75</td>
<td>30</td>
<td>1</td>
<td>0.50 ( \pm ) 0.10</td>
</tr>
<tr>
<td>30 Jul-18 Aug 75</td>
<td>20</td>
<td>1</td>
<td>0.96 ( \pm ) 0.24</td>
</tr>
<tr>
<td>19 Aug-17 Sep 75</td>
<td>30</td>
<td>1</td>
<td>1.07 ( \pm ) 0.23</td>
</tr>
<tr>
<td>25 Apr-14 Jul 75</td>
<td>8</td>
<td>10</td>
<td>0.28 ( \pm ) 0.11</td>
</tr>
<tr>
<td>3 Aug-2 Oct 75</td>
<td>6</td>
<td>10</td>
<td>0.45 ( \pm ) 0.21</td>
</tr>
<tr>
<td>25 Apr-14 Jul 75</td>
<td>4</td>
<td>20</td>
<td>0.30 ( \pm ) 0.17</td>
</tr>
<tr>
<td>3 Aug-2 Oct 75</td>
<td>3</td>
<td>20</td>
<td>0.56 ( \pm ) 0.36</td>
</tr>
<tr>
<td>25 Apr-14 Jun 75 and 3 Aug-22 Sep 75</td>
<td>1</td>
<td>50</td>
<td>0.28 ( \pm ) 0.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.21 ( \pm ) 0.17</td>
</tr>
</tbody>
</table>

The values in Table 1 are typically 0.5 cm sec\(^{-1}\). This exceeds the measurement error by an order of magnitude. Therefore, we attribute the tabulated values largely to the variance of \( \omega \). We conclude from the table that the nonlinear part of the velocity field \( \omega = \nu - \nu_L \) has a standard deviation in the spring of \( \sigma_w = 0.4 \pm 0.1 \text{ cm sec}^{-1} \). In the summer the standard deviation is larger: \( \sigma_w = 1.1 \pm 0.3 \text{ cm sec}^{-1} \). For longer times \( \Delta \) the departures from linearity are smaller. For \( \Delta = 20 \) days, \( \sigma_w \) equals 0.3 \( \pm \) 0.1 cm sec\(^{-1}\) in spring and 0.6 \( \pm \) 0.3 cm sec\(^{-1}\) in summer.

For a point of comparison, consider the full variance \( \sigma^2 \) of the velocity. Let an estimate of \( \sigma^2 \) be

\[
s^2 = \frac{1}{4N - 3N} \sum_{i=1}^{N} \sum_{j' = 1}^{4} (\tilde{\nu}_{i,j'} - B_{i,j'})^2
\]

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The full variance of the velocity, $\sigma^2$, includes the linear and the nonlinear variation in $v$ across the array. From Table 2, where $\sigma^2$ appears, we conclude that the full variations in $v$ over 100 km are typically 1.2 cm sec$^{-1}$ in spring and 2.1 cm sec$^{-1}$ in summer for one-day velocities, and that the variability decreases for larger $\Delta$.

We can deduce the linear variance using

$$\sigma^2 = \sigma_w^2 + \sigma_{\text{linear}}^2$$

Clearly, it will be impossible to estimate with confidence the coefficients of the linear variation ($A$ and $B$) when the measurements are contaminated by nonlinear variations of about 50%. The situation does not improve for longer time scales.

Measurements of the displacement of sea ice can also be made using remote sensing techniques. From a pair of Landsat images, for example, it is possible to estimate the displacements of many identifiable points within a 100 km region over a one-day period. In an unpublished note describing such measurements, Nye and Hall report that the residuals (the difference between each displacement and the best linearly varying displacement field) are typically 0.32 km (0.37 cm sec$^{-1}$), based on five pairs of images and about 20 displacement measurements per pair, from March and April 1973. Our result of $\sigma_w = 0.4$ cm sec$^{-1}$ from spring 1975, and using a different measurement technique, confirms their value.

Confidence intervals for the velocity derivatives can be stated in terms of $\sigma_w$. If the problem is set up as $Z = \bar{X}\hat{\beta} + \varepsilon$, where $Z$ contains the measurement, $\bar{X}$ the geometry of the array, and $\hat{\beta}$ the unknown coefficients, then the least squares solution for $\hat{\beta}$ is $\hat{\beta} = (\bar{X}^T\bar{X})^{-1}\bar{X}^TZ$ and a 90% confidence interval for each coefficient is

$$\hat{\beta}_i \pm 1.64 \frac{\sigma_w\sqrt{\bar{C}_{ii}}}{\sqrt{\bar{C}}}$$

where $\bar{C} = (\bar{X}^T\bar{X})^{-1}$. For the array used here, with $\bar{x} = \bar{y} = 0$, the matrices are

---

**TABLE 2.** Estimated variability of velocity over 100 km. Tabulated value is estimated RMS value of $(v - \bar{v})$, with 90% confidence intervals.

<table>
<thead>
<tr>
<th>Time</th>
<th>No. of Points</th>
<th>$\Delta$</th>
<th>$\sigma^*$, in cm sec$^{-1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>($\mu$-component)</td>
</tr>
<tr>
<td>1-30 May 75</td>
<td>30</td>
<td>1</td>
<td>1.15 $\pm$ 0.15</td>
</tr>
<tr>
<td>31 May-29 Jun 75</td>
<td>30</td>
<td>1</td>
<td>1.15 $\pm$ 0.15</td>
</tr>
<tr>
<td>30 Jul-18 Aug 75</td>
<td>20</td>
<td>1</td>
<td>2.35 $\pm$ 0.35</td>
</tr>
<tr>
<td>19 Aug-17 Sep 75</td>
<td>30</td>
<td>1</td>
<td>1.90 $\pm$ 0.20</td>
</tr>
<tr>
<td>25 Apr-14 Jul 75</td>
<td>8</td>
<td>10</td>
<td>0.70 $\pm$ 0.20</td>
</tr>
<tr>
<td>3 Aug-2 Oct 75</td>
<td>6</td>
<td>10</td>
<td>1.00 $\pm$ 0.30</td>
</tr>
<tr>
<td>25 Apr-14 Jul 75</td>
<td>4</td>
<td>20</td>
<td>0.65 $\pm$ 0.25</td>
</tr>
<tr>
<td>3 Aug-2 Oct 75</td>
<td>3</td>
<td>20</td>
<td>0.65 $\pm$ 0.25</td>
</tr>
<tr>
<td>25 Apr-14 Jun 75 and</td>
<td>1</td>
<td>50</td>
<td>0.40 $\pm$ 0.20</td>
</tr>
<tr>
<td>3 Aug-22 Sep 75</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
\[
\beta = \begin{bmatrix}
B \\
A_x \\
A_y
\end{bmatrix}, \quad \bar{X} = \begin{bmatrix}
\bar{x}_1 \\
\bar{x}_2 \\
\bar{x}_3 \\
\bar{x}_4
\end{bmatrix}, \quad \bar{Z} = \begin{bmatrix}
\bar{v}_1 \\
\bar{v}_2 \\
\bar{v}_3 \\
\bar{v}_4
\end{bmatrix},
\]

and

\[
C = (\bar{X}^T \bar{X})^{-1} = \begin{bmatrix}
0.25 & 0 & 0 \\
0 & 1.04 \times 10^{-4} \text{ km}^{-2} & -0.37 \times 10^{-4} \text{ km}^{-2} \\
0 & -0.37 \times 10^{-4} \text{ km}^{-2} & 1.14 \times 10^{-4} \text{ km}^{-2}
\end{bmatrix}
\]

Thus, \( \sqrt{\sigma_{11}} = 1/2; \sqrt{\sigma_{22}} = 1.02 \times 10^{-2} \text{ km}^{-1}; \) and \( \sqrt{\sigma_{33}} = 1.07 \times 10^{-2} \text{ km}^{-1} \). These quantities are roughly proportional to \( n^{-1/2} \), and the last two are also proportional to \( L^{-1} \).

A histogram is shown in Figure 5 of velocity derivatives for spring data (60 one-day periods) computed as above. Also shown are several confidence intervals. The 90\% confidence interval is ±0.7 \( \times 10^{-7} \text{ sec}^{-1} \). If such an interval is applied as an "error bar" to each calculated velocity derivative, it will frequently--about 40\% of the time--be impossible to resolve, with confidence, even the sign of the derivative. In practice, quantities of most interest are combinations of velocity derivatives (divergence or curl, for example). Confidence intervals for those quantities can be constructed from \( \sigma_{ij} \) and \( C \). They generally can be expected to have estimation errors larger than the errors in the individual derivatives.

When the spatial derivative of velocity, \( A \), is large, the confidence interval is smaller in a relative sense. This conclusion depends on the assumption that the nonlinear part of the velocity field is independent of the size of the linear variation. A test of contingency between the calculated derivatives of \( v_L \) and the residuals \( \sum_j r_{v_j}^2 \) supported this assumption.

Conclusions from this section are the following:

1. The nonlinear part of the 100 km daily velocity field has a standard deviation of 0.4 cm sec\(^{-1}\). The linear part has a standard deviation of 1.1 cm sec\(^{-1}\). These numbers are greater in the summer and smaller for longer time scales, but their ratio is always about 0.5.

2. Since the available measurements include the nonlinear part of the field, estimates of the linear coefficients have associated uncertainties of about 0.7 \( \times 10^{-7} \text{ sec}^{-1} \).

3. The absolute variance \( \sigma_{\omega}^2 \) appears to be independent of the linear variation. Therefore, in a relative sense, large gradients may be estimated with errors as small as 20\%.

**TOTAL DEFORMATION OVER 150 DAYS**

We have discussed the spatial derivatives of the velocity field, but the displacement field can be treated analogously, the only difference being an appropriate division by \( \Delta \) to get from displacements to velocities. Consider as an example the change in configuration after 150 days. The least squares solutions for \( \Theta \) and \( \bar{\Theta} \) are
\[ \sigma(270,120) = \begin{bmatrix} -123.3 \text{ km} \\ -355.6 \text{ km} \end{bmatrix}, \; \hat{F}(270,120) = \begin{bmatrix} 0.6840 & -1.3448 \\ 0.3715 & 0.7237 \end{bmatrix}, \]

and the sum of the squared residuals \((x - \hat{F}X)^T (x - \hat{F}X)\) equals 355 km\(^2\).

To construct a confidence interval for the coefficients in \(F\), we take \(\sigma \approx 19\) km and get \(\pm 0.31\) (using Equation 17 and \(\sqrt{\sigma_{ij}} = [100 \text{ km}]^{-1}\)). It seems difficult to interpret these confidence limits in terms of, say, divergence or shear.

Alternatively, examine the decomposition discussion above: \(F = DSR\). We find

\[
F = \begin{bmatrix} 0.9973 & 0 & 0.7229 & -0.2940 & 0.6342 & -0.7732 \\ 0 & 0.9973 & -0.2940 & 1.5028 & 0.7732 & 0.6342 \end{bmatrix}.
\]

Table 3 shows how much the total variance can be reduced by including divergence, shear, and rotation in the deformation. For instance, when the motion is fit by the best irrotational tensor, \(x = DSX\), the unexplained variance is 14,076 km\(^2\). By including rotation, \(x = DSRX\), the variance is reduced further to 711 km\(^2\). The effect due to rotation then is the difference, 13,365 km\(^2\); the effect due to shear is 4329 km\(^2\); and the effect due to divergence is 1 km\(^2\). Tests for statistical significance confirm that \(C\) and \(R\) are definitely significant, that \(S\) is marginal, and that \(D\) might as well be set equal to the identity. The principal values of \(S\) are 1.5970 and 0.6228. Reducing each principal value by 1.0 implies a pure shear deformation of about 50%. The \(\pm 31\%\) confidence interval gives some idea how far to trust this shear estimate. This conclusion of course applies only to this particular set of data. The method is generally applicable and is a useful way to determine what parts of the motion are significant.

<table>
<thead>
<tr>
<th>TABLE 3. Variance for total deformation over 150 days. (Primes indicate that the array mean has been removed.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td>Divergence and shear</td>
</tr>
<tr>
<td>Divergence and rotation</td>
</tr>
<tr>
<td>Shear and rotation</td>
</tr>
<tr>
<td>Divergence, shear, and rotation</td>
</tr>
</tbody>
</table>

**DISCUSSION**

The data set considered here from spring and summer 1975 in the Beaufort Sea implies that the true deformation cannot be represented well as constant over lengths of 100 km. The scale of 100 km was originally selected for the measurement program based on an argument that it was large enough to contain many deformational features but small enough to resolve large-scale variations in the forcing field. By choosing a length which covered many floes, the argument ran, the velocity discontinuities between floes would tend to average out to the underlying linear velocity.
field. The argument is incomplete because it ignores the magnitude of the discontinuities. For a field with large amplitude discontinuities, even if there are many of them and they do tend to average out, it will be difficult to estimate the deformation based on only a few measurements. Each measurement will be affected by large local "errors." These can be averaged out by taking more measurements, but not by taking a larger length scale.

Equation 17 can be used in advance to predict the accuracy of estimates from a given array of points, and it may be a useful tool for designing future experiments. Of course, our estimate of $\sigma_w^2$ may not apply to different length scales.

Hibler et al. [1974] have discussed the variability of deformation estimates on a scale of about 10 km. Their term inhomogeneity variation refers to the uncertainty in deformation estimates due to the nonlinear variations of the motion over 10 km. There are important differences between their work and ours. They considered shorter length and time scales, and their data were obtained in March and April 1972. They used about a dozen measurement points. Some of their calculations may be distorted because nonindependent data appear to have been used. Nevertheless, their important conclusion that local variations significantly affect deformation estimates is well documented.

Following our definition, the nonlinear contribution to the velocity field, $\sigma_w^2$, is an increasing function of length scale. Whether $\sigma_w/L$ will increase or decrease with $L$ remains unanswered. The conjecture of Hibler et al. [1973], that $\sigma_w \propto L^{-1/2}$, should be regarded with caution.

The spatial velocity spectrum may appear somewhat as sketched in Figure 6. A length scale $L$ divides the spectrum into two parts, the areas of which are the variances of $v_L$ and of $w$. The results here imply that the area to the right of $f = 1/100$ km is roughly half the area to the left. In their Table 3 Hibler et al. [1974] indicate a value of $\sigma_w \approx 0.063$ cm sec$^{-1}$ determined over a 10 km scale. When compared with our value of $\sigma_w \approx 0.4$ cm sec$^{-1}$ for 100 km, Hibler's result implies that only a small percentage of the variance occurs at wavelengths of less than 10 km.

We have emphasized the difficulties in estimating the deformation of sea ice. These arise because of real nonlinear variations in the ice velocity field. The estimate of the variance of the nonlinear component given here can be used to design measurement programs aimed at estimating deformation. The general conclusion is that a dense array of measurements is needed to resolve the underlying deformation well. In addition to making it difficult to estimate the underlying deformation well, the sizable nonlinear variations imply that the underlying deformation does not fully characterize the deformation field. Isn't it time to re-examine our models of thickness distribution and stress-strain laws which are formulated in terms of the mean deformation?

ACKNOWLEDGMENT

We thank Rich Hall for providing the displacement measurements from Landsat images. This work was supported by the National Science Foundation Grant OPP71-09031, formerly GV 28807, to the University of Washington for the Arctic Sea Ice Study.
REFERENCES


FIGURES

Fig. 1. The variation of u, the x-component of displacement, with distance x along a line. Period 21-23 March 1973. From ERTS imagery. Reproduced from Nye and Thomas [1974].

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Fig. 2. Configuration of the measurement array at eight times separated by 20-day intervals during spring and summer 1975.

Fig. 3. Ratio of area at time $t$ to the area on 1 May 1975 for each inner triangle and for the least squares fit. Gaps occur in three of the curves because of missing data at one camp (BF).
Fig. 4. Daily estimates of $\sigma_u^2$. (+), $u$-component of velocity; (□), $v$-component of velocity.

Fig. 5. Histogram of spatial velocity derivatives for sea ice from 60 one-day periods in spring 1975.

Fig. 6. Power spectrum of spatially varying ice velocity. Curve is not based on data. Our only insight into the shape of the spectrum at this point is that the length scale of 100 km divides the variance (area under the spectrum) roughly in half.
INTRODUCTION

The Polar Gas Project was formed in 1972 with the aim of investigating the feasibility of a transportation system to move natural gas energy from the Arctic Islands to southern markets. Because of its higher throughput and lower unit costs, a pipeline emerged early in the investigations as the most attractive transportation option. The plan is to build the pipeline in two phases, the first running south from Melville Island along the west side of Hudson Bay joining the TransCanada Pipelines System at Longlac, Ontario, and the second extending farther north at a later date to discoveries in other areas, such as King Christian and Ellef Ringnes Islands. The major challenge that has been met in proving the technical feasibility of the pipeline is the establishment of construction methods for the marine crossings between the islands. The route selected traverses Bathurst Island, Cornwallis Island, Somerset Island and then to Boothia Peninsula, the most northerly land mass of mainland Canada. It includes five main inter-island crossings, Byam Channel, Austin Channel, Crozier Strait, Pullen Strait, and East Barrow Strait. These crossings have a cumulative length of 145 kilometers. The longest crossing is East Barrow Strait at 58.4 kilometers, and the deepest is Crozier Strait with a maximum water depth of 322 meters.

Of these crossings, Austin Channel and Byam Channel remain ice covered for most of the year with a relatively thick, stable ice cover occurring every year during the period March through the middle of June. These two channels are not therefore suitable for water-borne marine construction techniques but are ideal for construction techniques which can make use of the ice sheet as a working platform. As a result of the requirement to use the ice sheet in this way, laboratory, field and analytical studies were conducted, and continue to be conducted, in order to achieve a complete understanding of the capacity of sea ice to support construction loads. Ice-based pipeline installation techniques presently under consideration by Polar Gas will involve the deployment of construction equipment next to a trench or slot which has been cut in the ice. It was therefore considered particularly important to investigate the effect of a free edge on the load bearing capacity of an ice sheet, and emphasis was given to this aspect during the course of our studies.

The purpose of this paper is to describe the sea ice bearing capacity investigations which have been conducted during the past four years for the Polar Gas Project. These investigations can be readily categorized into laboratory studies, field studies and analytical studies and they will be described under these headings in
order to demonstrate clearly the extent of the work which has been carried out during this period.

LABORATORY STUDIES

At the time the Polar Gas investigations into sea ice bearing capacity commenced, relatively little information was available on the tensile and compressive strength of sea ice. The objective of the small sample test program was to determine the variation of the mechanical and rheological properties of sea ice with temperature and salinity. With these relationships, the vertical variation of temperature and salinity which are measured in a natural sea ice sheet can be used as input into a finite element program to determine the strength and elasticity of designated horizontal layers of the ice sheet. The integrated effect of the multi-layered properties of the ice sheet then gives the flexural rigidity and strength for the composite ice sheet.

In 1973 and 1974, blocks of sea ice, taken from the Byam Channel ice cover, were transported to the south in insulated boxes packed with dry ice to maintain them at low temperature and to prevent brine drainage. At the test laboratory, cylindrical ice samples from the horizontal plane with dimensions of one inch in diameter and three inches in length were tested in tension and compression to determine the ultimate strength and elastic modulus. Tests were performed at a strain rate of $10^{-4}\text{sec}^{-1}$ which is in the brittle range in tension, and which is in the transition zone from brittle to ductile in compression.

Methods which have been used in the past to test small ice samples in tension have included the ring tensile test, the small beam flexural strength test and the uniaxial tensile test. For many years the ring tensile test has been a popular method of testing small ice samples. This method requires that an elastic stress distribution be assumed in order to compute failure stresses. However, the time dependent plasticity of the ice, and its pressure melting phenomenon, raise doubts as to the validity of this method of testing. Another area of concern was that the internal state of stress for the ring tensile test sample is complex compared to the uniaxial test sample. For these reasons, the uniaxial technique was adopted for all small sample tests.

Plots were made of brine volume against strength and elastic modulus for both the tensile and compressive tests, and curve fitting was carried out to derive equations of exponential form. Traditionally, it has been assumed that strength varies with the square root of the brine volume, and that elastic modulus varies directly with brine volume. These relationships are not valid at high brine volumes since they yield negative values for strength and elastic modulus. Consequently, an exponential form was adopted for both strength and elastic modulus, since it yielded realistic low positive values at high brine volumes. Crystallographic analysis of the ice used in the small sample testing revealed that most of the ice was of the normal columnar type but that the upper layers contained saline snow ice in one block, and agglomerate sea ice in the other. The different ice types were found to vary in strength characteristics in some measure, and a composite exponential relationship was derived to account for this fact in further analysis.

As a result of the small sample test program, it became apparent that the compressive strength of sea ice is sufficiently great that the bearing capacity is not controlled by compressive failure of the ice sheet. Therefore, in the computations which were carried out during further analysis, the compressive strength was assumed to be 3.5 times the tensile strength at a given brine volume. Also, no evidence was found to suggest that there is a difference between the
elastic modulus in tension and the elastic modulus in compression at a given brine volume. This fact simplified the computation of stress distribution in the ice sheet, and all subsequent computations assumed the tensile elastic modulus relationship to apply throughout the ice sheet.

In addition to the tests described above which were carried out to determine the elastic properties of sea ice, a large number of small sample tensile creep tests were carried out in 1973, which were conducted to correlate the secondary creep strain rate to the temperature and applied stress. Another objective of these tests was to determine the total time to the onset of tertiary creep and the total strain at that time. The results showed great dispersion, which made interpretation difficult and quantitative creep laws could not be developed from the test results. However, it was concluded that the strain rate in the secondary stage of creep depends strongly on the applied stress and temperature. It was also concluded that the total strain at the onset of tertiary creep is weakly dependent on stress, but that the time to onset of tertiary creep decreases rapidly with increasing stress and is weakly dependent on the ice temperature.

A further area of laboratory study carried out by Polar Gas was a series of tests conducted on 1/50th scale synthetic ice sheets. The primary reason for carrying out these tests was to observe the effect of such variables as load area, distance between loads, ice geometry, and the distance of a load from a free edge, on the behaviour of the ice sheet. The information obtained from this laboratory work was extremely useful in planning an extensive series of full scale tests on the Byam Channel ice cover in 1974.

FIELD STUDIES

Although a limited field test program was conducted on Byam Channel in 1973, by far the most comprehensive series of tests took place during the period 27 March 1974 to 25 May 1974, also on Byam Channel. The program consisted of ice right-of-way studies, full scale load testing using water-filled tanks, and in-situ cantilever beam testing.

The objective of the ice right-of-way studies was to investigate various methods of modifying the natural sea ice cover to make a stronger, smoother working platform for marine pipeline construction. These studies consisted of the following field investigations:

(i) Clearing snow from an area of natural ice 300 meters by 60 meters to determine the effect upon the natural rate of growth of the ice sheet, and also to assess the benefits of snow fencing in keeping an ice right-of-way clear of snow;

(ii) Thickening an ice sheet by flooding an area of natural ice with sea water to determine the effect upon load bearing capacity caused by the increased ice thickness and by the change in mechanical properties of the overall ice sheet; and

(iii) Assessing different flooding techniques on sixteen ice plots each of which measured 30 meters by 30 meters. Variables which were investigated included the thickness of each flooded layer, the use of different ratios of snow and sea water, and the carrying out of ice surface preparation between flooding operations on some of the plots.

The properties of the ice sheet were monitored throughout all these studies, by the use of thermistor strings and wire thickness gauges which were installed at the test sites. The thermistor strings provided measurements of the temperature profiles.
through the ice sheet at selected locations. Wire ice thickness gauges were installed to monitor ice thicknesses at each location in order to avoid the requirement to auger holes in the ice cover.

The full scale tank tests were conducted on the Byam Channel ice sheet between 27 March and 18 May, 1974. These tests were designed to evaluate and confirm the ice strength parameters predicted by the laboratory tests and mathematical models. The main objective of the tests was to determine the load bearing capacity of a sea ice sheet having various boundary conditions such as the infinite (continuous) sheet, the semi-infinite (slotted) sheet and the wedge configurations. During an on-ice construction operation, the semi-infinite boundary condition could result from cutting a trench through an infinite ice sheet, or by the intersection of an ice sheet by a natural crack. Similarly, the wedge configuration could result from a natural crack traversing a trench which has been cut in an ice sheet. Tank tests were also conducted to evaluate the long-term bearing capacity of an ice sheet when subjected to sustained loads.

The experimental procedure involved filling 6.1 meter diameter wooden tanks with sea water until the ice sheet collapsed, or until the tank was filled to capacity. Spot measurements of deflections were made with dial gauges and survey levels. In order to detect the formation of the first radial crack, continuous measurements of deflections were made with linear variable differential transducers (LVDT's). Water levels in the tank were measured by float mechanisms and pressure transducers. Other data collected for each test included the ice sheet thickness, the temperature profile, the salinity profile and a record of the crack pattern at the completion of the test.

The second phase of the sea ice strength studies involved extensive cantilever beam testing in the same area as the tank tests. During the program, 89 cantilever tests were carried out both on the natural sea ice and on the built-up right-of-way and experimental plots. Sixty-eight of the cantilever beam tests were short-term tests, the objective of which was to determine the flexural strength and the modulus of elasticity of sea ice for comparison with the values predicted from the small sample test results. The cantilever tests on thickened ice sheets were conducted for the purpose of examining the effect on the ice properties as a result of thickening the ice sheet by flooding.

The other twenty-one cantilever tests were long-term tests which were conducted to determine the creep characteristics and failure criteria for both natural and built-up ice sheets.

The experimental procedure for the cantilever tests involved measuring deflections with dial gauges and LVDT's. Loads for the short-term tests were applied by hydraulic jacks, and loads for the long-term tests were applied by placing ice blocks on the cantilever. Other data collected for each test included the ice sheet thickness, the temperature profile, the salinity profile, the exact dimensions of the cantilever, and the location of the point of failure.

Figure 8 shows a summary of all field and laboratory testing carried out in 1973 and 1974, and Table 1 presents typical comparisons of actual and predicted first crack loads for the tank test program.
### Table 1 – Correlation of Tank Test Results with Computer Analysis

<table>
<thead>
<tr>
<th>TYPE OF TEST</th>
<th>ice thickness (m)</th>
<th>actual first crack load (kN)</th>
<th>load predicted by computer analysis (kN)</th>
<th>ratio of actual to predicted load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infinite sheet</td>
<td>1.58</td>
<td>605</td>
<td>676</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>1.54</td>
<td>636</td>
<td>645</td>
<td>0.99</td>
</tr>
<tr>
<td>Semi-infinite sheet</td>
<td>1.62</td>
<td>312</td>
<td>472</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>1.70</td>
<td>463</td>
<td>659</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>2.04</td>
<td>610</td>
<td>467</td>
<td>1.31</td>
</tr>
<tr>
<td></td>
<td>1.70</td>
<td>356</td>
<td>356</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.79</td>
<td>654</td>
<td>556</td>
<td>1.18</td>
</tr>
<tr>
<td>Wedge</td>
<td>1.65</td>
<td>476</td>
<td>481</td>
<td>0.99</td>
</tr>
<tr>
<td>Trench-end</td>
<td>1.74</td>
<td>516</td>
<td>445</td>
<td>1.16</td>
</tr>
<tr>
<td>Multiple load on semi-infinite sheet</td>
<td>1.75</td>
<td>636</td>
<td>596</td>
<td>1.07</td>
</tr>
</tbody>
</table>

### Analytical Studies

Analytical studies into the load bearing characteristics of sea ice carried out by Polar Gas since 1973 have been directed towards the development of computer programs which will solve bearing capacity problems related to pipeline installation techniques which use the ice sheet as a working platform. The definitions of moment capacity and failure of the ice sheet were developed and these definitions, together with the parameters derived from laboratory and field testing, were utilized in the development of computer programs which can solve short-term and long-term bearing capacity problems.

The moment capacity of an ice section, defined here as the maximum resisting moment that the ice section can develop while allowing limited cracking to occur, is an indication of the overall elastic strength of the section when resisting bending. Since natural ice sheets have decreasing elasticity and tensile strength with depth, the maximum moment capacity at the time that the first crack occurs is controlled by the ice strength some distance above the bottom of the ice sheet. If the ice in the top or bottom of the ice cover is weak due to a high brine volume or poor quality, the moment capacity can therefore be substantially increased by ignoring this layer of weak ice. The determination of maximum moment capacity is accomplished by an iterative procedure, which consists of letting a crack develop at the critical layer (which then becomes ineffective in resisting forces) and allowing the crack to penetrate until it reaches a layer which maximizes the resisting moment. The resisting moment is then the true moment capacity because further cracks will necessarily cause complete failure of the section. A computer program was developed to calculate the maximum moment capacity of an ice section for which the thickness, salinity profile and temperature profile are known. The program was developed from the small sample testing results and was verified by...
the short-term cantilever tests and full scale tank tests.

As a result of the work carried out both in the laboratory and in the field, the failure of an ice sheet under short-term loading was deemed to occur at the time the first crack appeared. Since the collapse load for an infinite or semi-infinite sheet may be as high as 3 or 4 times the first crack load, this definition of failure might appear to be somewhat conservative. However, for a construction operation where areas of ice might undergo several loading cycles, it was believed preferable to avoid the formation of any cracks. Although the analytical studies have not yet been concluded for the determination of the long-term load bearing characteristics of an ice sheet, it is believed that there may be sound practical reasons for defining failure in this instance in terms of deflection rather than in terms of crack formation.

The results obtained from the laboratory studies and field programs described in this paper were utilized in the development of a finite element computer program designated PLATE, with which an accurate estimate of the short-term bearing capacity of an ice sheet can be made. The program may be utilized for a single load or for any configuration of loads and may be applied to an ice sheet with any configuration of edge conditions. Infinite sheets, semi-infinite sheets, wedges, and non-uniform crack configurations can be simulated accurately in the computer analysis. In addition, the salinity and temperature profiles together with the ice sheet thickness (which may vary from element to element) are used as input into the program. The PLATE program has been extensively verified, both by comparison with classical solutions for a loaded plate on an elastic foundation, and by comparison between computer analysis and actual results from full scale tests on sea ice.

Although a creep version of the PLATE program was developed for use in the analysis of long-term loading cases, it became apparent that its complexity would make testing and operation of the program extremely laborious and expensive. Consequently, it was decided to develop a further finite element computer program designated AXI which would analyze axisymmetric cases for both short-term and long-term loading. This program was completed in 1976 and is now undergoing extensive testing. In addition, on-going studies are presently being conducted on laboratory-grown ice sheets in order to develop parameters for the creep portion of the AXI program.

SUMMARY

1. Small sample uniaxial tests were conducted to relate tensile and compressive strength and modulus of elasticity to the brine volume. Experimental curves of an exponential form were fitted to the data.

2. The flexural strength of the sea ice sheet in bending with the bottom fibre of the ice cover placed in tension was analyzed from the results of upward loaded cantilevers, and from tank tests at the formation of the first radial cracks. The results were found to be consistent and in the range predicted from small sample tests.

3. The flexural strength of a sea ice sheet in bending with the top fibre in tension was analyzed from the results of downward loaded cantilevers and apex loaded wedges. The results were found to be highly variable and in general less than predicted from the uniaxial test results. It is believed that the inconsistency of this mode of flexure may be the result of the variable nature of the surface ice, and of natural thermal cracks which exist in the upper part of the ice sheet.
4. The flexural strength of sea ice sheets which had been artificially thickened was found to be enhanced substantially when the upper fibre was placed in tension. This may be due to the fact that surface defects such as cracks and poor quality ice were covered by stronger ice. In this respect, right-of-way preparations were considered to be successful in improving the bearing capacity of an ice sheet.

5. The finite element program PLATE has made possible the estimation of the occurrence of the first crack in an ice sheet subjected to any configuration of short-term loading and for any ice sheet geometry. The program also takes into account variations in ice sheet thickness and the non-homogeneous nature of the sea ice with respect to temperature and salinity.

6. The finite element program AXI, with input of parameters derived from field tests and on-going laboratory studies, will be used in the solution of long-term bearing capacity problems.
FIGURE 1 - POLAR GAS PIPELINE ROUTE

FIGURE 2 - POLAR GAS MARINE CROSSINGS
FIGURE 3 - DITCHING MACHINE CUTTING TRENCH IN ICE SHEET

FIGURE 4 - ICE FLOODING OPERATION
FIGURE 5 - TEST STRIPS THICKENED BY FLOODING

FIGURE 6 - FULL SCALE TANK TEST
FIGURE 7 - LONG-TERM CANTILEVER TEST

<table>
<thead>
<tr>
<th>FIELD</th>
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<tr>
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<td><strong>1974</strong></td>
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<tr>
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<td>ice thickness measurements</td>
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<td>salinity measurements</td>
<td>salinity measurements</td>
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<tr>
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<td>5 full scale load tests</td>
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</tr>
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<td>ice flooding (right-of-way) tests</td>
</tr>
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<td>77 short term tensile tests *</td>
</tr>
<tr>
<td>98 long term tensile tests *</td>
<td>25 short term compressive tests *</td>
</tr>
<tr>
<td>model testing on laboratory grown ice</td>
<td></td>
</tr>
</tbody>
</table>

* tests carried out on ice from Byam Channel

FIGURE 8 - SUMMARY OF LABORATORY AND FIELD TESTING
INTRODUCTION

The Naval Civil Engineering Laboratory (NCEL) in Port Hueneme, California, has conducted several field programs involving tests of large, in situ sea ice beams (Dykins, 1971; Vaudrey, 1975). As a complement to laboratory tests, the field beam tests have the advantage of applying the load to a relatively large sample of ice, avoiding the brine drainage problem and simulating a realistic loading condition in the ice sheet. Since sea ice is, by nature, a non-homogeneous, anisotropic, and viscoplastic material, true flexural strengths and moduli are difficult to determine. Consequently, any beam experiments provide only an index of strength and modulus values. However, these indices can be useful for comparison with laboratory or model basin results and as information regarding ice forces on conical structures and bearing capacity of ice sheets.

Field operations and procedures are described for two different large-scale ice-beam experiments: one designed to produce elastic sea ice response and the other, a creep test series. These sea ice beams were cut from ice sheets, two meters thick, and tested in situ as either cantilevers or simply supported beams. Deflections and applied loads were continuously monitored while recording ice sheet temperature and salinity profiles for correlating data to a brine volume parameter. Field results are compared with those obtained from small flexural and tensile specimen tests performed in the laboratory. Sea ice mechanical properties (e.g., elastic modulus, flexural strengths, time-dependent deformation) found from these tests provide input to bearing capacity calculations leading to safe loading criteria for aircraft and vehicular operations, as well as the description of effects of cargo storage, parked aircraft and snow overburden on ice-sheet behavior.

ELASTIC FIELD TEST PROGRAM

The objective of this test program was to provide data on index flexural strengths and elastic moduli for comparison with previous laboratory flexural and tensile results. These data along with laboratory information were to establish reliable input to sea ice bearing capacity computer codes.
Operational Procedure

Equipment - A site was selected on landfast first-year sea ice in the Chukchi Sea near Barrow, Alaska. Since the ice beams were to be tested in situ their depth would be governed by the present thickness, $h$, of the ice sheet (usually about two meters). Therefore, a special gasoline-hydraulic chain saw was designed and fabricated with a cutter bar 3.7 meters long. The combined saw and power unit weighed 71 kN and was towed from site to site on jackable skis. During operations shown in Figure 1, the saw winched itself along a rail system by means of deadman anchors emplaced in the ice. The actual cut was 4.1 cm wide and a maximum of 3 meters deep.

A loading ram running off the hydraulic power source of the saw was also specially designed and built for this test program. The ram had a push-down capacity of 890 kN with reaction loads supplied through toggle bars secured to the bottom of the ice sheet. The ram in Figure 2 is shown set up for a beam test with the chain saw in the background.

Beam Cutting - To facilitate the beam tests and minimize repeated equipment setups, an ice plug (approximately 1.2 x 1.2 x 2 m) was extracted from the ice sheet. Then a cantilever beam was cut with its free end corresponding to the plug hole. Once the cantilever was tested and failed, the broken remnant was tested as a simple-supported beam. The simple-supported beam was moved into the hole a short distance so the sling supports could be passed around the beam without binding. After both cantilever and simple beams were tested, the procedure was repeated from the other side of the plug hole.

To prevent rapid freezeback a cut was made about two-thirds of the way through the ice sheet, and the ice chips were cleared away before completing the cutting operation. The cutter bar was laterally positioned on its movable table to give a beam width of $\approx 0.6 h$ or a maximum of 120 cm. A beam length, $L > 8 h$, was maintained so that shear deformation could be neglected.

Load Application - Before the beam was completely cut through, two sets of 30-cm diameter holes were drilled through the ice sheet on both sides of the beam to provide access for the reaction force toggle bars. Once freed, the loading ram was quickly positioned at the free end of the beam, and both toggles were anchored securely to the bottom of the ice sheet. A 30-cm line load was hydraulically applied perpendicular to the beam at a ratio of 200-250 kPa/sec, failing the beam in 2-4 seconds. Once the cantilever had failed, the ram was repositioned for mid-point loading, and the ice beam segment was reloaded as a simply-supported beam with a chain and sawhorse system providing reaction support at each end.

Measurements

Temperature/Salinity - A thermocouple station was installed at the test site, providing a temperature profile every 15 cm through the ice sheet. The station was allowed to stabilize before the beam tests began. During testing the thermocouple station was monitored every day. Salinity cores were taken from each beam after failure and cut into 15-cm long segments. Typical ice temperature and salinity profiles are shown in Figure 3.
Load/Deflection - Both a BLH 100 kN load cell and a series of Bourns linear displacement potentiometers continuously recorded load and deflections on a strip-chart recorder. Potentiometers were positioned as close to the line load as physically possible to measure deflections at the load application point (see Figure 4). For the simply-supported beam, the support points were also monitored to determine the amount of support displacement.

Results

Index Flexural Strength - Flexural strengths for in situ sea ice beams were calculated using the elastic beam equations $\sigma_f = 6Pl/bh^2$ and $\sigma_f = 3Pl/2bh^2$ for cantilever and simply-supported beams, respectively. Of course, these formulae give simplistic results to a more complicated problem; therefore, they represent index strengths and not absolute values. A comparison of field beam test results with laboratory data is shown in Figure 5 plotted as a function of the square root of brine volume, $\sqrt{v}$. The laboratory results were obtained from small-scale four-point sea ice beam tests conducted at NCEL. The data are plotted using individual field flexural strengths and mean values for each isothermal laboratory sample group, with a 95% confidence level about its mean. The field beam data points are shown for an averaged temperature and salinity profile of each beam at the time of the test. A straight-line curve fit based on least-squares analysis can be written for the flexural strength, $\sigma_f$

$$\sigma_f = 960 - 1920 \sqrt{v}$$

which agrees quite well with the results compiled by Weeks and Assur (1967). There is no indication that stress concentrations at the root of cantilevers cause them to fail at a lower stress than simple beams (Määttänen, 1975). In fact, several cantilever beams failed up to a meter away from the root, but their strengths were essentially equal to the mean strength of those beams that failed at the fixed end. Thus, potential stress concentrations at the root seemed to have a minimal effect on the index flexural strength.

To compare the temperature-salinity effect on the flexural strength with the effect on the horizontal (perpendicular to crystal growth) tensile ice strength, the best-fit lines of each are plotted in Figure 6. The least-squares curve for the horizontal tensile strength represents the results from over 440 uniaxial tensile tests on laboratory-grown sea (7-9 ppt) and brackish water (1-2 ppt) ice (Dykins, 1970). The two equations given in the figure have almost identical slopes; of course, this similarity should be expected since flexure causes a tensile failure. The indicated ultimate tensile strength of the ice based on beam flexural formula is approximately 150 kN/m$^2$ greater than the observed uniaxial tensile strength (horizontal). This disparity is a common characteristic of brittle materials that do not have the same tension and compression properties, since the neutral axis in the beam specimen shifts toward the stiffer side of the beam. Thus, the fiber stress computed by the flexural formula is less than the true fiber stress on the compression side of the beam and greater than the actual fiber stress on the tension side.
Apparent Elastic Modulus - There is no adequate substitution for a direct method to determine the modulus of elasticity; however, while conducting field beam experiments deflections are easily measured, and an apparent or flexural elastic modulus can be found from beam formulas. For a cantilever loaded at its free end, the deflection, $y_c$, at the load point becomes

$$y_c = \frac{4P}{E_b} \left( \frac{b}{h} \right)^3$$  \hspace{1cm} (2)$$

and for a simple beam with mid-point loading, the deflection, $y_s$, under the load is given by

$$y_s = \frac{P}{4E_b} \left( \frac{b}{h} \right)^3$$  \hspace{1cm} (3)$$

where $P$ is the applied load; $b$, $b$, and $h$ are the beam length, width, and thickness, respectively; and $E$ is the elastic modulus. It should be noted, however, that two factors can influence measured deflections: (1) presence of an elastic foundation; and (2) shear deformation.

The presence of a seawater foundation beneath a cantilever or simply-supported sea ice beam can be accounted for by the following correction ratios:

$$\frac{y'_c}{y_c} = \frac{3}{3 + (\lambda b)}$$  \hspace{1cm} (4)$$

$$\frac{y'_s}{y_s} = \frac{24}{24 + (\lambda b)}$$  \hspace{1cm} (5)$$

where $\lambda$ is the beam characteristic length and $y'_c$ and $y'_s$ are the deflections for a cantilever and simple beam on an elastic foundation. Both factors in equations (4) and (5) come from a power series expansion of the deflection for a beam on an elastic foundation (Hetenyi, 1946).

With a rectangular cross section and assuming Poisson's ratio ($\mu$) of sea ice to be 0.3, the ratio between shear ($\Delta_s$) and bending ($\Delta_b$) deflection at the end of a cantilever and the center of a simple beam is given by

Cantilever: \quad $\frac{\Delta_s}{\Delta_b} = 0.98 \left( \frac{h}{b} \right)^2$  \hspace{1cm} (6)$$

Simple beam: \quad $\frac{\Delta_s}{\Delta_b} = 3.90 \left( \frac{h}{b} \right)^2$  \hspace{1cm} (7)$$

532
For a \( l/h \) ratio of 8.0, the cantilever shear deformation is 1.5\%, while the elastic foundation factor is about 1\%. On the other hand, for the same ratio the simple beam shear deformation is 6.0\%, while the elastic foundation factor is less than 0.5\%.

Since the error would be small in either cantilever or simple beam configuration, the apparent elastic modulus for the field beam tests was calculated from equations (2) and (3). The results appear in Figure 7 as a function of the square root of brine volume along with the mean values of isothermal laboratory sample groups. All laboratory strain data were measured with extensometers bonded to the underside of 5 x 5 x 40 cm simple beams subjected to two-point loading. A straight-line approximation using least squares is calculated for all flexural tests.

\[
E_f = 5.32 \times 10^6 - 1.3 \times 10^7 \sqrt{\nu}
\]  

(8)

where \( E_f \) is the apparent elastic modulus in kPa.

The mean value for the field beams is shown for an average temperature, even though all tests were performed on beams having a temperature gradient. Sea ice properties are temperature-dependent (Schwarz, 1975), but ice sheet profile thermal dependency is accounted for during bearing capacity analysis discussed briefly in the Summary.

**IN SITU CREEP TESTS**

A series of seven large sea ice beams were tested in situ to determine long-term deformation histories as verification of laboratory creep tests and supplemental data for input to bearing capacity analysis techniques.

**Operational Procedure**

**Equipment** - For sea ice creep tests the giant hydraulic chain saw is insufficient since the 4-cm wide cut will refreeze within 3-4 hours. Therefore, a 30-hp ladder-type trencher, cutting a 20-cm wide trench, was used for beam cutting to extend the freezeback time (Vaudrey, 1977). For an initial straight cut of 150-cm the trenching speed was 1.5 m/min, taking about 10 min to initially cut each side of the beams (see Figure 8). Total time for completely freeing a beam was one hour. The chain was outfitted with specially designed conical ice teeth shown in Figure 9 to chip and excavate ice more efficiently.

A 4:1 lever arm was employed to apply the dead load downward with a toggle bar on the bottom of the ice sheet providing the reaction load. The load is applied over a 20 x 20-cm square plate through an adjustable load column. The loading lever arm setup is shown in Figure 10.

**Beam Test Preparation** - Once partial trenches were made, the dry chips were removed, then the sides of the beam were completed. All floating chips were scooped from the flooded trenches. The water had to be clear at the beginning of the test. To prevent freezeback during the 2-3 day test, electrical heat
tape, rated at one watt per centimeter, was floated in the trenches which maintained a surface water temperature between -1° and 0°C. Before applying the load, the beam trenches were covered with plywood strips to provide insulation to reduce the possibility of freezeback.

Measurements

Temperature/Salinity Profiles - Salinity cores were taken from each beam and recorded every 15 cm, giving profiles that fell within the narrow range depicted in Figure 3. Ice sheet temperature profiles were recorded daily at a thermocouple station near the test site; however, these profiles bear little resemblance to the actual temperature changes occurring in a long-term test once the sides of the beam contact seawater. To show how ice beam temperatures have changed after a 48-hr test, a symmetrical plot of isotherm contours are depicted in Figure 11 for one-half of a sea ice beam. No attempt was made during this test series to describe the temperature-dependent effects on beam deformations.

Load/Deflections - While maintaining a constant dead load with a lever arm system, a load cell continuously recorded the actual applied load to monitor potential changes in load due to either wind oscillations of the lever arm or changes in the moment arm length as the load lever passed beyond the horizontal. No significant load deviations (+3%) were noticed during any given test. At specified time intervals, step loads were added to define the boundary between viscoelastic and viscoplastic behavior by correlation of incremental loads with its corresponding incremental deflections. Deflections were continuously monitored with a 15-cm stroke linear potentiometer, positioned at the free end of the beam near the load.

Results

Since there was a significant two-dimensional ice temperature change within the beam cross section, a definite deformation-temperature relationship would be very difficult to establish. Therefore, the usefulness of these tests are limited to a verification of the general viscoelastic constitutive relation developed from isothermal laboratory experiments and an approximate comparison of values obtained for those constitutive equations.

At times, the landfast ice sheet can be slightly upwarped or downwarped so it no longer is in complete equilibrium with the elastic foundation below. Consequently, it was hoped to identify the amount of beam prestressing by measuring the deflection at the moment the cantilever was cut free. All of the beams deflected downward under their own weight, giving an initial tensile prestress in the upper fibers of approximately 90 KN/m². This prestress is based on the assumption that the stress in the ice sheet was negligible at the time the beam was cut.

Strain histories for two typical beam creep tests are shown in Figure 12. Step loads were added to measure corresponding deflections; the figure indicates a viscoplastic response of beam #3 going from σ₁ to σ₂; whereas both step loads (σ₁ → σ₂ and σ₂ → σ₃) for beam #7 show viscoelastic behavior. Elastic foundation effects were considered when calculating strains from measured deflections.
These strain histories generally follow the behavior of a four-parameter fluid (Flügge, 1967), which for beam #7 can be described by a creep function, $J(t)$, of the form

$$J(t) = 3.32(10^{-4}) - 2.51(10^{-4})e^{-0.008t} + 2.1(10^{-7})t$$

for the first 40 hours, at an average overall beam temperature of $-11^\circ C$ and at an average ice surface temperature of $-18^\circ C$. No accurate correlation could be made between time-dependence strains and ice temperatures. The relaxation function, $Y(t)$, the inverse of $J(t)$, can be determined for equation (9),

$$Y(t) = 2.12(10^6)e^{-t/2.9} + 5.92(10^5)e^{-t/1690}$$

where $Y(t)$ represents the time-dependent elastic modulus with the term coefficients given in kPa and the exponential coefficients are relaxation times given in minutes. As a comparison, relaxation functions for laboratory isothermal beams at $-10^\circ C$ and uniaxial tensile specimens at $-20^\circ C$ are given by

$$Y(t) = 1.76(10^6)e^{-t/3.8} + 6.75(10^5)e^{-t/820}$$

$$Y(t) = 2.61(10^6)e^{-t/3.5} + 1.01(10^5)e^{-t/1350}$$

At $Y(0)$, initial modulus values for all three tests lie between $3.5-4.0(10^6)$kPa. Since the upper beam fibers are in tension at $-18^\circ C$, it is reasonable to compare field beam results with laboratory tensile tests at $-20^\circ C$.

**SUMMARY**

These two test programs give index flexural strength and apparent elastic modulus values for sea ice, as well as comparative constitutive relationships defining viscoelastic behavior of sea ice for a specific average ice temperature. These relationships along with laboratory results have been utilized in two mathematical models describing the short-term and long-term bearing capacity of sea ice sheets. These models have been developed into finite element computer codes, both capable of including: (1) temperature-dependency as a function of ice sheet thickness; (2) superposition of circular loads; and (3) support reactions from an elastic foundation (Vaudrey and Katona, 1975; Vaudrey and Katona, 1975). The required ice thickness for landing aircraft (see Figure 13) or supporting ground transportation can be determined, as well as allowable parking times and prediction of ice sheet deformation beneath cargo storage areas and large snow beams.

**ACKNOWLEDGMENT**

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Figure 1. Specially-designed gasoline-hydraulic chain saw cutting ice beam in first-year ice sheet.

Figure 2. Hydraulic loading ram with line-load bar and ratchet mechanisms for anchoring toggle bars to the bottom of the ice sheet.
Figure 3. Typical ice temperature and salinity profiles.

Figure 4. Close-up of loading ram showing load cell cage and linear potentiometer for measuring deflections.
Figure 5. Flexural strength versus the square root of brine volume.

Figure 6. Comparison of flexural and horizontal tensile strength versus the square root of brine volume.
Figure 7. Apparent modulus of elasticity versus the square root of brine volume.

$E_f = 5.23 \times 10^6 - 1.3 \times 10^7 \sqrt{v}$

Figure 8. Davis TF-700 ice trencher cutting a cantilever beam for creep test.
Figure 9. Close-up of trencher boom showing specially-designed conical ice teeth.

Figure 10. Lever-arm dead-load system with load cell cage and reaction anchor for applying creep loading.
Figure 11. Ice beam temperature profile after 48 hours showing influence of seawater on sides of beam.
Figure 12. Creep strain time history for two typical beam tests (average ice surface temperature: -18°C).

Figure 13. Aircraft landing and parking on ice sheet surfaces.
1. INTRODUCTION

The deepest pressure ridges in an icefield are of particular interest to engineers and glaciologists. When an icefield is advected into shallow water the deepest keels are the first to take the ground and so determine the extent of sea bottom scouring. Oil drilling platforms must be built to withstand the impact of the largest ridges that are expected to exist in their area, and the design of polar icebreakers requires a knowledge of the strengths and sizes of the largest ridges that occur at a high enough frequency to be unavoidable. In the Arctic Ocean the deepest keels add greatly to the form drag of the ice cover, and may cause wave drag by generating internal waves (Hunkins, 1974). Kinematic models simulating ridge growth have been developed (Parmenter and Coon, 1972) in which a strip of rubble (formed by the collapse of thin ice within a lead) is allowed to be crushed between two converging ice sheets. The unsymmetrical piling of the rubble above and below the sheets causes progressive bending failure of the sheets, and the broken blocks add to the rubble to build the ridge. An examination of deep keel shapes and maximum drafts provides the data to test such kinematic models. The present study is an analysis of all keels of 30 m draft or greater found during a 3900 km sonar profile of the Arctic Ocean.

The profile was obtained in October 1976 during a co-operative experiment involving the British nuclear submarine HMS SOVEREIGN and an Argus aircraft of Maritime Command, Canadian Forces. The submarine recorded an upward-looking sonar profile while the aircraft flew along an identical track during the same period using a laser profilometer to record ice surface elevation (Wadhams, 1977a; Wadhams and Lowry, 1977). The main aim of the experiment was to derive statistical relationships between the surface and bottom topography of the ice by comparing corresponding sections of sonar and laser profile (Wadhams, 1977b). A causal point-to-point comparison of the profiles is not possible because of mutual navigation errors. The sonar profile was recorded on Teledeltos paper using a Kelvin Hughes MS45 echo sounder feeding a 45 kHz transducer with overall beamwidths to the 3 dB points of 17° fore-and-aft and 5° athwartships. The chart record scales were 2 m: 10 mm in the vertical plane and 1 minute: 75 mm in the horizontal plane. This gave considerably better resolution than the recorders used in earlier profiling experiments (Lyon, 1961; Swithinbank, 1972), especially when the submarine cruised slowly. Fig.1 shows a keel of 35.7 m draft and width at least 600 m recorded at a speed of 7 knots (3.6 m s⁻¹). The effect of the fore-and-aft beamwidth is seen in the individual hyperbolae surrounding strong reflecting points on the ridge, but in general the shape of the keel is well reproduced. The sonar profile is thus a
valuable data source on individual under-ice features.

Independent keels exceeding 30 m in draft were identified using a transparent overlay and a Rayleigh selection criterion (Williams et al., 1975) whereby a keel is deemed independent if the troughs on either side of the crest regress at least half way towards the local draft of level ice. Sea level could be estimated to $\pm$ 20 cm by interpolation between the nearest open cracks on either side of the keel, since the submarine's depthkeeping was excellent and open water revealed itself clearly by giving multiple sonar echoes. Each keel was then traced from the chart record, the ridge being considered to exist as an entity until either

a) the ice draft dropped below 5 m, or
b) the draft reached a minimum from which it began to rise to form a new independent keel.

2. THE DISTRIBUTION OF KEEL DRAFTS

45 independent deep keels were found in 3907 km of track, an average of one keel per 87 km. The distribution of drafts was as follows:–

<table>
<thead>
<tr>
<th>Draft Range</th>
<th>Number of Keels</th>
</tr>
</thead>
<tbody>
<tr>
<td>30-32 m</td>
<td>24 keels</td>
</tr>
<tr>
<td>32-34 m</td>
<td>11 keels</td>
</tr>
<tr>
<td>34-36 m</td>
<td>4 keels</td>
</tr>
<tr>
<td>36-38 m</td>
<td>4 keels</td>
</tr>
<tr>
<td>38-40 m</td>
<td>1 keel</td>
</tr>
<tr>
<td>40 m+</td>
<td>1 keel</td>
</tr>
</tbody>
</table>

The deepest keel (no.39 in fig.3) was the only one whose draft exceeded the scale limit of the recorder (40 m) so that its peak was lost. From extrapolation of the slopes it is conservatively estimated to have a draft of 42-43 m. The deepest keel yet recorded in the Arctic Ocean had a draft of 47 m (Lyon, quoted by Weeks et al., 1971).

Hibler et al. (1972) proposed a theoretical ridge height distribution which has been extensively tested against observation. The probability density $P(h)$ was obtained by finding the most likely distribution of ridges that will yield a given volume of deformed ice assuming that all ridges are geometrically congruent. It is given by

$$P(h) \, dh = 2 \, h \, \exp(kh_0^2) \exp(-kh_0^2) \, dh$$

(1)

where $h$ is the mean draft, $h_0$ is a low value cutoff and $k$ is a parameter which must be derived iteratively from the relationship

$$\exp(-kh_0^2) = \frac{h}{(k\pi)^{1/2}} \text{erfc}(k^{1/2}h_0)$$

(2)

To estimate the fit of the deepest keels to (1) it was necessary to measure over a much larger range of depths, so the transparent overlay technique was used to find the numbers of independent keels with drafts greater than 10 m in 5 m intervals. 8367 ridges were found (2.14 per km) and fig.2 shows the observed and theoretical distributions plotted on a log-lin histogram. The parameters were: $k = 0.006$, $h_0 = 10$, $h = 14.45$. There is an excellent fit between theory and observation for all the moderate depth classes, showing the essential validity of Hibler's theory. However, at drafts exceeding 30 m the observed numbers of keels definitely exceed the theoretical predictions. The numbers are:

<table>
<thead>
<tr>
<th>Draft Range</th>
<th>Observed</th>
<th>Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>30-35 m</td>
<td>37</td>
<td>27.3</td>
</tr>
<tr>
<td>35-40 m</td>
<td>7</td>
<td>3.5</td>
</tr>
<tr>
<td>40 m+</td>
<td>1</td>
<td>0.3</td>
</tr>
</tbody>
</table>

It is noteworthy that existing tests of (1) against observation (Hibler et al., 1972; Hibler, 1975) used short lengths of track in which the higher depth classes were pooled together in order to attain a statistically reasonable number of keels. Any deviation from theory at the extreme tail of the distribution therefore would not be
detected. The present profile is the longest to be tested against theory, so until further evidence is available we must assume that there is a significant positive deviation of observation from theory for ice keels beyond 30 m in draft. Thus the engineer who wishes to use (1) to estimate the frequency of very deep keels in an icefield by extrapolation from limited data must exercise caution, and for safety's sake should increase his estimates of deep keel frequencies by 50-100%.

3. THE GEOGRAPHICAL DISTRIBUTION OF DEEP KEELS

Fig. 3 shows all 45 deep keels plotted along the submarine's track chart. The geographical distribution reflects the division of the Arctic into ridging zones proposed by Weeks et al. (1971) and analysed, for the SOVEREIGN data, by Wadhams and Lowry (1977). Weeks et al. identified an "offshore zone" of very heavy ridging stretching some 250 km from the north coasts of Greenland and the Canadian Archipelago into the Arctic Ocean. This is followed by the "central Arctic" zone of milder ridging. Wadhams and Lowry found heavy ridging along the submarine's westward track from 82°N to 85°N 70°W, with the most intense ridging being near this turning point and for the first 200 km of the northward leg towards the Pole. At 87°N there is an abrupt transition to milder ridging which prevails until the Greenland coast is approached again. The distribution of the deepest keels follows the same pattern. Within the central Arctic zone (from 87°N 70°W northward to the Pole and down to the position of ridge 40) there are only 5 deep ridges in 2000 km of track, whereas in the offshore zone (from 82°N west to 70°W and north to 87°N 70°W) there are 39 deep ridges in 1050 km. The lightly ridged area is the Trans Polar Drift Stream, which carries mainly first- and second-year ice across the Eurasian Basin from Siberia down towards the East Greenland Current. The heavy ridging occurs when the Drift Stream's approach to the coast of Greenland causes convergence in the ice cover and consequent deformation. At 70°W the Drift Stream has given way to the Beaufort Gyre, containing mainly older ice, and the deformation of this older ice against the coast produces the especially intense ridging in that area.

There is also a statistically significant clustering effect in the geographical distribution. This is seen especially in ridges 1 and 40-44; 14-20; 22-25; 26-30; and 31-34. The clusters in the 70°W region are shown more clearly in fig. 4. Ridges 14-20, for instance, are all contained within a triangle of side 16 km, while 26-28 are spaced only 1 km apart. The first possibility is that each cluster represents a single ridge sampled at several points; such a ridge would have a very sinuous shape. It is quite possible that a closely spaced pair can be explained in this way, and as an overall explanation it cannot be rejected in the absence of two-dimensional information on ridge orientations. However, it is more likely that a cluster of very deep keels is a relic of a single past event of intense convergence in which one or more long, deep keels were formed. In the subsequent deformation of the ice cover the long ridges were split into independent linkages which drifted and revolved relative to one another while remaining in reasonably close company. In practical terms, e.g. to an icebreaker captain, the clustering effect implies that if a very deep ridge is encountered there are likely to be others not far away.

4. THE SHAPES OF DEEP KEELS

(i). Observed profiles

Many careful measurements have been made of the subsurface shapes of pressure ridges, using drilling techniques or sonar profiling from the side (e.g. Wittmann and Schule, 1966; Weeks et al, 1971; Kovacs, 1972; Kovacs et al, 1973). All authors agree that no
simple geometrical model is adequate to describe all ridges, and that even the concept of "slope angle" cannot be applied to some multi-year ridges with bowl-shaped or semi-elliptical keels (Kovacs et al., 1973). Measurements made from the ice sheet have the advantage that a true orthogonal cross-section of the ridge can be obtained. In the SOVEREIGN profiles each ridge has an unknown orientation so that the observed slope angle is always less than the true slope; the sonar beamwidth reduces the observed slope still further. Nevertheless, the number of ridges in the sample permits some tentative statistical conclusions to be drawn.

Fig. 5 shows a representative selection of keel profiles, drawn without vertical exaggeration but on various scales. The first three ridges (10, 15 and 19), drawn on the largest scale, have simple triangular topography. The slopes are virtually constant, and the apparent flattening of the crest can be largely explained by the effect of sonar beamwidth (eqn. 5). Ridges 1 and 29 show more distinct flattening of the crest. The experiment was carried out in mid-October so that most, if not all, of the ridges have experienced at least one summer melt period and hence qualify as multi-year ridges. They are extremely strong on account of the refreezing of summer meltwater into the voids which previously existed between rubble blocks. Their crests have become rounded by summer melting and by erosion due to shear currents, which proceeds more rapidly in summer and which can transport entire ice blocks (Rigby and Hanson, 1976). Ridges 13, 27 and 39 have steep initial slopes leading to gently rounded crests. This may be the result of erosion of the crest, or it may be an example of a process described by Parmerter and Coon (1972), who postulated a maximum keel draft which is a function of the strength of the ice, the density of the rubble blocks in the ridge and the thickness of the ice sheet. Once a keel has reached its maximum draft further compression leads to a lateral growth into a wide, flat-bottomed hummock. Ridge 43 appears a well-developed example of this. Ridge 39, the deepest keel, is assumed to have a similar shape—otherwise its draft may be even greater than 43 m. Ridges 21, 25, 28 and 30 have been traversed at an acute angle so that the sonar is sampling along the keel crest rather than across it. The ultimate case is ridge 6 (see also fig. 1), where over 600 m of the keel have been profiled.

From each ridge profile a pair of slope angles was estimated. Fig. 5 shows that this can be a difficult undertaking, and a definite procedure was used. The region within ± 10 m range of the crest was ignored as possibly subject to sonar distortion. The remainder of the half-ridge was sampled as far as the point at which the slope changed radially—usually becoming more gentle but sometimes (e.g. ridge 27) becoming more steep. A linear regression was made on equally spaced points along this part of the keel—thus the slope is measured from the deeper part of the keel profile. 88 values were obtained (one keel was impossible to estimate) and their distribution is shown in fig. 6. Slopes vary from almost zero to 51°, but with a concentration around 16-28°. The mean slope is 23.9°.

Another estimate of mean slope was made by drawing a "composite ridge" (fig. 7), an amalgam of all ridges, in which depth points, normalised as fractions of the keel draft, were taken from the keel tracings and averaged in bins of width 4 m. A is the two-sided composite drawn out to a range of 80 m, beyond which the shallower keels cause the composite ridge to become almost flat. Since there is no reason to expect any left-right bias (assuming that the centre of the sonar beam was vertical), ridge A was itself averaged into a one-sided composite ridge B. This ridge consists of a flattened crest (probably an artefact); a steep initial slope (a) of angle 26.8° extending from 6 to 16 m from the crest; and a gentler slope (b) of angle 17.7° extending out to about 50 m. The angles were calculated by taking the depth of the composite keel as the mean draft of all the keels.
(ii). Theoretical slope angle distribution

Several geometrical shapes have been suggested as approximations to a typical keel profile. The simplest is the isosceles triangle proposed by Makarov in 1901 (Zubov, 1945) to describe new ridges; as the ridge ages the shape progresses by erosion and melting into a semi-ellipse. Intermediate between these extremes is the Wittmann and Schule (1966) model of an isosceles triangle with a rounded crest.

Figures 6 and 7 show slopes which are biased by the angle at which the ridge is crossed and by the effect of sonar beamwidth. We can estimate a true slope distribution from the data of fig.6 provided we make two assumptions:

(a) that keels have random orientations—implying that keels oriented at right angles to the submarine's track will be sampled relatively more frequently than keels lying nearly parallel to the track;

(b) that a true cross-section through all keels will give the same geometrical shape. The simplest shape to choose is the Makarov model of an isosceles triangle with slope angle $\alpha$.

If the submarine's track is oriented at an angle $\theta$ to a true cross-section, i.e. at $(\pi/2 - \theta)$ to the line of the keel (fig.8), the slope angle $\beta$ encountered by the submarine is given by

$$\tan \beta = \cos \theta \tan \alpha$$  \hspace{1cm} (3)

The beamwidth of the transducer further distorts the slope. For simplicity we shall ignore the small athwartships beamwidth, and assume that SOVEREIGN's sonar had only a fore-and-aft beamwidth of half-angle $\lambda$ ($\lambda = 8.5^\circ$). The observed profile generated by the first return from each sound pulse is shown in fig.9. Here $D$ is the depth of the submarine below the ambient level ice draft ($D = 72$ m) and $h$ is the depth of the keel below the same datum. The transit time of a sound pulse is neglected. The recorded profile gives the keel a greater apparent width, a reduced slope angle $\gamma$ and a rounded crest, although the absolute draft of the crest is correctly recorded. The additional half-width $x_o$ of the keel is given by

$$x_o = D \left[ \sin \lambda - \cot \beta (1 - \cos \lambda) \right]$$  \hspace{1cm} (4)

and the lateral range $x_1$ to which the apparent rounding of the crest extends is given by

$$x_1 = (D - h) \tan \lambda$$  \hspace{1cm} (5)

which is about 5-6 m for $h = 30-40$ m. The apparent slope angle $\gamma$ is given by

$$\tan \gamma = \frac{\sin \beta}{\cos (\beta - \lambda)} \quad \beta > \lambda$$  \hspace{1cm} (6)

or

$$\tan \gamma = \sin \beta \quad \beta \leq \lambda$$  \hspace{1cm} (7)

since when $\beta \leq \lambda$ the first return from a pulse is a specular reflection from the ridge slope, whereas when $\beta > \lambda$ the first return comes from the leading edge of the beam footprint.

Under our assumption (a) the probability density function of $\theta$ is given by

$$P(\theta) d\theta = \cos \theta \, d\theta$$  \hspace{1cm} (8)

Equations (3), (6) and (7) yield the transformations from $\theta$ to $\gamma$:-

$$\cot \gamma = \cos \lambda \sec \theta \cot \alpha + \sin \lambda \quad \gamma > \gamma_0$$  \hspace{1cm} (9)

$$\cot \gamma = \sec \theta \cot \alpha \sqrt{1 + \cos^2 \theta \tan^2 \alpha} \quad \gamma \leq \gamma_0$$  \hspace{1cm} (10)
where \( \gamma_0 \), the angle at which \( \beta = \lambda \), is given by
\[
\gamma_0 = \tan^{-1}(\sin \lambda) \tag{11}
\]
Thus the probability density function for \( \gamma \) is
\[
P(\gamma) \, d\gamma = P(\theta) \left| \frac{d\theta}{d\gamma} \right| d\gamma = \frac{\csc^2 \gamma}{(\cot \gamma - \sin \lambda)} \frac{f^2}{(1 - f^2)^{\frac{1}{2}}} d\gamma \quad \gamma > \gamma_0 \tag{12}
\]
or
\[
P(\gamma) \, d\gamma = \frac{\cot^2 \alpha \tan 2\gamma}{2 \left[ \cos 2\gamma (\cos 2\gamma - \sin^2 \gamma \cot^2 \alpha) \right]} d\gamma \quad \gamma \leq \gamma_0 \tag{13}
\]
where
\[
f = \frac{\cos \lambda \cot \alpha}{(\cot \gamma - \sin \lambda)} \tag{14}
\]
P(\( \gamma \)) is the function which is sampled in fig.6 provided \( \alpha \) takes only one value. The mean apparent slope angle \( \bar{\gamma} \) is given by
\[
\bar{\gamma} = \int_{0}^{\gamma_{\text{max}}} \gamma \, P(\gamma) \, d\gamma \tag{15}
\]
where \( \gamma_{\text{max}} \), the maximum apparent slope angle occurring when \( \theta = 0 \), is given by
\[
\gamma_{\text{max}} = \tan^{-1}\left( \frac{\sin \alpha}{\cos (\alpha - \lambda)} \right) \tag{16}
\]
from (6).

Equations (12) to (15) were solved numerically, and a typical curve of \( P(\gamma) \) is shown in fig.10, using \( \alpha = 33^\circ \), the average slope angle for first-year keels found by Weeks et al (1971) and Kovacs (1972). This agrees closely with an average value of \( 32^\circ \) given by Wittmann and Schule (1966). The probability increases rapidly near \( \gamma_{\text{max}} \) (= \( 30.9^\circ \)), giving \( \bar{\gamma} = 27.0^\circ \). The shape of the curve does not agree with fig.6, showing that a single value for \( \alpha \) is inadequate to describe the distribution of apparent slopes. It is clear that fig.6 is actually a convolution of \( P(\gamma) \) with \( P(\alpha) \), the probability density function for \( \alpha \). To evaluate \( P(\alpha) \) we must deconvolve fig.6. This was done numerically as follows. The bin with the highest slope angles (48-52\(^\circ\)) was emptied by postulating a number \( n(\alpha) \) of ridges with \( \alpha \) such that \( \gamma_{\text{max}} = 52^\circ \) and with \( n \) sufficient to produce the required number of ridges with apparent slopes of 48-52\(^\circ\) using the results of (12)-(15). The contributions made by \( n \) to all the other bins were then calculated. The process was then repeated for the next lowest bin (44-48\(^\circ\)), having made allowance for the number already described by the first \( n(\alpha) \). Thus all bins were emptied successively in descending order. The distribution of \( n(\alpha) \) has been added to fig.6; this is now the distribution of true ridge slopes. The slope angles vary from \( 8^\circ \) to \( 56^\circ \); the average slope angle is \( 32.1^\circ \). This happens to be in excellent agreement with the average values quoted by other researchers.

The agreement is pleasing but possibly fortuitous, since the deconvolution process involves a large step size. Having shown that this type of calculation is feasible, we now plan to apply it to the whole digitised SOVEREIGN profile, carrying out automatic slope analyses on keels of lower draft than 30 m. The vastly greater numbers of such ridges will increase the reliability of the probabilistic technique described above.

5. SUMMARY

An examination has been carried out of 45 pressure ridge keels with draft exceeding 30 m found during a submarine transit of the Arctic Ocean. This is the largest number of keels yet studied in this way. A probabilistic technique has been developed
to derive the true distribution of slope angles from the observed distribution, and the average slope angle was found to be 32.1°, in close agreement with averages found by experimenters who have worked on ridges (all shallower than 30 m) in situ. Thus there is no evidence for a significant difference in average slope between deep keels and shallow keels. The deep keels vary greatly in shape, progressing from a simple triangle to flattened bowls and more complex shapes. Their geographical distribution is similar to that of shallower ridges, but with a greater relative difference in ridge frequency between heavily and lightly ridged areas. There is a distinct clustering effect, suggesting that a group of deep keels spreads from a single origin. There are significantly more deep keels than predicted by the theory of Hibler et al. (1972); the deepest observed keel had a draft exceeding 42 m.

ACKNOWLEDGEMENTS

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REFERENCES


FIG. 1 (above) Sonar trace of a deep Arctic pressure ridge keel (keel no. 6; 84°40'N 35°43'W).

FIG. 2 (right) Comparison of observed and theoretical keel frequencies.
FIG. 3
Keels exceeding 30 m in draft plotted on track of SOVEREIGN

FIG. 4
Deep keels in vicinity of 70°W
FIG. 5 Representative profiles of deep keels. Range is measured from point of greatest draft.
FIG. 6 Observed distribution of slope angles. Dashed line is inferred distribution of true slopes.

FIG. 10 (above)
The function $P(\gamma)$ for
$\alpha = 33^\circ$. $\gamma_{\text{max}} = 30.9^\circ$
$\bar{\gamma} = 27.0^\circ$.

FIG. 7 "Composite ridge" obtained by averaging all the normalized ridge profiles. B is average of two sides of A.
FIG. 8 Geometry of a triangular keel of slope $\alpha$ cut by a submarine at a slant angle.

FIG. 9 The apparent profile of a triangular keel recorded by a sonar with overall beamwidth $2\lambda$. 
STUDY OF SEA ICE USING IMPULSE RADAR

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INTRODUCTION

Although sea ice presents one of the most serious obstacles to ocean engineering under Arctic conditions, techniques to measure and monitor its properties, such as thickness and strength, are still being developed. This ability is particularly important since the properties of sea ice vary dramatically both spatially and temporally.

Impulse radar has been used by several researchers to study sea ice and icebergs (Campbell and Orange, 1974a,b; Goodman et al, 1977; Kovacs, 1977a,b; Rossiter and Gustajtis, 1977). The technique is being used routinely in the Arctic to measure ice thickness variations, although the electrical properties of the ice must be known in order to obtain an absolute thickness estimate.

This paper presents results from studies made of first year fast ice near Newfoundland. Particular attention is given to estimating the electrical properties of ice using impulse radar. The complex reflection coefficient from the ice surface is determined using Fourier techniques, and this approach is a promising first step toward quick remote estimates of the electrical properties of sea ice in situ. These properties are significant, since both the electrical and the mechanical properties of sea ice are dominated by the influence of its brine volume fraction.

IMPULSE RADAR SYSTEM

The impulse radar method consists of transmitting a short pulse of radio-frequency energy. The pulse is partially reflected from the surface, and, if the material has low enough electrical losses, from subsurface interfaces. Impulse radar varies from more conventional types of radar in that instead of transmitting many cycles of a particular frequency, a single, broad-band pulse is transmitted.

The advantages of impulse radar are that relatively low frequencies can be used, giving the ability of penetrating lossy materials (like sea ice), while retaining adequate time resolution for accurate depth estimates. The main disadvantage is that it is not possible to focus the antenna radiation pattern, which is similar to that of a half-wave dipole. Therefore it is necessary to make measurements on or near the surface in order to achieve reasonable spatial resolution and to receive a strong enough return signal.

The impulse radar system used was built by Geophysical Survey Systems, Incorporated, (GSSI). The system generates a pulse every 20 microseconds, and many pulses are
sampled sequentially to make up one audio-frequency trace. The transmitted pulse width is controlled primarily by the antenna size. Two sets of antennas were used, with centre-frequencies of 80 MHz and 400 MHz, although only the higher frequency results are discussed here. The received signals were plotted on a graphic recorder, and recorded on magnetic tape for later processing.

An individual data trace consists of a start-of-scan pulse, followed by the transmitted pulse. Reflections are received at greater delay times, and are amplified by a time-gain amplifier. The window during which echoes are received can be varied on the control unit from about 25 ns to 6 μs. After the sampling procedure 3.2 to 51.2 scans per second can be recorded on tape.

Measurements were made both on the ice and from the air. For surface measurements the equipment was mounted in a sled, and the antennas were towed behind (see Figure 1). For aerial work the system was placed in the passenger compartment of a Bell 204 helicopter, and powered directly from the aircraft. The antennas were slung in a small net 4 to 6 m beneath the helicopter. Data were collected at heights of 5 to 20 m above the surface.

RESULTS

Field Location

Measurements were made in Notre Dame Bay, near Twillingate, on the northeast coast of Newfoundland. Measurements were made at various sites, over first-year fast ice. Comparison tests were also made over frozen fresh-water ponds in the area. Measurements were made during February and March, 1977. A number of studies were conducted at this site simultaneously with the impulse radar experiments. These included extensive physical property measurements of ice core samples; wave-induced strain studies (Squire and Allan, 1977); and remote sensing imaging radar and aerial photography (Worsfold et al, 1977).

The fast ice in the area was typically 30 to 70 cm thick, had salinities of 2 to 6 parts per thousand, densities of 0.86 to 0.89 Mg m\(^{-3}\), and was within -2 to -4°C. The surface was very smooth, and no snow cover was present. The ice was polycrystalline, and little or no columnar crystal structure was observed, even near the ice bottom.

Thickness Profiling

Impulse radar profiles of the ice taken in Manuel's Cove showed remarkable variation from point to point. Near the shore the ice was essentially transparent, so that strong reflections were obtained from the ice bottom. About 200 m from the shore, pieces of transparent ice 1 to 5 m across alternated with sections of opaque ice (Figure 2). Further from shore the opaque ice dominated, so that beyond 500 m from the shore no ice bottom reflections could be detected. These features could be observed both from the surface and from the air, using the 400 MHz antennas.

There was no surficial expression indicating the differing ice types, although the physical properties of the two were dissimilar (Table 1). Although the salinity of the transparent ice was lower than that of the opaque ice, this difference does not fully explain the large variation in radar responses. It is postulated that the porous, polycrystalline bottom surface of the opaque ice admitted significantly more brine into that ice from below through its drainage channels, so that it had a higher brine content, and hence greater conductivity. It is possible the transparent
Table I. Physical Properties of Ice Samples, Manuel's Cove

<table>
<thead>
<tr>
<th></th>
<th>Transparent Ice</th>
<th>Opaque Ice</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (cm)</td>
<td>41±1</td>
<td>60±10</td>
</tr>
<tr>
<td>Density (Mg m⁻³)</td>
<td>0.88±0.02</td>
<td>0.87±0.02</td>
</tr>
<tr>
<td>Salinity (‰)</td>
<td>1.3 - 2.2</td>
<td>1.2 - 3.7</td>
</tr>
<tr>
<td>Bottom Surface (from cores)</td>
<td>Smooth; large columnar crystals</td>
<td>Slushy; polycrystalline</td>
</tr>
<tr>
<td>Dielectric Constant</td>
<td>4.5±0.5</td>
<td>&gt;5?</td>
</tr>
</tbody>
</table>

Ice froze from lower salinity water (a stream runs into Manuel's Cove). After freezing the two types of ice could have been broken into pieces by wave action, the pieces blown together, and refrozen.

Using the relationship:

\[ \varepsilon_r = (ct/2d)^2 \]  

where \( \varepsilon_r \) is the relative dielectric constant of the ice, 
\( c \) is the speed of electromagnetic waves in free-space \( (3 \times 10^8 \text{ m s}^{-1}) \), 
\( t \) is the arrival time of the reflection \( (\text{s}) \), and 
\( d \) is the ice thickness \( (\text{m}) \),

the dielectric constant of the transparent ice was estimated to be 4.5±0.5. This value is in reasonable agreement with values for low salinity first-year Arctic ice measured in the laboratory by Vant (1976), at similar frequencies and temperatures. Consistent values of the dielectric constant were also estimated by separating the transmitter and receiver and making a second run (see Kovacs, 1977a). This technique does not require an independent thickness measurement, but the traverses must be made on the surface (i.e. not from a helicopter).

Surface Reflection Coefficient

In order to try to estimate the dielectric properties of the opaque ice, which was the predominant type of sea ice in the survey area, the antennas were raised off the surface of the ice, and the surface reflection was examined in detail. It can be shown that for a homogeneous, non-magnetic material,

\[ \varepsilon^* = \frac{(1-R^*)^2}{1+R^*} \]  

where \( \varepsilon^* \) is the complex relative permittivity of the material, and 
\( R^* \) is the complex reflection coefficient of a plane wave normally incident onto a plane surface of the material.

\[ \varepsilon^* = \varepsilon_r + i \varepsilon_r \tan \delta \]  

where \( \varepsilon_r \) is the relative dielectric constant, 
\( \tan \delta \) is the loss tangent, and 
\( i = \sqrt{-1} \).
Therefore, in order to estimate $\varepsilon_r^*$, the complex permittivity for ice, we need to obtain $R_1^*$. This was done by comparing the reflection from the ice to that from a sheet of aluminum screening laid on the ice surface, and assuming that the screen was a perfect reflector. Then,

\[ R_1^* = \left( \frac{R_i^*}{R_s^*} \right) R_s^* \]

\[ = \left| \frac{R_i^*}{R_s^*} \right| \exp \left[ i(\varphi_i - \varphi_s) \right] R_s^* \]

(4)

where $R_i^*$ and $R_s^*$ are the complex reflection coefficients for ice and screen respectively, $|R_i^*|$ and $|R_s^*|$ are their respective moduli, and $\varphi_i$ and $\varphi_s$ are their respective phases.

If the screen is a perfect reflector,

\[ R_s^* = -1. \]

We can measure

\[ \rho = \left| \frac{R_i^*}{R_s^*} \right| \quad \text{and} \quad \Delta \varphi = \varphi_i - \varphi_s. \]

(6)

Hence,

\[ R_i^* = -\rho \exp \left[ i\Delta \varphi \right], \]

(7)

which can be substituted into equation (2) to find $\varepsilon_r^*$. Values of $\varepsilon_r$ and $\tan \delta$ are plotted for various values of relative amplitude, $\rho$, and phase, $(\Delta \varphi - 180^\circ)$, in Figures 3 and 4, respectively.

Amplitude and phase values were obtained for reflections from the sea ice using the 400 MHz antennas. The experimental procedure was to hold the transmitting and receiving antennas, which were mounted side by side, at various heights up to 2 m above the ice, then repeat the measurements over a sheet of fine-mesh aluminum screening about 1 m square. The traces are shown in Figure 5. Each trace shown is an average of about 15 scans to reduce random noise, although systematic noise and ringing are preserved.

Data processing was done on a Hewlett-Packard 5451B Fourier Analyzer system. Each of the traces shown in Figure 5 was shifted so that the surface reflection was at the beginning of the trace, and a 128-point Fourier Transform was performed. The resulting amplitude spectra are shown in Figure 6(a). It can be seen that the bulk of the energy is within the frequency range of 200 to 600 MHz (data collection and sampling rates were sufficiently high that frequencies of up to 2.5 GHz would be retained).

The ratio of the two amplitude spectra was taken to obtain the relative amplitude spectrum shown in Figure 6(b). The values vary rather widely ($0.6 \pm 0.2$) and the significance of the frequency dependence suggested by the curve is not yet certain. The phase difference, $\Delta \varphi$, is even more variable from frequency to frequency ($-20^\circ$ to $-90^\circ$), but is consistently negative. This phase lag can be seen by careful inspection of Figure 5.

Because the phase difference is negative, the electrical properties cannot be estimated assuming a single layer (Figures 3 and 4), for which a positive phase differ-
ence is required. The phase lag is too large and too consistent to be explained by measurement errors in antenna height above the surface. This phase lag was not observed in comparative tests over fresh-water ice, for which a dielectric constant of 3.2 ± 0.2 was obtained using this method, in complete agreement with known values. Therefore, it is postulated that the near-surface of the sea ice (top 10 cm) was layered, and that the calculation of the reflection coefficient must include contributions from two or more layers. This explanation is physically realistic, since there had previously been a snow cover on the surface, which had melted and refrozen.

Work is now in progress to model the reflection coefficient from a n-layer dielectric medium (see Wait, 1958). Of course any inversion of the data to obtain the properties will no longer be unique, since there will be three additional parameters to be estimated for each added layer. However, it may be possible to put reasonable bounds on the properties. It will also be possible to explicitly model a snow layer over ice.

DISCUSSION

Sea ice is a complex, highly variable mixture of ice and brine. These two constituents have widely different electrical properties -- the former is very resistive and has a dielectric constant of 3.2; the latter is very conductive, and has a dielectric constant near 80. Hence the electrical properties of the mixture depend largely on the brine volume (Vant, 1976). The brine volume is also highly important to the mechanical properties of sea ice (Assur, 1958). Therefore, if even crude estimates of the electrical properties of sea ice could be determined quickly, in situ or remotely, they could give an important indication of the strength of the ice.

It has been shown in the results presented that the impulse radar return signal from ice shows large variations within the same region. Where the ice was transparent enough to obtain a bottom reflection, the dielectric constant indicated is in agreement with previous results. Where the ice was opaque, it is assumed that the electrical losses were significantly greater, and that therefore the brine volume was larger, and the ice structurally weaker. Attempts to estimate the electrical properties of the lossy ice from the surface reflection coefficient showed that phase measurements are essential. It is clear that considerable information is contained in the actual shape of the impulse radar signal, and this information can be brought out by digital processing of individual radar traces.

Whether it will be practical to make surface reflection estimates from an airborne platform is still uncertain. In order to obtain reasonably accurate electrical properties, the phase of the return signal needs to be referenced (for example, to a reflection from open water), and known to an accuracy of about ±10°. Hence the height variations of the antenna above the surface must be monitored to ±10 cm at 80 MHz.

CONCLUSIONS

The impulse radar response from sea ice shows dramatic variations indicating varying ice types, even when there is no surficial evidence of these changes. The measurements can be made from the surface (under favourable conditions), or from the air, using a low-flying helicopter. If the ice is transparent, ice thickness variation profiles can be made. If the ice is opaque, it is suggested that its electrical losses are greater, and that the ice may be mechanically weaker. Hence the nature of the impulse radar response could be used to categorize sea ice, and impulse radar could be included in many sea ice observation and remote sensing applications.
The complex reflection coefficient from the ice surface can be determined from impulse radar measurements made above the surface. In the area surveyed the reflection coefficient suggested near-surface layering in the ice. However, this approach shows promise in the future for making remote estimates of the electrical properties of sea ice, in particular for distinguishing first year, multi-year, and fresh water ice.

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REFERENCES


Figure 1. Functional sketch of GSSI impulse radar system for use on ice surface. For helicopter studies the system was mounted in the passenger compartment and the antennas were slung beneath the aircraft.
Figure 2. Impulse radar data collected on ice surface (Manuel's Cove). Dark bands indicate strong signals. Upper bands are transmitted signal. Lower patches are reflections from bottom of ice. Note absence of bottom reflection in centre of figure.
Figure 3. Calculated values of relative dielectric constant, as a function of relative amplitude and phase of the reflection coefficient from a plane surface at normal incidence.
Figure 4. Calculated values of loss tangent, as a function of relative amplitude and phase of the reflection coefficient from a dielectric surface at normal incidence, in intrinsic material.
Figure 5. Impulse radar traces using 400 MHz antennas held 2.1 m above sea ice surface (right) and perfectly reflecting screen (left). Amplitude scale (horizontal) is same for both traces. Vertical scale is time delay after start-of-scan pulse. Later returns are increased by a time-gain amplifier. Note the decreased amplitude and the phase lag of the ice surface reflection compared to the reflection from the screen.
Figure 6a. Amplitude spectra of the reflected wavelets, shown in Figure 5. Note the broad-band nature of the impulse radar signal.

Figure 6b. Modulus of the ice surface reflection coefficient as a function of frequency, obtained by taking a ratio of the spectra in 6a. Dashed lines are extrapolations.
THE FREQUENCY AND MAGNITUDE OF DRIFT-ICE GROUNDINGS FROM ICE-SCOUR TRACKS IN THE CANADIAN BEAUFORT SEA

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ABSTRACT

A study of the morphology of ice scours in the Canadian Beaufort Sea and their variation with water depth is described. Within specific bathymetric zones scour depth frequencies are distributed exponentially and Gumbel's extreme-value distribution is used to describe maximum scour depths. When combined with related information on sedimentation, the drift-ice regime, and sea level change, the statistical nature of ice-scour tracks is used to: (1) differentiate areas of contemporary and relict scouring, and (2) build a theory for estimating the rate of scour additions for various depths of ice keel penetrations beneath the seabed. Scour additions measured over periods of a few years by repetitive seafloor mapping are described also.

INTRODUCTION

The grounding of sea ice pressure ridges and glacier ice islands constitutes a significant hazard for seabed installations such as wellheads and pipelines on the Arctic continental shelf. Design for protection of such installations requires knowledge of the depth and frequency of bottom scouring by drift-ice keels. If the extreme scour depth (magnitude) and impact frequency at a particular site could be predicted from a knowledge of ice keel depths and movement, and bottom sediment type, then ocean bottom structures could be safely buried. Prediction of future extreme scouring depths for engineering design purposes requires a comprehensive record of past groundings that rarely if ever exists. The probability of ice-scour occurrence must then be estimated from the inferred draft of ice keels and their flux across an area of concern. Alternatively the seabed surface which is commonly creased and scoured in areas of drift-ice grounding, may yield information on the past performance of the ice grounding process. This paper illustrates some of the basic methods for the latter approach and develops results on the limit, depth, and rate of ice scouring for the Canadian Beaufort Sea.

METHODS AND SOURCES OF DATA

The ice-scour pattern on a seabed is a complex cumulative record that depends on sea ice regime as well as sedimentation and sea level change. Clearly the Holocene geological history of a shelf must be appreciated before the ice-scour hazard can be interpreted. Most of these conditions are met in the Beaufort Sea where extensive marine geological and geophysical studies have been completed by government and the petroleum industry with respect to ice-scour occurrence and its geological setting (Hnatiuk, 1972; Pelletier and Shearer, 1972; Shearer and Blasco, 1975; Hnatiuk et al., 1976; Hnatiuk and Brown, 1977; Lewis, 1977).
Side-scan sonar, echosounding, sediment grab and core sampling, and seismic reflection and refraction were all conducted since 1970 throughout the Canadian Beaufort Sea culminating in 1974 and 1975 with the joint industry-government Beaufort Sea Project that undertook an environmental assessment of the region prior to offshore drilling in 1976. Selected aspects of the ice-scouring study (Lewis, 1977) are presented here based on a statistical summary of the morphology of ice-scour tracks, repetitive seafloor mapping, and general geological investigations of the sediments, shelf and history of the Beaufort Sea. A total of 2100 line km of echosounding data and 3600 line km of side-scan sonar data distributed on ship traverses throughout the Beaufort shelf area (Fig. 1) were examined for ice-scour track characteristics.

THE BEAUFORT SEA ENVIRONMENT

The continental shelf in the southeastern Beaufort Sea is an extension of the low-lying Arctic Coastal Plain. The shelf, composed of a thick succession of Tertiary and Quaternary sediments, slopes very gently northward to the 100-m isobath up to 150 km offshore. The shelf surface is regular and smooth on a large scale, broken only by major embayments such as Mackenzie Canyon and Kugmallit Valley (Fig. 1).

The shelf was relatively stable during the Late Quaternary but has been subject to wide swings in shoreline position with worldwide eustatic changes in sea level. For the past 14 000 years the sea has inundated the previously exposed shelf from at least the 70-m isobath to the present coastal position. Fine-grained sediment (mud) derived from erosion of unconsolidated coastal bluffs and suspended matter in the Mackenzie River discharge is accumulating on the shelf at an average rate of 0.5 mm/yr east of Mackenzie Canyon (Pelletier, 1975; Lewis, 1977). Here the mud deposit is continuous, ranging from a few metres to over 20 m thick. This cohesive material is an ideal recording medium for drift-ice groundings for it is easily incised by dragging ice keels yet it preserves the scour form by resisting normal marine erosion.

Strong winds tend to blow in early winter from the W and NW (Wilson, 1974). Strong winds are also common from SE and N. By applying stress on the winter ice surface such winds are important sources of energy for sea bed ice-scouring.

For about 3 summer months the sea over the continental shelf is frequently free of ice although it is always subject to closure when storms drive the polar pack ice southwards. Each fall and winter the shelf waters are ice-covered (Kovacs and Meller, 1974; Reimnitz et al., 1977). A fast ice zone starting at the shore and building around grounded ridges may extend outward to about the 20-m isobath through winter. A shear zone composed of first-year ice with some multi-year floes and ice island fragments extends from the seaward edge of the fast ice to the edge of the continental shelf approximately. It is a highly deformed and severely ridged ice zone that acts as an interface between the westward circulating ice of the Pacific Gyre (centred in the Arctic Ocean) and the fast ice. The polar pack ice zone, composed mostly of multi-year floes, covers the Arctic Ocean and rotates clockwise in the long term. Its short term movement is governed by the wind stress field.

The sea floor is virtually saturated with ice-scour tracks (Figs. 2 and 3) from 15- to 40-m water depths. Scour density diminishes rapidly in deeper water, essentially reaching zero by 80 m. The nearshore zone (<10 m) may be ice-scoured but its non-cohesive sandy sediment appears to be smoothed out seasonally by wave and current action. The dominant scour orientation is (110-290°) ± 40°. Scour-track widths range from a few metres to hundreds of metres, the widest are multiple scours cut by adjacent projections on a single ice keel. Scour depths on the average range from 0.5 to 1.0 m but maximum depths range to at least 6 m below the seafloor. Some very rare
occurrences range to 7.6 m and one unverified scour is 10 m deep. The scour morphology is principally related to water depth but is also influenced by topographic irregularities such as embayments, depressions and shoals.

MEASUREMENT OF SCOUR DEPTH

Scour depth is scaled from echosounder profiles (Fig. 2) of the seabed. Most profiles were generated by an Edo-Gifft system or a Kelvin-Hughes system having transducer frequencies ranging from 12 to 30 kHz and beam angles between 17° and 28°. Scour depths were scaled by estimating the vertical distance of the deepest point on the scour profile below the level of the adjacent 'undisturbed' seafloor. This measure was easily done where scours are widely-separated; where closely-spaced the measure became somewhat more subjective. Scours deeper than 6 dm (2 ft) were generally distinctive and satisfactorily measured to the nearest 3 dm (1 ft). Shallow scours less than 6 dm deep were not recognized easily and their numbers are likely underestimated severely.

EXPONENTIAL DISTRIBUTION OF SCOUR DEPTH

The frequency distribution of scour depths, whether for scours from a particular locality or for over 2000 scours throughout the Beaufort Shelf, is characteristically exponential in form. The shallowest scours are most common and with increasing scour depth the frequencies decrease rapidly such that scours of extreme depth occur rarely. The distribution is shown clearly by histograms of scour depths (>5 dm) derived from echo sounding profiles of the seabed. The histogram form suggests that scour frequencies decrease exponentially with increasing scour depth. This is expected generally, because the deeper scouring events presumably require production and dissipation of extreme amounts of kinetic energy and are therefore rarer occurrences.

The simplest analytic form for the scour distribution is \( n = n_0 e^{-kd} \) where

\[ \begin{align*}
  d &= \text{scour depth in decimetres (0.1 m);} \\
  k &= \text{a parameter of the distribution (dm}^{-1}); \\
  n &= \text{number of scours per dm of scour depth per km along the seafloor;} \\
  n_0 &= \text{number of scours per dm at zero depth per km. This last quantity is} \\
  \text{physically interpreted to represent the number of scours that are just barely} \\
  \text{visible and about to be sedimented in.}
\]

The analytical exponential distributions are compared with selected sample data in 10-m bathymetric increments from a 135-km transect across the shelf north of Pullen Island (Fig. 1). The fit is reasonably good with correlation coefficients on the regression of \( \log n \) vs \( d \) ranging from -0.82 for the 50- to 60-m depth interval to -0.95 for the 20- to 30-m depth interval.

The total number of scours (per km), \( N_0 \), in the distribution is equal to the sum of all scours (area) under the curve (Fig. 4) and is found by integrating equation (1) from 0 to \( \infty \) depth:

\[ N_0 = \int_0^\infty n \, dd = n_0 \int_0^\infty e^{-kd} \, dd = \frac{n_0}{k}, \text{ the total scours per km.} \]  

EXTREME-VALUE DISTRIBUTION OF SCOUR DEPTH

The ability to predict extreme penetration depths for ice keels in an area of interest is important from an engineering point of view. This was accomplished by reading the maximum scour depths from 321 sections of sea floor profiles, each 1.852 km (1 nautical mile) long, distributed throughout the scoured terrain of the Beaufort Shelf.
The sample transects were grouped within 10-m increments of water depth to ensure similarity in the initial scour distribution for determination of the extreme values. Each group contained between 24 and 88 sample transects.

The statistics of extremes (Gumbel 1954, 1958) is applied to the observed scour depths: 1) to determine if extreme scour depths can be predicted with a given probability and 2) to test if unusually deep seabed depressions fall within the range of scour depths that are reasonably expected for the scour population and 3) to see if the series of maximum scour depths exhibits regular behaviour with water depth. Use of these statistics requires 4 conditions which are generally met: (i) large numbers (>6) of (ii) unlimited, (iii) independent variates with (iv) similar initial distributions (satisfied for maximum scour depths in similar water depths).

Extreme-value theory is developed in terms of an asymptotic approximation, that is, one increasingly valid for larger and larger n. The most useful limiting distribution, the first asymptotic distribution, applies to initial distributions like the exponential. Its cumulative distribution function is

$$\Phi(x) = e^{-e^{-y}}$$

where y is a linear function of the observed variate x with coefficients estimated from the sample of maximum values. The reduced variate y is

$$y = \frac{1}{\beta} (x - \mu)$$

where \(\mu\) is the mode of the frequency curve. It defines the position of the curve on the x-axis. The parameter \(\beta\) is a measure of spread related to the standard deviation for the cumulative distribution above. (Krumbein and Lieblein, 1956).

In using the extreme-value method the j observed maxima are ranked, 1 to j in order of size from the smallest to the largest. These values are then transformed into cumulative frequencies \(P_i\) by the relation \(P_i = i/(j + 1)\) where i is the rank of the i\(^{th}\) observation counting from the smallest. The \(P_i\) are plotting positions on a cumulative probability scale for each maximum scour depth observation. The data are plotted on special extreme-value probability paper (Fig. 5) designed so that the ideal extreme-value distribution will plot exactly as a straight line. Thus the closeness of the plotted points to a straight line is an indication of how well the data fit the theory.

The extreme-value probability paper (Fig. 5) has a uniform scale along the horizontal axis that is used for the observed maximum scour depth values. The vertical axis then serves as the probability scale and is marked according to the double exponential formula (3). For any point on the extreme-value line this scale gives the probability that a 1-km segment of seafloor will have a maximum scour depth less than the value indicated on the horizontal axis. The complement of this value, that is, \((1-\Phi(x))\), gives the average probability that a 1-km segment of seafloor will have a maximum scour depth that equals or exceeds the same indicated value.

The T scale (recurrence interval) on the right-hand vertical axis is the reciprocal of \((1-\Phi(x))\). Its value is the average number of samples, that is the average number of km of seafloor that must be observed, before the indicated maximum scour depth occurs or is exceeded once. For convenience, the scour depth equivalent to a recurrence interval of 100 is termed the hundred-km scour. This means that for the example in Fig. 5 one would expect to survey 100 km of seafloor, on the average, in the 38 to 48 m depth range, before encountering a scour of 5.24 ± .8 m depth or greater depth.
The control curves indicating the $P = 0.68$ confidence band around the theoretical line of expected extremes, is calculated and plotted according to the procedure described by Gumbel (1954, p. 48). The fact that most of the data points fall within these curves (Fig. 5) and that it is estimated all would fall within a 95% confidence band confirms the theoretical extreme-value distribution as a valid description of the frequency of occurrence of maximum scour depths.

The equation for the line of expected extremes is determined from (4) as

$$x = \gamma y + \mu$$

through the methods of Gumbel (1954), essentially by a least squares regression on the plotted data.

The expected extreme depths are based on distances across the seafloor. Depths for any selected probability level may be used to estimate 'safe' burial depths for seabed installations with respect to the total 'geological' record of drift-ice groundings.

IDENTIFICATION OF OFFSHORE AREAS OF RELICT SCOUR

The differentiation of active and relict scour on the Beaufort Sea shelf depends on the premise that in zones of active scouring, scour depths will increase with increasing water depths offshore. In relict scour zones the ancient scours are expected to be diminished in depth by long term sedimentary infilling, by wave or current action on the bottom and by the action of benthic fauna.

An offshore increase in active scour depth is anticipated for several reasons. It is well known that in Arctic sea ice deep keels are far less numerous than are shallow keels. In fact Hibler (1974) and Kovacs (1972a) infer that the probability of encountering keels decreases exponentially with increasing keel depth. It is also generally accepted that wind stress is a dominant source of energy for the sea-ice pressuring and scouring process (Kovacs 1972b). It is obvious that only shallow keels can enter or be developed in shallow water and that in deeper water only the deep keels can touch and drag the bottom. Assuming wind stress is applied equally over both offshore and nearshore ice surfaces in the low coastal plain setting of the Beaufort Sea, and that this energy results in ice drift and bottom drag, a relatively greater amount of energy will be available for scouring by deep keels than for shallow grounded keels simply because there is a greater ice surface area per deep keel. In other words the widely-scattered deep keels can collect more energy from wind stress on the ice surface than the more closely-spaced shallow keels. The expected result, given that other controlling factors are equal such as ice state, sediment resistance and seabed slope etc., is deeper scours in deeper water. Of course scouring will be most frequent in shallow water where ice keels of appropriate depth are more numerous.

The extreme scour depths having a probability of 1% for exceedance were calculated from all available data and organized bathymetrically to show the actual variation of scour depth with water depth in the Canadian Beaufort Sea (Fig. 6). The rising curve to 47 m water depth is interpreted as the active scouring zone. The decline in offshore scour depths reflects a zone of relict scour in which the seafloor was progressively abandoned by the ice canopy with eustatic rise in sea level between 10 000 and 3000 years ago (Lewis, 1977).

Each point on the profile represents an extreme value derived from all the scour depths measured within a 10-m water depth interval, the interval mid-depth being the plotting point. The value at 47 m is based on scours in the 42- to 52-m depth interval and the value at 49 m is based on scours in the 44- to 54-m depth interval, etc. The
extreme scour depths decline between these points and continue to do so in deeper waters. Scour depths likely start to drop significantly in the 42- to 44-m depth range and the deepest scours tend to occur at about the 44-m depth contour. The responsible keels are 6 m deeper (0.99 probability). Thus the seaward limit for the hazard zone due to bottom impact by drift ice is inferred to be the 50-m depth contour from the data analyzed in the study.

An inferred 50-m depth limit for scouring ice keels is remarkably similar to related data on Arctic sea ice morphology. The maximum keel depth observed from submarine sonar traverses in the Arctic Ocean is 47 m (Lyon, 1967). Consideration of ice strength and pressure ridge-keel formation suggest an upper limit in the order of 40 to 50 m (Parmerter and Coon, 1973; G.R. Pilkington, pers. comm.) because the pressure required for further keel growth exceeds the strength of ice and the pressure keel is apparently widened, not deepened. Wadhams (1975) also estimates maximum keel drafts of 45 and 48 m for the Arctic Ocean ice cover based on a pressure ridge density model.

### REPETITIVE SEAFLOOR MAPPING TO DETECT SCOURING FREQUENCY

The direct method for testing the longevity of an ice-scour pattern and for measuring the frequency of new ice impacts is to map a particular area of the seafloor on a time series, say annually. Micro-relief, including ice scour, is well recorded by side-scan sonar and this technique was applied at 2 sites, each about 14 km², in the Beaufort Sea. Both sites are in 15 to 20 m of water depth north of Pullen Island located where the fast ice-shear zone interface is believed to occur during winter. The sites were surveyed first in 1971 and 1972 by the Arctic Petroleum Operators Association and then again in 1974 for the joint industry-government Beaufort Sea Project (Hnatiuk, 1972; Hnatiuk and Brown 1977; Lewis, 1977). The side-scan records were corrected for distortion by anamorphic photography and pieced together to form mosaics which could be compared visually for scour additions (Fig. 3). These mosaics reveal an intensely disturbed seafloor showing evidence of dragging ice forms by both single-pronged and multiple-tipped keels.

Analysis of the mosaics reveals that 1 scour is added each year on the average for each 3 km² of seafloor. Approximately 1.1 ± 0.9% of the seafloor is disturbed annually. If this disturbance were added each year without overriding previous scours the seafloor would be completely reworked in 90 years. The actual rate, of course, is extremely variable; adjacent sites in the 15- to 20-m bathymetric zone reveal periods ranging from 50 to 500 years for complete reworking of the seabed. Scour additions, measured on a transect basis rather than an areal basis, come to 0.19 ± .06 scours/year/km (0.35 ± .12 scours/year/n. mile).

### THEORETICAL CONSIDERATIONS OF SCOURING FREQUENCY

It is possible to speculate on the temporal frequency of ice scouring on a smooth seabed covered with mud sediment that records all ice groundings but resists normal marine erosion by current and wave action. These ideal and simplistic conditions appear to hold for much of the gently-sloping Beaufort Sea shelf north of Mackenzie Delta and Tuktoyaktuk Peninsula. Certain other assumptions underlie the theoretical model for scouring frequency: (1) Only dragging ice keels incise the sediments and create the micro-relief seen on the seafloor. (2) The areal and temporal distribution of ice groundings is random in the short term but statistically uniform in the long term within specific bathymetric zones. (3) The present nature of sedimentation, climate and sea ice regime is representative of conditions during the last few centuries and millenia. (4) Once cut, a scour track remains on the sea floor without change until it is overridden by another ice keel or is infilled and buried by
sediment accumulation on the shelf. (5) The subsequent calculations are based in part on scour widths and depths measured from seafloor profiles. It is further assumed that the echosounding traverses providing this data are run approximately normal to the major scour trend.

It is reasonable to suppose that the scour process has reached some sort of equilibrium over the long term such that the average number of scours seen on the bottom remains constant with time. That is, the number of new ice keel impacts equals the number of scours leaving an area in a given period of time. Under these conditions an annual scour budget may be set up in which:

\[
\frac{\text{Number of new impacts}}{\text{Number of scours infilled}} = \frac{\text{Number of scours removed by}}{\text{new impacts (superimposition)}}
\]

The first term on the right hand side of the scour budget (6) is evaluated with general reference to a histogram of the scour depth distribution (e.g. Fig. 4) with depth classes of width \( t \). The area of each rectangle equals the number of scours in that class because the class ordinate is expressed in scours per unit depth and the class width is in the same depth units. Sediment accumulating in each scour bottom reduces the scour depth and, after a period of time, moves it into a shallower class of the scour depth distribution. Ultimately each scour passes through all the shallower depth classes until it is filled in and vanishes from the scour count. Let \( u \) be the annual sedimentation rate or the annual decrement in scour depth.

The annual number of scours entering class \( m \) for example, by infilling of class \( m+1 \) scours is \( u/t \cdot n_{m+1} \). The annual number of scours leaving class \( m \) is \( u/t \cdot n_m \) and the net annual loss from class \( m \) is \( u/t \cdot (n_m - n_{m+1}) \). The total annual loss from all classes is

\[
\sum_{m=0}^{\infty} u/t \cdot (n_m - n_{m+1}) = u/t \cdot n_0 = u n_0
\]

for class widths of unity. The expression reduces to \( n_0 \) because there are no scours of infinite depth (\( n_\infty = 0 \)) and all other terms except the first cancel out. This term, \( n_0 \), is the initial value for the exponential distribution (1) of scour depth. The above result (7) will equal the annual number of new ice impacts in situations where scour superimposition may be neglected.

A correction for superimposition is made by considering scour widths and ice keel impacts across a transect on the seafloor of length \( L \). The location of an impact is random and therefore the probability of an ice keel hitting one existing scour of width \( w \) in the interval \( L \) is \( w/L \) or \( p \) say. The probability of hitting the average total number of scours \( N \) in the interval is \( p \cdot N \) and the probable annual number of superimpositions by \( V_o \) new impacts per interval \( L \) is \( p \cdot N \cdot V_o \)

The rate of scouring in terms of the annual number of all new impacts over a seafloor interval \( L \), say 1 km, is found by substituting equations (7) and (8) in the scour budget (6) leading to: \( V_o = u n_0 / (1 - p N_o) \)

The above expression (9) is easily generalized to describe impact rates for scours deeper than any reference depth, \( d \), since the scour budget and its ancillary reasoning are not necessarily limited to the zero depth case by any of the underlying assumptions. What is really required is an estimate for the number of scours which will disappear from the scour count for the subpopulation of scours deeper than \( d \), that is, values of \( n_d \) and \( N_d \) are required analogous to \( n_0 \) and \( N_0 \) in equation (9). The former
(n_d) can be calculated from the expression n = n e^{-kd} once the parameters n and k are determined for the total scour population. The analogous value for N in the denominator of equation (9) is N_d, the total number of scours per km deeper than d. N_d is shown to be n /k e^{-kd} by integrating the scour exponential depth distribution (1) from d to \( \infty \). Thus the generalized expression for impact rates is

\[ V_d = \frac{un_d}{(1 - pN_d)} \]  

(10)

The annual rates of scouring ice keels and their return periods for seafloor intervals (Fig. 7) are calculated across 1-km transects for various points on a generalized section across the Beaufort Shelf using equations (9) and (10) with values derived from the exponential scour depth distribution and with long term sediment accumulation rates from mud thickness considerations (Fig. 6). These average values apply generally to the area between Mackenzie Canyon and Kugmallit Valley from which the bulk of the basic data are derived. The similarity of these results with short term scour frequencies derived from repetitive mapping between the 15- to 20-m isobaths is encouraging (Fig. 7). However, additional corroborative mapping results over long periods of time and in varied settings are required before the speculative rates derived herein can be verified and refined.

CONCLUSIONS

The scouring process by sea-ice keels appears to be a series of bottom impacts, random in space and time over the short term but statistically coherent over the long term. Because the scour record on a muddy seabed is preserved for long periods of time until it is infilled by sedimentation or destroyed by subsequent ice scouring, the seabed ice-scour tracks provide a good record of the long term average behaviour of the drift-ice impact process.

The record of scour depths is exponentially distributed and yields information on the magnitude of ice keel penetration. The occurrence of maximum scours within 1-km sample transects describes the probability of encountering extreme depths with application of Gumbel's extreme-value distribution and may be used to estimate safe burial depths for seabed engineering structures.

Within the Beaufort Sea, the statistical parameters of scour distribution vary with water depth such that extreme scours become deeper in deep water in contemporary ice-scouring zones. The reverse trend indicates relict scour zones which occur beyond the 50-m isobath in Beaufort Sea. This break in trend is fundamentally related to the geological history of the shelf, particularly sea level change and sedimentation, during the past 10 000 years.

Under some general environmental assumptions which appear to hold for the Beaufort Sea at least east of Mackenzie Canyon, the scour count (number per line km) could represent a dynamic equilibrium over the long term in which scour losses by sedimentation or superimposition by new scours are balanced by scour additions. The annual rate of ice impacts for any reference level at or below the seabed is shown by a theoretical derivation to be a simple function of the sedimentation rate, scour widths and the parameters of the scour depth distribution. Impact rates, calculated from these variables, compare well with the available but meagre data on scour additions from seabed mapping. They permit the investigator to estimate and project the frequency that a buried structure, such as a pipeline, would be struck by drift-ice keels over wide areas of the continental shelf.
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Figure 1. Bathymetry of the Canadian Beaufort Sea. Side-scan sonographs and echograms were obtained for a study of ice scour morphology on traverses within the area encircled by the heavy line. The location of the seafloor mosaics shown in Fig. 3 is indicated north of Pullen Island.

Figure 3. Portions of repetitive seafloor images from 1972 (left) and 1974 (centre) showing scour additions over a two-year period (right). The images are constructed as mosaics of side-scan sonographs. The encircled features labelled 2 and 3 are unique persisting features in the pre-1972 scour pattern which serve to confirm that these mosaics depict the same portion of seafloor.
Figure 2.
Kelvin Hughes MS44F (30kHz) echogram (about 1/4 scale) showing the irregular micro-relief, typical of an ice-scoured seafloor. Note that the scours tend to become deeper and less frequent in deeper water.

Figure 4.
Examples of the exponential distribution of scour depths for the 20-30 m and 50-60 m intervals of water depth in the Beaufort Sea.

Figure 5.
The cumulative distribution of maximum scour depths observed per nautical mile at 43 m water depth (38 to 48 m) is shown on exponential extreme-value probability paper. The heavy line shows the equivalent distribution for maximum depths from 1 km sample transects.
Figure 6.
Extreme (P = .01) and mean scour depth, scour azimuth, scour width, scour density (for all depths and deeper than 2 m) and sedimentation rates vs water depths for the Beaufort Sea Continental Shelf, Canada.

Figure 7.
Theoretical ice keel impact rates and return periods for a 1 km section of a pipeline placed at various depths relative to the seafloor. Values are plotted vs water depth for the Beaufort Sea Continental Shelf, Canada.
ABSTRACT

The main subject of this paper is the impact forces necessary to break up by bending failure a floating ice sheet when it collides with a sloping structure with a certain velocity.

Theoretically the ice floe is considered as a semi-infinite large elastic plate resting on an elastic foundation and having uniform time-dependent boundary conditions. The inertia force of the plate and the hydrodynamic force of the water are included. Solutions to the equations for the vertical and the horizontal deformations are given. These are used to find the failure distance, failure time, and failure forces. Model tests in the laboratory have been performed and a comparison between test results and theory is shown. In this connection a statistical way of determining the failure forces is suggested.

INTRODUCTION

Considering the interaction between drifting ice sheets and sloping structures placed in open waters and rivers it is obvious that the collision velocity may be of great importance e.g. on the maximum of the ice forces developing and on the frequency by which they are acting - the dynamic effects. Nevertheless the Russian and Canadian design codes together with the great majority of other formulae given in the literature concerning dynamic ice pressures do not include this dependence (Michel, 1970, and Neill, 1976).

The fundamental failure mechanism in the case of sloping structures is the bending failure. Depending on the extension and the shape of the structure under consideration the bending failure can either stand alone or follow after the formation of cracks radial to the structure. This dynamic bending failure has earlier been treated by Reeh, 1972, using the plate theory for an elastic plate of finite length with only vertical forces, and by Tryde, 1972, who used a more empirical method. Neither of these included the horizontal forces or the hydrodynamic influence from the water.

In this approach a semi-infinitely large ice sheet is considered. It is assumed that the stiffness of the structure is much greater than the stiffness of the ice-water system, which means that all deformations, including local crushing, caused by the impact will take place in the ice-water system. The ice is considered a homogeneous,
isotropic, and elastic material, which will suffice in the case of quickly varying loads of short duration. The water is regarded a homogeneous and incompressible ideal fluid.

LIST OF SYMBOLS

\( \beta_v \) angle of inclination of structure to horizontal
\( u \) velocity of ice sheet in direction perpendicular to structure
\( h \) thickness of ice (constant)
\( \ell = \frac{D}{\gamma} \) characteristic length
\( D = \frac{Eh^3}{12(1-\nu^2)} \) flexural rigidity of ice sheet
\( E \) Young's modulus of ice
\( \nu \) Poisson's ratio of ice
\( \gamma = \rho_w g \) specific mass density of foundation
\( \rho_i \) (mass) density of ice
\( \rho_w \) (mass) density of water
\( r_t \) tension strength of ice
\( r_c \) uniaxial crushing strength of ice
\( r_{bc} \) bending strength in compression
\( \sigma_{bc} \) resulting stress under bending and compression
\( \mu \) friction coefficient
\( k \) contact coefficient between ice and structure

PHYSICAL FORMULATION

The situation under consideration is shown in Fig. 1. An ice floe of thickness \( h \) and drifting at velocity \( u \) encounters a sloping plane with inclination angle \( \beta_v \) at time \( t^* = 0 \). The edge of the floe crushes resulting in deformations of the ice-water system. The deformations increase until a rupture stress has been reached somewhere in the ice sheet. The floe then breaks and the impact forces drop to zero. If the ice sheet has sufficient energy or if external forces are acting, the rupture situation may be repeated.

Neglecting the change in \( u \) during the impact, we assume for the vertical force

\[
V^* = kr_c u t^* \quad [\text{N/m}] \tag{1}
\]

where \( k \) is the contact factor introduced by Korzhavin, 1962. The horizontal force may then be written

\[
H^* = kr_c u t^* \frac{\tan(\beta_v) + \mu}{1 - \mu \tan(\beta_v)} \quad [\text{N/m}] \tag{2}
\]

Because \( H^* \) acts eccentrically on the sheet near the underside, it produces a bending moment given by

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Having uniform conditions in the x-direction (beam situation) along the structure, see Fig. 2, the physical problem can be formulated as a two-dimensional mathematical problem of solving two partial differential equations, one of fourth order concerning the vertical deflection ($w$) and another of second order governing the horizontal deformation ($v$) (Timoshenko et al., 1959).

Transverse deflection

For the ice sheet the theory of a thin plate resting on an elastic foundation will be applied to predict the vertical deflection. For the water base the equation of continuity and the equation of motion may be applied. The major influence from the water on the deflection of the plate in a dynamic situation is, apart from the buoyancy, the hydrodynamic mass. In this investigation the hydrodynamic mass will be taken into consideration by adding a constant to the inertia term in the plate equation.

Neglecting the effect on the bending from horizontal forces in the middle plane of the plate, the differential equation for the deflection line, in dimensionless form, may be written

$$
\frac{\partial^4 w(y,t)}{\partial y^4} + c_1 \frac{\partial^2 w(y,t)}{\partial t^2} + w(y,t) = 0
$$

with the dimensionless variables $x = x^*/\ell$, $y = y^*/\ell$, $w = w^*/h$, and $t = t^*/\sqrt{\rho_1 g \ell}$. The boundary conditions are

$$
y = 0: \frac{\partial^3 w(y,t)}{\partial y^3} = c_M \frac{\partial w(y,t)}{\partial t} = 0
$$

$$
y \to \infty: w(y,t) = \frac{\partial w(y,t)}{\partial y} + 0
$$

$$
t = 0: w(y,t) = \frac{\partial w(y,t)}{\partial t} = 0
$$

where

$$c_M = k \frac{\rho_1}{2 D} \frac{h}{\ell} \sqrt{\frac{\rho_1}{\ell}} \frac{\tan(\beta_v) + \mu}{1 - \mu \tan(\beta_v)}$$

$$c_V = k \frac{\rho_1}{D} \frac{h}{\ell} \sqrt{\frac{\rho_1}{\ell} \gamma h}$$

$$c_{MV} = \frac{c_M}{c_V} = \frac{1}{2} \frac{\tan(\beta_v) + \mu}{1 - \mu \tan(\beta_v)}$$

and $c_1 = c_2 + 1$ is an inertia coefficient including the inertia of the plate and the added mass ($c_2$) of the water.
The solution to eq. (4) for the deflection line can be written (Spørensen, 1976)

\[ w(y,t) = \psi_1(y) t + \frac{1}{\pi} \int_{1/\sqrt{c_1}}^{\infty} \psi_2(y,r) \frac{\sin(rt)}{r^2} \, dr \tag{5} \]

where

\[ \psi_1(y) = \left[ (c_M - \sqrt{2} c_V) \cos \left( \frac{\sqrt{2}}{2} \right) - c_M \sin \left( \frac{\sqrt{2}}{2} \right) \right] \exp \left[ - \frac{\sqrt{2}}{2} \gamma \right] \]

\[ \psi_2(y,r) = (a^2 c_V - a^2 c_M) \left[ \cos(ay) - \sin(ay) + \exp(-ay) \right] \]

and

\[ a = \frac{1}{\sqrt{c_1 r^2 - 1}} \]

Looking at eq. (5), \( \psi_1(y) \) is the solution to the corresponding static problem and the second term accounts for the dynamic part.

**Longitudinal deformation.**

For the inplane deformations the theory of shells will be applied. Assuming no friction between ice and water the dimensionless differential equation governing this problem can be written

\[ \frac{\partial^2 v(y,t)}{\partial y^2} - c_3 \frac{\partial^2 v(y,t)}{\partial t^2} = 0 \tag{6} \]

with the boundary conditions

\[ y = 0 : \frac{\partial v(y,t)}{\partial y} = -c_4 t \]
\[ y \to \infty : v(y,t) \to 0 \]
\[ t = 0 : v(y,t) = \frac{\partial v(y,t)}{\partial t} = 0 \]

where

\[ c_3 = \frac{1}{3} \left( 1 - \mu \tan(\beta_Y) \right)^2 c_M V \]
\[ c_4 = k \left( \frac{r_c u \sqrt{V}}{E h} \frac{1}{\gamma} (1 - \nu^2) \frac{\tan(\beta_Y) + \mu}{1 - \mu \tan(\beta_Y)} \right) \]

Solving eq. (6), we get the following expression for the longitudinal deformation

\[ v(y,t) = \begin{cases} 0 & 0 < t < \sqrt{c_3} y \\ \frac{1}{2} c_3 c_4 (t/\sqrt{c_3 y})^2 c_3 y & y \leq t \end{cases} \tag{7} \]

**Rupture situation**

Now, being able to calculate the forces working on the surface of a given section of the plate, it is possible to estimate the stresses. It is then assumed that the failure of the plate will take place where the stresses have their maximum and at the time when the stresses equal the rupture strength, which will be assumed known.
Knowing that the maximum stresses will appear at either the upper or the lower side of the plate, these stresses can be found from the following expression

\[
\sigma_{bc}(y,t) = \frac{H^*(y,t)}{h} + \frac{M^*(y,t)}{1/6 h^2} \quad [\text{N/m}^2]
\]

(8)

For the horizontal force we have

\[
H^*(y,t) = \frac{Eh}{1-\nu^2} \frac{\partial v^*(y,t)}{\partial y^*} = \frac{Eh}{1-\nu^2} \frac{\partial v(y,t)}{\partial y} \quad [\text{N/m}]
\]

(9)

and the moment is given by

\[
M^*(y,t) = -D \frac{\partial^2 w^*(y,t)}{\partial y^*} = -D \frac{h}{\bar{k}^2} \frac{\partial^2 w(y,t)}{\partial y^2} \quad [\text{Nm/m}]
\]

(10)

By substituting eq. (7) into eq. (9) and eq. (5) into eq. (10) and inserting the results in eq. (8) we obtain for the stresses \( \sigma_{bc}(y,t) \) in the plate

\[
\begin{align*}
\left( \frac{\sigma_{bc}}{r_c} \right) &= -c \left[ \frac{1}{3} C_{MV} \left( t - \frac{1}{\sqrt{3}} \frac{1-\mu}{\tan(\beta_Y)} c_{MV} y \right) \\
&\quad \pm \left[ \Phi_1(y) t + \frac{1}{\pi} \int_{1/\sqrt{C_1}}^{\infty} \frac{\Phi_2(y,r)}{r^2} \sin(rt) \, dr \right] \right] 
\end{align*}
\]

(11)

where

\[
\Phi_1(y) = \left[ (\sqrt{2} - C_{MV}) \sin\left(\frac{\sqrt{2}}{2} y \right) - C_{MV} \cos\left(\frac{\sqrt{2}}{2} y \right) \right] \exp\left[ -\frac{\sqrt{2}}{2} y \right]
\]

\[
\Phi_2(y,r) = (C_{MV} - a) [\cos(ay) - \sin(ay) - \exp(-ay)]
\]

\[
a = \frac{\sqrt{c_1 r^2 - 1}}{C = 6k u h^{-3/4} \left( \frac{Eo_i}{12(1-\nu^2)} \right)^{1/4}}
\]

and concerning the signs:

+ gives the stress at the top side and

- gives the stress at the underside.

In order to find the failure distance \( y_f \), we take

\[
\frac{\partial}{\partial y} \left( \frac{\sigma_{bc}}{r_c} \right) = 0 \quad (12)
\]

The value \( y = y_f \) found from eq. (12) is then inserted into eq. (11). It is now possible to find the failure time \( t_f \) and thereby the failure distance as functions of the corresponding rupture strength, when \( (\sigma_{bc}/r_c) = (r_{bc}/r_c) \). These relations are shown in Fig. 3 and Fig. 4, and it shall be noticed that the curves only depend on system variables and not on the rupture strength, which means that these relations can be used for any kind of material provided the load-deformation diagram of the material is known, as shall be illustrated later.
Looking at Fig. 3 and Fig. 4 it is seen that the curves related to \(c_{MV} = 0\) (no eccentricity moment) are straight lines. For \(c_{MV} \neq 0\) the curves are almost parallel to the curves for \(c_{MV} = 0\), at least over a certain interval depending on the \(c_{MV}\)-values. Considering these intervals, which can be found from the figures, we get the following relations between the variables \(y_f\) and \(t_f\) and the rupture stress

\[
y_f = 0.89(1+4.0c_{MV}) \left( \frac{1}{C} \frac{r_{bc}}{r_c} \right)^{1/3}
\]

and

\[
t_f = 2.41(1+2.26c_{MV}) \left( \frac{1}{C} \frac{r_{bc}}{r_c} \right)^{2/3}
\]

which means that both \(y_f^*\) and \(t_f^*\) are nearly proportional to the thickness \(h\) of the plate. Another interesting thing is that both are also increasing with increasing \(c_{MV}\)-values.

Using the maximum horizontal force acting on a vertical structure as reference a reduction coefficient \(C_F\) can be defined

\[
H^* = C_F \frac{H_{max}}{r_c h} [N/m]
\]

and by combining eq. (2), eq. (14), and eq. (15) we get the following expression for the reduction coefficient

\[
C_F = 1.1(1+2.26c_{MV}) \left( \frac{(1-\nu^2)c_1}{E} \right) k^{1/3} u^{1/3} \left( \frac{r_{bc}}{r_c} \right)^{2/3} \frac{\tan(\beta_V)+\mu}{1-\mu \tan(\beta_V)}
\]

An interesting result is that the failure force is not proportional to the contact factor, but to \(k^{1/3}\) in correspondence with the loading velocity \((r_c u)\).

**EXPERIMENTAL VERIFICATION**

In order to verify the theoretical investigation, model tests have been performed. The stationary structure (model) was placed in a flume, and floating on the water an 'ice' floe then drifted against the model along with the water, and the 'ice' floe was broken. For lack of cooling facilities an artificial ice material, approximately scaled in comparison to real sea ice according to the Froude model law, was used. During a test the horizontal and vertical forces together with the moment force and the velocity were continuously recorded. From test to test the inclination angle and the thickness of the plate were varied.

**Artificial ice**

In connection with each test the bending strength without compression, the crushing strength, the tension strength, and the (mass) density of the material were measured. The measurements made it possible to estimate the 'working diagram' and thereby the rupture strength that are necessary in order to use the theory.

The load-deformation diagram for the artificial material used is shown in Fig. 5, from which it is seen that the material acts elastic-plastically in tension and elastically in compression. Basing the calculations on this load-deformation diagram the following rupture strength expression is found:

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\[
\left( \frac{r_{bc}}{r_c} \right) = \begin{cases} 
\frac{1}{(2n+1)^2} & [-(4(3n+1)m^2 + (-8n^2 + 8n+4)m + n(8n+3))] 
0 \leq m \leq \frac{1}{2} \\
1 & 5 \leq m \leq 1 
\end{cases} 
\tag{17}
\]

where \( m = \frac{H(y, t)}{H_{\text{max}}} \) and \( n = \frac{r_t}{r_c} \).

During the tests the material properties were approximately:

\[ \rho_i = 900 \text{ kg/m}^3 \quad r_t = 0.6 \times 10^5 \text{ N/m}^2 \]
\[ r_b = 1.56 \times 10^5 \text{ N/m}^2 \quad r_c = 6 \times 10^5 \text{ N/m}^2 \]
\[ E = 3 \times 10^8 \text{ N/m}^2 \]

With these values relation (17) is shown in Fig. 6.

Test results

In Figs. 7, 8, and 9 the results from two tests with different \( c_{\text{MY}} \)-values (obtained by changing the inclination angle) are shown in comparison with the expected values. The measured reduction coefficients and the measured failure times are in good agreement with the theory, but the measured failure distances are smaller than would be expected from the theory. This may perhaps be explained by the fact that the tests were performed with a model of finite width. This means that the plate at both sides of the structure prevented the section of the plate in front of the structure to be lifted as much as should be expected in the case of a semi-infinite structure. The test results have not been corrected according to this assumption, but it can be done when the corresponding problem of impact of a semi-infinite ice sheet against a sloping structure of finite width has been investigated. This investigation is taking place at the moment.

During these tests the failure of the plate was developing in the following way. First two cracks, one at each side of the structure, were made in the direction of motion forming a cantilever beam with the same width as the structure. Then the forces increased and the beam was broken off. This rupture pattern was repeated very regularly until the floe was stopped by the work carried out in the failure process.

The test results for the \( t_f \)-values and the \( C_f \)-values have been corrected for the forces necessary to make the first two cracks.

The \( k \)-factor

Normally the edge of the ice sheet that meets the structure during the impact will differ from a straight line. This means that the area of local crushing for a given time is smaller than would be expected if this effect was neglected, resulting in a decrease in the loading velocity.

This effect is included in the test results by introducing the contact factor \((k)\) where \( k \) has been estimated for each impact in the following way

\[
k = \frac{H_m}{r_c} \frac{1-u \tan(\beta_v)}{\tan(\beta_v)} \frac{1}{u} \frac{\tan(\beta_v)}{\tan(\beta_v) + \mu} \tag{18}
\]
where \( H_m \) is the measured failure force and \( t_{f,m} \) is the measured failure time.

The k-values from 30 impacts estimated by eq. (18) are shown in Fig. 10 together with the probability density function for a beta distribution corresponding to these k-values. The beta distribution has been chosen because by definition we have \( 0 \leq k \leq 1 \). From Fig. 10 it is seen that the k-values, at least with this material, are almost following a normal probability distribution. Assuming that this is true for all the important variables involved in \( H^* \) and that these are uncorrelated too, \( H^* \) will also be normal distributed. If eq. (15) is written

\[
H^* = f(k, u, h, r_c)
\]

the mean value in the normal distribution is given by

\[
m_H = f(m_k, m_u, m_h, m_{r_c})
\]

and the variance is found from

\[
\sigma_H^2 = \left( \frac{\partial f}{\partial k} \right)^2 \sigma_k^2 + \left( \frac{\partial f}{\partial u} \right)^2 \sigma_u^2 + \left( \frac{\partial f}{\partial h} \right)^2 \sigma_h^2 + \left( \frac{\partial f}{\partial r_c} \right)^2 \sigma_{r_c}^2
\]

To estimate the design force (e.g., the '100 year' force) the only information now needed is the total number of impacts during a given period. If field measurements are missing, a rough calculation by using the mean values to estimate the time between two impacts, given by \( t^* = \frac{y}{u} \), and then dividing this value into the period when ice is acting, can be used.

**REMARKS**

In this paper are only shown the results from two tests, but a detailed report including the complete test program with both artificial and natural ice model tests will be published later (spring 1978) as a part of the fulfilment for the Ph.d. degree.

**REFERENCES**


FIGURES

Fig. 1 Ice sheet hitting a sloping plane.

Fig. 2 Semi-infinite plate on elastic foundation with uniform edge load.
Fig. 3 Dimensionless fracture time $t_f$.

Fig. 4 Dimensionless fracture distance $y_f$. 

\[ t_f = \frac{t^*}{h^{1/2} \left( \frac{\sigma_c}{\gamma} \right)^{1/2}} \]

\[ c_{MV} = 0.2, 0.1, 0.05, 0.025, 0.0 \]

\[ c_{MV} = \frac{1}{2} h^{1/4} \left( \frac{12(1 - v^2) \gamma}{E} \right)^{1/4} \cdot \frac{\tan(\eta\mu) + \mu}{1 - \mu \tan(\eta\mu)} \]

\[ \left[ \frac{1 - C \sigma_{bc}}{r_c} \right]_f = \frac{1}{6} h^{3/4} \frac{h}{k u} \left( \frac{12(1 - v^2) \gamma}{E p_r^2} \right)^{1/4} \left( \frac{r_{bc}}{r_c} \right) \]
Fig. 5 Principal load-deformation diagram.

Fig. 6 Rupture strength diagram.

Fig. 7 Comparison between theory and test results for the failure distance.
Fig. 8 Comparison between theory and test results for the failure time.
Fig. 9 Comparison between theory and test results for the reduction coefficient.

Fig. 10 Distribution of measured k-values together with the corresponding beta distribution.
BUCKLING ANALYSIS OF A SEMI-INFINITE ICE SHEET

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ABSTRACT

The buckling analysis of a semi-infinite plate on an elastic foundation is performed by the finite element method, and the results are plotted in a non-dimensional form. The non-dimensional buckling effective pressure is also plotted, and this graph can be used to determine the value of aspect ratio above which the ice sheet of a particular crushing strength fails in the buckling mode.

1. INTRODUCTION

One of the most important forces in the design of structures in ice infested water is the horizontal thrust applied to the structures by ice. Although considerable amount of experimental and analytical research has taken place to determine the horizontal forces exerted by ice on structures, this problem has not been solved fully, and the designers have generally relied on empirical formulae, design codes and their individual judgement. When an ice sheet impinges on a single vertical structure, such as a pier, pile or tower, it fails either in a crushing mode or in a buckling mode. It has been observed by many investigators (Hirayama, et al., 1973, Zabilansky, et al., 1975, Afanasav, et al., 1972) during small scale tests that the ice sheet fails in the buckling mode when the aspect ratio (structure width/ice thickness) is greater than six.

In this paper, we present the buckling analysis of a semi-infinite ice sheet which is loaded by an uniformly distributed load acting in the plane of the ice sheet over a distance of the straight edge (Figure 1). The ice sheet is treated as an isotropic, homogeneous plate resting on an elastic foundation. The theoretical formulation of the problem of buckling of a plate on an elastic foundation is the same as that of a plate except for the extra term in the differential equation related to the elastic foundation.

Hetenyi (1946) presented the deflection and buckling analysis of infinite beams on elastic foundations. Vlasov and Leont'en (1966) have discussed the theoretical formulation of the buckling of beams, plates and shells resting on elastic foundations, and they have presented solutions of such structures considering only the simply supported boundary conditions. Kivisild (1969) has presented a few formulae to calculate the buckling load without giving any derivation. Kerr (1976) has solved the buckling problem of a tapered beam of ice floating on water. The justification of considering a tapered beam is due to the fact that radial vertical cracks have been observed to originate from the loading region and radiate outward in the ice sheet. Takagi (1976) developed a theoretical solution to the buckling problem of a floating plate stressed uniformly along the periphery of a hole.
The buckling analysis of a plate on an elastic foundation for arbitrary boundary conditions, plate geometry and in-plane loading is complex, and the results of a few numerical analysis are available in the Handbook of Structure Stability (1971). In this paper, the buckling loads and modes of buckling of a semi-finite ice sheet are determined by the finite element analysis. This approach is quite general to perform buckling analysis of plates on elastic foundations for arbitrary boundary conditions, plate geometry and in-plane loading.

2. STATEMENT OF THE PROBLEM

The general differential equation governing the buckling problem of a thin plate on an elastic foundation due to in-plane loading only, can be written in the following manner (Timoshenko and Gere, 1961)

\[ \nabla^4 w + k w = N_{xx} w_{xx} + 2N_{xy} w_{xy} + N_{yy} w_{yy} \]  

(1)

where

- \( w \) - out-of-plane deflection
- \( D \) - flexural rigidity of the plate
- \( k \) - foundation stiffness
- \( N_{xx}, N_{xy}, N_{yy} \) - in-plane stress resultants (force per unit length)
- \( \Delta \) - biharmonic operator in x and y co-ordinates

\[ \Delta = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} \]

The semi-infinite plate on an elastic foundation, as shown in Figure 1, is used as a mathematical model to study the buckling of an ice sheet under a compressive in-plane load of intensity \( P \) which is distributed over a length \( 2b \). The boundary conditions for this problem are as follows:

at \( x = 0 \)  
\[ M_{xx} = V_x = 0 \] (free edge)

and at \( r = \infty \)  
\[ w = w_x, r = 0 \] (clamped edge)

(2)

where \( r = (x^2 + y^2)^{1/2} \)

\( M_{xx} \) and \( V_x \) are the bending moment and the effective shear force. The expressions for in-plane stress resultants \( N_{ij} \) due to the in-plane compressive loading can be written in the following manner (Flügge, 1962)

\[ N_{xx} = \frac{P}{2b \pi} [\Theta_2 - \Theta_1 + \frac{1}{2} (\sin 2\Theta_2 - \sin 2\Theta_1)] \]

\[ N_{yy} = \frac{P}{2b \pi} [\Theta_2 - \Theta_1 - \frac{1}{2} (\sin 2\Theta_2 - \sin 2\Theta_1)] \]

(3)

\[ N_{xy} = \frac{P}{4b \pi} [\cos 2\Theta_2 - \cos 2\Theta_1] \]

where \( \Theta_1, \Theta_2 \) and \( \frac{P}{2b} \) have been defined in Figure 1. The expressions for \( \frac{2b N_{ij}}{P} \) are non-dimensional but these depend upon the width of the in-plane loading \( 2b \).

The differential equation (1) is transformed into non-dimensional co-ordinates \( \xi = x/\xi \) and \( \eta = y/\eta \) where \( \xi = \sqrt{D/k} \), the characteristic length of the plate, thus we have
\[ \nabla^4 w + w = \lambda \left[ N_{\xi\xi} w, \xi\xi + 2N_{\xi\eta} w, \xi\eta + N_{\eta\eta} w, \eta\eta \right] \]  \hspace{1cm} (4)

where \( \nabla^4 \) - biharmonic operator in \( \xi \) and \( \eta \) co-ordinates.

\[ \lambda = \frac{P}{k^3}, \] \hspace{1cm} non-dimensional buckling load

The non-dimensional buckling load \( \lambda \) will depend upon the mode of buckling and the ratio \( b/l \). Hence, for any given mode of buckling, the non-dimensional buckling load \( \lambda \) is a function of the ratio \( b/l \) only.

3. THE FINITE ELEMENT ANALYSIS

Many textbooks (e.g. Gallagher, 1975) discuss the general procedure of the linear buckling analysis of plates by the finite element method. A similar procedure is followed here for the buckling analysis of plates on elastic foundations. The linear buckling analysis disintegrates into two separate analyses; the in-plane stress resultants \( (N_{ij}) \) are first determined for an arbitrary chosen load intensity, and these are then used to determine the critical load intensity (taken to be proportional to the arbitrary chosen load intensity) to cause neutral stability of the plate on elastic foundation. The details of the finite element formulation and procedure are omitted here because they can be found in any one of the many books written on the subject (e.g. Zienkiewicz, 1971; Holland and Moan, 1969, etc.) or in the thesis written by Hamza (1977).

A finite element computer program has been developed to solve the buckling problem of a plate on an elastic foundation using 16 degrees of freedom rectangular compatible finite elements (Figure 2). This computer program is capable of incorporating any kind of boundary condition provided the domain is rectangular. This restriction may also be removed by using triangular or isoparametric elements to fit any irregular shaped domain.

The results of the finite element analysis were first checked by comparing them with the available exact theoretical solutions in the literature for a simply-supported beam and a rectangular plate on elastic foundation. After a good agreement was found between the results of the finite element analysis and the exact solution for test problems, the program was then used to perform a numerical buckling analysis of semi-infinite plate on elastic foundation.

4. BUCKLING OF SEMI-INFINITE ICE SHEET DUE TO UNIFORMLY DISTRIBUTED IN-PLANE LOAD

Because of the limitations of the computer capacity, it is quite difficult to perform the buckling analysis for a semi-infinite domain using the finite element approach. So, a finite large size plate (Figure 3) has been used to simulate the semi-infinite domain of a floating ice sheet.

Due to the symmetry of the problem about the centre line, one half of the plate is discretized into different meshes. Small size finite elements have been used near the in-plane load due to the rapid changes in the stresses and deflection, and large size finite elements have been used in the regions which are further away from the load. The symmetric and anti-symmetric boundary conditions, for the deflection \( w \), are imposed along the line of symmetry to determine the buckling loads for the symmetric and anti-symmetric modes of buckling for the whole plate.

The finite element computer program is used, first, to check whether the finite size of the plate has any effect on the simulation of the semi-infinite domain. Two
different kinds of boundary conditions, simply supported and clamped, are imposed on the edges CE, EF and CD (Figure 3) which simulate the edges that are supposed to be at infinity. The resulting effect of imposing the two different kinds of boundary conditions on the buckling loads and modes is found to be very small (see Table 1).

Since an approximate numerical approach is being used, one must make sure that the numerical results of the analysis converge as the plate is discretized into larger numbers of finite elements. Meshes of 3x3, 4x4, 5x5, 6x6 and 7x7 elements are used to discretize one-half of the plate as shown in Figure 4. The convergence of the first two buckling loads is presented in Table 1 and in Figure 5. It can be noticed that the convergence is good and the mesh of 6x6 elements gives as good results as that of 7x7 mesh. The mesh of 6x6 elements has been used to perform the buckling analysis, and the numerical results for the buckling loads have been calculated for different values of $b/\lambda$ ratio which is varied by changing the value of the modulus of the elastic foundation for low values of $b/\lambda$ ratio and the width of the applied in-plane load for high values of $b/\lambda$ ratio (see Table 2).

The lowest non-dimensional buckling load ($\lambda = P/\kappa \lambda^3$) is plotted with respect to $b/\lambda$ ratio in Figure 6, which also shows a curve passing through most of the points obtained by computations. The curve can be expressed by the equation,

$$\lambda = \frac{2b}{\lambda} + \frac{3.32}{1 + \frac{b}{2\lambda}}$$  \hspace{1cm} (5)

which was suggested by Nevel (personal communications, 1977). One can notice that the curve tends to approach the value of the non-dimensional concentrated buckling load ($\lambda = \frac{P}{\kappa \lambda^3} = 3.32$, where $P$ is the concentrated buckling load) as the value of $b/\lambda$ ratio approaches zero. For high values of $b/\lambda$ ratio, the curve tends to approach the Hetenyi's (1946) buckling load of semi-infinite beam on elastic foundation.

The first three typical buckling modes have been sketched in Figure 7 which depict two symmetric and one anti-symmetric mode about the centre line.

The effective buckling pressure, $p = P/(2bh)$, corresponding to the lowest buckling load is plotted in non-dimensional form $\{(ph)/(k\lambda^2)\}$ with respect to $2b/\lambda$ ratio in Figure 8, in which the points represent the computed results by finite element method and the curve represents the equation,

$$\frac{p}{k\lambda^2} = \frac{\lambda}{2b/\lambda} = 1 + \frac{3.32}{2b/\lambda + (b/\lambda)^2}$$  \hspace{1cm} (6)

If the material properties and crushing pressure ($p_c$) of the ice sheet are known, one can determine from the above equation whether the mode of failure will be in crushing or buckling mode for a given aspect ratio. Alternatively, one can determine the critical aspect ratio for an ice sheet of known material properties and crushing pressure such that if for a given situation the aspect ratio is greater than the critical aspect ratio, the failure will be in buckling mode, otherwise in crushing mode. Using equation (6), the critical aspect ratio can be derived as given below:

$$\left(\frac{2b}{h}\right)_{Cr.} = 2(\ell/h)\left[\sqrt{1 + \frac{3.32}{(p_c-1)}} - 1\right]$$  \hspace{1cm} (7)

where $\frac{p_c}{k\lambda^2}$, $p_c$ being the crushing pressure of the ice sheet.
Taking some typical values of material properties and crushing pressure of ice sheet, the critical aspect ratio has been plotted with respect to the ice thickness in Figure 9. During small scale tests (Nevel, et al., 1977), it has been observed that the mode of failure is buckling when the aspect ratio is greater six to eight, which is in the range of values predicted by equation (7) for ice sheet thickness from 5 cm to 10 cm.

Though many people have observed buckling of ice sheet in the field, but very few accounts have been reported. Roscoe E. Perham (personal communication, 1977) observed buckling of 8 inches thick ice sheet which was pushed against a round cell structure of 33 feet in diameter. In this case, the aspect ratio is about 50 which is again in the range of values predicted by equation (7) (see also Figure 9).

5. CONCLUSION

A finite element computer program is written and tested to determine the loads and modes of buckling using conforming rectangular plate element. Although this computer program can only analyse the stability of a rectangular plate, the method is quite general to handle any geometry and boundary conditions of the plate.

The buckling loads of a semi-infinite plate on an elastic foundation are determined and the lowest buckling load is plotted against the width of the partially distributed load in a non-dimensional form. From this graph, the effective pressure is determined which will cause the plate to buckle. For a particular crushing strength of ice sheet, one can determine the value of critical aspect ratio above which the ice sheet fails in the buckling mode. The computed critical aspect ratios for thin ice sheets are in the range of values of aspect ratios at which buckling failure has been observed during small scale tests.

ACKNOWLEDGMENTS

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REFERENCES


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Figure 3.

Figure 4.
Figure 5.

![Graph showing the relationship between number of elements and load factor, λ, for different element sizes. The graph includes data points for 3x3, 4x4, 5x5, 6x6, and 7x7 element sizes.]

Figure 6.

- Finite element results.
- Non-dimensional buckling load of a semi-infinite beam (Hetenyi 1946).
- Non-dimensional concentrated buckling load.

Curve defined by the equation:

\[ \lambda = \frac{P}{k\ell^3} \]

\[ \ell = \sqrt{\frac{Eh^3}{12(1-\nu^2)k}} \]

\[ \lambda = \frac{2b}{\ell} + \frac{3.32}{1 + \frac{b}{2\ell}} \]
First mode
Second mode
Third mode

Figure 7.

Effective Pressure, \( p = \frac{P}{2bh} \)

\[ \frac{p h}{k \ell^2} = \frac{\lambda}{2b/\ell} \]

• Finite element results.

Non-dimensional buckling pressure of a semi-infinite beam (Hetenyi 1946).

Curve defined by the equation:

\[ \frac{\lambda}{2b/\ell} = 1 + \frac{3.32}{\frac{2b}{\ell} + \left(\frac{b}{\ell}\right)^2} \]

Figure 8.
Figure 9.
### Table 1

Non-dimensional partially distributed buckling load of semi-infinite plate for different discretizations of the plate*

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Non-dimensional buckling load \((\lambda) = \frac{P}{k^3}\)

S symmetric mode about the centre line.

A anti-symmetric mode about the centre line.

5.0625 simply supported edges.

5.0615 clamped edges.

\(h=0.0254\ m, b=0.1524\ m, b/k=0.355, \nu=0.3, k=9.126\ kN/m^3\)

\(E = 206.700\ kN/m^2\)
Table 2
Non-dimensional partially distributed buckling load of a semi-infinite plate for different \( b/l \) ratio

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\* \( \lambda = \frac{P}{kz^3} \)
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