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THE ENVELOPE WAVE SPECTRUM

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Honolulu, Hawaii
United States

ABSTRACT

The envelope wave spectrum for constant wind speed would be that which envelopes all wave spectra from early wave generation to the fully-developed sea. The generally accepted form of the wave spectrum for the high frequency end of the spectrum is \( f^{-5} \) or \( T^3 \). The "overshoot" at high frequencies, say from 1 to more seconds period, could contain several times magnitude more energy than the fully-developed sea spectrum. The low frequency end of the spectrum has a term \( e^{-f} \) or \( e^{-T} \). In the analytical forms of any spectrum, for strong wind speeds, there will be wave periods that cannot exist because of fetch and duration limitations on this earth. There are two important hazards to consider prior to using the fully-developed sea spectrum for design: (1) If the structure is responsive to high frequency waves, then the structure will be under-designed, and (2) If the structure is responsive to low frequency waves, then the structure will have been designed for long wave periods that do not really exist. The envelope spectrum takes into account the above difficulties.
INTRODUCTION

The envelope wave spectrum for constant wind speed would be that which envelopes all the wave spectra from early wave generation to the fully-developed sea. Figure 1 is an example of the envelope spectrum using the individual spectra from Barnett (1972). The generally accepted form of the wave spectrum for high frequency end is $f^{-5}$ or $(T^3)$. This is true for both the Bretschneider (1959) and the Pierson-Moskowitz (1964) spectrums. As a matter of fact, in the normalized form, both of these spectrums are identical. In the young or non-fully developed seas, they both still have the energy proportional to $f^{-5}$ or $(T^3)$.  The difference is that the Pierson-Moskowitz spectrums is nestled for growing seas, whereas the Bretschneider spectrum has an "overshoot" for the growing seas. The "overshoot" at high frequencies, say from 1 to more seconds period, could contain several times more energy than the fully-developed sea spectrum.

The low frequency end of the spectrum has a term $e^{-T^n}$ or $(e^{-T^n})$. In the analytical forms of either spectrum, for strong wind speeds, there will be wave periods that cannot exist because of the fetch and duration limitations on this earth.

There are two important hazards in using the fully-developed sea spectrum for design, and this goes for both the Pierson-Moskowitz spectrum and the Bretschneider spectrum. Using the fully-developed sea spectrum, these hazards are as follows:

1. Structural hazard: If the structure is responsive to high frequency waves, then the structure will be under-designed.

2. Economic hazard: If the structure is responsive to low frequency waves, then the structure will have been designed for long wave periods that do not really exist.

The envelope spectrum accounts for the greater amount of energy at high frequency (low wave periods) during the initial generation (short fetch and duration of wind) than for a fully-developed sea. The envelope spectrum also eliminates or reduces the energy at low frequency (long wave periods) that cannot possibly exist on this earth.

For example, using a 30-knot wind and forecasting relations, it was found out that for high frequency, the energy of the envelope spectrum is proportional to $f^{-4.65}$ instead of $f^{-5}$. This is an important finding in view of the fact that some authors are wishing to abandon the $f^{-5}$ law. The $f^{-5}$ law still applies to any particular instantaneous spectrum, but the $f^{-(m+1)}$ law applies to the envelope of the experiments, where $(m+1)$ is less than 5 as has been found by other investigators.

Finally using the Bretschneider (1970) wave forecasting relationships, reduced to high wind speeds and short fetches, it is found for the envelope spectrum for $U = 30$ knots for high frequency waves that the energy is proportional to $f_{-4.36}$.

THE NORMALIZED SPECTRUM

The normalized period and frequency spectrums of Bretschneider (1959) and Pierson-Moskowitz (1964) are given respectively by

$$S(\nu) = 4\nu^{-5} e^{-\nu^{-4}}$$

(1)
\[
\frac{S(f)}{S(f_o)} = e^{5/4 (f/f_o)^{-5}} e^{-5/4 (1/f_o)^{-4}}
\]

\[
S(f_o) = 0.605 \text{ m}^2 \text{ sec}
\]

\[
f_o = 0.23
\]

FIGURE 1 Spectra of Waves Generated in North Sea by Winds Offshore Island of Sylt, Germany, on 15 September 1968 During Joint Sea Wave Project (Barnett, 1972) with Envelope Spectrum Added
During the past few decades much progress has been made in defining the wave spectrum, but there has been some misunderstanding on the proper interpretation of the various forms presented. Figures 2 and 3 show the generally accepted forms of the normalized frequency and period spectra respectively, in terms of significant wave height and period.

Figures 4 and 5 show an alternative form of figures 2 and 3 respectively, where \( S(f_0) \) and \( f_0^{-1} \) represents maximum energy density and the corresponding period, respectively.

The area under the normal spectrum in each case is equal to unity, i.e.:

\[
\int_0^\infty S(\tau) \, d\tau = - \int_0^\infty S(\nu) \, d\nu = 1
\]

The area under the alternative form is close to 0.7.

**STANDARDIZED SPECTRUM**

The standardized forms of the frequency and period spectrum of Bretschneider (1959) are given respectively by

\[
S(f) = \frac{H_s^2}{T_s} S(\nu) \tag{4}
\]

\[
S(T) = \frac{H_s^2}{T_s} S(\tau) \tag{5}
\]

The area under the standardized form is \( H_s^2 \), i.e.

\[
\int_0^\infty S(T) \, dT = - \int_0^\infty S(f) \, df = H_s^2 = 16 (1/4 H_s)^2
\]

Equation 6 gives a term 16 times the total variance, since \((1/4 H_s)^2\) is equal to the total variance.

**THE STANDARDIZED BRETSCHNEIDER SPECTRUM**

The standardized spectrums (Bretschneider, 1959) for period and frequency respectively from equations 1, 2, 4 and 5 are as follows:

\[
S(f) = 4 \left( \frac{H_s}{T_s^2} \right)^2 f^{-5} e^{- \left( \frac{1}{f T_s} \right)^5} \tag{7}
\]

\[
S(T) = 4 \left( \frac{H_s}{T_s^2} \right)^2 T^3 e^{- (T/T_s)^4} \tag{8}
\]
\[ S(\nu) = 4\nu^4 e^{-\nu^2} \]

\[ \nu = \frac{1}{T_0} \int S(\nu) d\nu = 1.0 \]

\[ S(T) = \int S(\nu) \frac{d\nu}{H_0^2} [S(T)] \]

\[ S(T) = T^2 e^{-T^2} \]

\[ \tau = \frac{1}{T_0} \int S(T) d\tau = 1.0 \]

\[ S(T) = H_0^2 T_0^2 [S(T)] \]

\[ \int S(T) d\tau = H_0^2 \]

\[ \nu = \frac{1}{T_0} \]
NON-DIMENSIONAL FREQUENCY SPECTRUM

\[ S(f) = \frac{1}{4} H_\alpha^2 f_0^{-1} \left( \frac{f}{f_0} \right)^{-3} e^{-3/4 \left( \frac{f}{f_0} \right)^4} \]

\[ S(f_0) = 1 \]

FIGURE 4 NON-DIMENSIONAL WAVE FREQUENCY SPECTRUM

![Non-dimensional frequency spectrum graph]

<table>
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<tr>
<th>f/f_0</th>
<th>S(f)/S(f_0)</th>
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NON-DIMENSIONAL PERIOD SPECTRUM

\[ S(T) = \frac{1}{4} H_\alpha^2 T_0^{-1} \left( \frac{T}{T_0} \right)^{-3/4} e^{-3/4 \left( \frac{T}{T_0} \right)^4} \]

\[ S(T_0) = 1 \]

FIGURE 5 NON-DIMENSIONAL WAVE PERIOD SPECTRUM

![Non-dimensional period spectrum graph]

<table>
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Bretschneider 6
Equations 1 and 2, and equations 7 and 8 are written for ease of memory. The area under the curves described by equations 7 and 8 is equal to $H_s^2$, as defined by equation 6.

FULLY DEVELOPED SEA SPECTRUM

Except for low wind speeds, the fully-developed wave spectrum cannot be achieved for high wind speeds, because fetches and wind durations for high wind speeds are not sufficiently long on the Planet Earth.

Various authors have thought or assumed that the fully developed sea spectrum was an envelope of the young seas, i.e. the family of spectra were nestled, and there were no overshoots.

Other authors recognized that there was in reality an overshoot and as time and distance increased after the overshoot, the energy at high frequency dropped to a lower level than the peak, probably through breaking and transfer of energy of the high frequency to the lower frequencies.

As shown by Bretschneider (1974) both the Bretschneider spectrum for all seas and the Pierson-Moskowitz spectrum for fully-developed seas can be represented by use of the Weibull (1951) distribution function.

THE ENVELOPE SPECTRUM

The envelope spectrum would be that for the overshoots at all frequencies. This can best be demonstrated by use of the latest wave forecasting relations given by Bretschneider (1970) which are fetch and duration limited, although other similar relationships could also have been used, for example, Wilson (1955) and also the original Sverdrup-Munk (1947) relations could demonstrate the same point.

Wave forecasting relations for deep water according to various authors (Bretschneider, 1970; Wilson, 1955; Bretschneider and Putz, 1951) can be represented approximately as follows:

\[
\frac{gH_s}{U^2} = A_1 \tanh \left[ B_1 \left( \frac{gF}{U^2} \right)^{m_1} \right]
\]
\[
\frac{gT_s}{2\pi U} = A_2 \tanh \left[ B_2 \left( \frac{gF}{U^2} \right)^{m_2} \right]
\]
\[
\frac{gF}{U} = 2 \int_0^{gF/U^2} \left\{ A_2 \tanh \left[ B_2 \left( \frac{gF}{U^2} \right)^{m_2} \right] \right\} \frac{1}{\delta \left( \frac{gF}{U^2} \right)} \, d\left( \frac{gF}{U^2} \right)
\]

where $A_1, A_2, B_1, B_2, m_1$ and $m_2$ are chosen according to data and are different according to different analysis of the various cited authors.
Substituting equations 9 and 10 into equations 7 and 8, one obtains

\[ S(f) = K f^{-5} e^{-\frac{f}{T_s}} \]  
and

\[ S(T) = K T^3 e^{-\frac{T}{T_s}} \]

(12)

where

\[ K = 4 \left\{ \frac{A_1 g \tanh \left[ B_1 \left( \frac{gF}{U^2} \right) \right]}{2 \pi A_2 \tanh \left[ B_2 \left( \frac{gF}{U^2} \right) \right]} \right\}^2 \]  

(13)

A similar expression could be written in terms of \( \frac{gt}{U} \), where \( t \) is the minimum duration of the wind and \( K \) is for minimum fetch length as governed by either actual fetch length or minimum wind duration.

**EXAMPLE FOR \( U = 30 \) KNOTS AND THE THIRD REFERENCE**

It will be convenient for practical use to put equations 16, 17 and 21 in terms of \( T_s = \) seconds, \( U = 30 \) knots, \( g = 32.16 \) ft. sec.\(^{-2} \), \( F = \) nautical miles, and the coefficients according to Pierson-Moskowitz (1964) are as follows: \( A_1 = 0.283, B_1 = 0.0125, m_1 = 0.42, A_2 = 1.2, B_2 = 0.077 \) and \( m_2 = 0.25 \). This results in the reduced equations as follows:

\[ H_s = 22.59 \left\{ \tanh \left[ 0.0771 F^{0.42} \right] \right\} \]  

(15)

\[ T_s = 11.9 \left\{ \tanh \left[ 0.227 F^{0.25} \right] \right\} \]  

(16)

\[ K = 0.102 \left\{ \frac{\tanh \left( 0.0771 F^{0.42} \right)}{\tanh \left( 0.227 F^{0.25} \right)} \right\}^2 \]  

(17)

The problem now is to select various values of \( F = 0.1 \) to 10,000 or more nautical miles and then calculate \( T_s \), then calculate \( K \), and then select various values of \( f \).

The above equations can be used to calculate \( S(f) \) and \( S(T) \) according to equations 12 and 13 respectively.
Figure 6 shows the growth of energy for various wave periods for $U = 30$ knots versus fetch length in nautical miles. The "overshoot" is apparent from figure 6, but also important is the fact that large wave periods cannot be generated for short fetch lengths.

The peaks of the curves of figure 6, that is the envelope spectrum have been used to establish the Weibull distribution, as shown in figure 7.

Finally, figures 8 and 9 show the standard enveloped period and frequency spectra, respectively for $U = 30$ knots. It should be noted that the area under the envelope spectrum will be greater than that for the fully-developed sea spectra. However, the use of this type spectra must neglect many long wave periods that cannot exist on the Planet Earth.

ANALYTICAL DEVELOPMENT FOR HIGH FREQUENCY END OF ENVELOPE SPECTRUM

Returning to equations 9, 10, 12 and 13 for high wind speeds and short fetches, one obtains

\[ \frac{g T_s}{2\pi U} = A \frac{B^2}{2} \left( \frac{gF}{U^2} \right)^m \]

and

\[ K = A g^2 \frac{(A^2 B^2)^{2}}{(2\pi A B)^2} \frac{2(m_1 - m_2)}{U} \]

For the present we will not use any particular values for $A$, $B$, and $m$, because according to various authors (Bretschneider, 1970; Wilson, 1955; Sverdrup-Munk, 1947; Bretschneider-Putz, 1951), these values are different. A summary of these values are given in Table I.

Also, as a matter of fact, these coefficient values could change for each author over various reaches of $gF/U^2$.

Now we will let

\[ T_s = A^2 \frac{m^2}{3} \]

and

\[ K = B^2 \frac{2(m_1 - m_2)}{3} \]

where

\[ A = \frac{2\pi U}{g} A \frac{B^2}{2} \left( \frac{gF}{U^2} \right)^m \]

and

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FIGURE 6 GROWTH OF WAVE SPECTRUM WITH FETCH DISTANCE, UNLIMITED WIND DURATION, FOR U = 30 KNOTS

-公式-

\( S(\tau) = 3.66 \tau^{4.06} e^{-\gamma^{2.6}} \)

\( S(\nu) = 3.65 \psi^{-4.05} e^{-\psi^{2.6}} \)

\( \psi = \frac{T}{T_b} \)

\( \nu = \frac{T}{T_b} \)

\( Y = -0.893 + 3.649X \)

\( \rho = 0.9998 \)

**FIGURE 7** THE NORMALIZED ENVELOPE PERIOD SPECTRUM BASED ON WEIBULL DISTRIBUTION FUNCTION FOR U = 30 KNOTS
Figure 8: The envelope frequency spectrum for \( U = 30 \) knots for \( f = 0.05 \) to \( 0.19 \) sec\(^{-1}\)

Figure 9: The envelope period spectrum and period spectrum for unlimited fetch and wind duration for \( U = 30 \) knots.
\[ B_3 = 4 \frac{g^2 (A_1 B_1)^2}{(2\pi A_2 B_2)^4} \left( \frac{g}{u^2} \right)^{2(m_1 - 2m_2)} \]  

whence equation 12 becomes

\[ S_p(f) = B_3 f^{-5} F^2(m_1 - 2m_2) e^{-\left(\frac{2m}{m_2}\right) - \left(\frac{2m}{m_1}\right) f} \]  

In order to find the fetch length \( F = F_\star \) for any value of \( f = f_\star \) that make \( S_p(f) = S_{p \star}(f_\star) = \text{max} \), we perform the following operation

\[ \frac{d [S_p(f)]}{d F} = 0 \]  

It then follows that

\[ S_{p \star}(f_\star) = B_3 e^{-\left(\frac{2m}{m_2}\right) f_\star} \left(\frac{2m}{m_2}\right)^{\frac{2m}{m_2} - \left(1 + \frac{2m}{m_2}\right)} \]  

\[ F_\star = \left[ \frac{2m}{m_2 - m_1} \right]^{\frac{1}{4m_2}} (A_3 f_\star) - \frac{1}{m_2} \]  

It should be noted from equation 27 that \( 2m_2 \geq m_1 \), otherwise the solution is imaginary and indeterminant.

Using the expressions for \( A_3 \) and \( B_3 \) (eqs. 22 and 23), we can write equations 26 and 27 as follows:

\[ S_{p \star}(f_\star) = K_\star \left[ \frac{8}{2 \pi U f_\star} \right] \]
\[
\left( \frac{g}{2 \pi U f_*} \right) = \left[ \frac{gF_*}{B_* U^2} \right]^{m_2}
\]

where

\[
S_{F_*}(f_*) = K_* \left[ \frac{gF_*}{B_* U^2} \right]^{m_2 + 2m_1}
\]

(29)

\[
K_* g^3 \frac{1}{U^5} = 8\pi \frac{(A_1 B_1)^2 (A_2 B_2)}{} \left[ \frac{1}{e} \left( 1 - \frac{1}{2} \frac{m_1}{m_2} \right) \right]^{1/4}
\]

(31)

and

\[
B_* = \begin{bmatrix} \frac{2m_2}{4m_2} \\ \frac{2m_2 - m_1}{m_2} \end{bmatrix} \begin{bmatrix} \frac{1}{4m_2} \\ \frac{1}{A_2 B_2} \end{bmatrix}
\]

(32)

When \( m_1 = 2m_2 \)

\[
K_* g^3 \frac{1}{U^5} = 8\pi \frac{(A_1 B_1)^2 (A_2 B_2)}{}^{-1/4}
\]

(33)

\[ B_* = \infty \text{ or } \frac{1}{B_*} = 0 \]. This means \( \frac{H_s}{T_s^2} = \text{constant} \).

Table I shows calculated values for \( A, B, m, \) etc. and \( K_* \) and \( B_* \) based upon the various wave forecasting relations.
<table>
<thead>
<tr>
<th>Reference Date</th>
<th>8 1947</th>
<th>9 1951</th>
<th>5 1955</th>
<th>2 1959</th>
<th>4 1970</th>
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<td>A_1</td>
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<td>0.260</td>
<td>0.260</td>
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<td>0.0684</td>
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<td>0.077</td>
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<tr>
<td>A_2B_2</td>
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<td>0.093</td>
<td>0.061</td>
<td>~0.07</td>
<td>0.0924</td>
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<tr>
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<td>—</td>
<td>.02376</td>
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**NOTE:** — approximate
REFERENCES


Artificial Island Construction in the Shallow Beaufort Sea

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Canada

Abstract

Oil and gas exploration has progressed from the Mackenzie Delta area of northern Canada into the shallow waters of the southern Beaufort Sea. During the past two years, two companies have built seven artificial drilling islands. Currents, wave actions and potential ice forces were considered in the design stages. The islands were instrumented and monitored for movement as well as for frost penetration. Data obtained during the first winter is presented.

A successful operation in the area demands careful planning and lead times of up to one and a half years. Water, air and road travel are seasonal and if pre-planning is poor, essential equipment and supplies may not be on site in time. Environmental concerns are equally as important. Much of the area is within the Kendall Island Bird Sanctuary, an area where several North American bird species come annually to nest. The northern residents also depend to a large extent on the spring whale hunt. The effect of the seasonality and environment are discussed as part of the logistics of operations.
INTRODUCTION

Oil and gas exploration has progressed from the Mackenzie Delta area of northern Canada into the waters of the southern Beaufort Sea. (Fig. 1 and 2) It has been demonstrated that in these shallow waters, drilling operations conducted from artificial islands are both feasible and economical. Sun Oil Company Limited has built two such islands and conceivably will build several more in the future. Unark L-24 and Pelly B-35 are completely different in design and it is interesting to note that of the seven artificial islands built to date, no two are alike.

Figure 1. Inuvik is the focal point of the exploration activity.

Figure 2. Map of East Mackenzie Bay illustrating artificial island locations.
GENERAL CLIMATIC CONDITIONS

The Mackenzie Bay area is characterized by mean daily temperatures ranging from 12°C in July to -29°C in January. Annual precipitations is low at 12 - 25 cm (5" - 10"), most of which falls as rain in August. Summer winds are moderate at 16 - 24 km/h (10 - 15 m.p.h.) but strong northeast/northwest winds in winter can be severe.

Fog conditions often occur during the open water season of July through September and the shallow waters remain frozen throughout the remainder of the year. Complete freezing usually occurs in waters less than 1.8 m (6') deep. Numerous ice pressure ridges occur in mid-bay, probably the result of the shallow waters.

Water temperatures rise from 1.2°C in early July to 18.5°C by the end of the month, due to the large volumes of water discharging from the Mackenzie River. By September the temperature is back down to 2.1°C with freezing occurring towards the end of the month.

Salinity also seems to be affected by the spring run-off of the Mackenzie and by freezing in the winter. Salinity is low in the spring, as would be expected, increases slightly by late August and early September and then decreases again as winter progresses. It is not uncommon to be able to drink the water with only minimal treatment during winter months.

Unark L-24

Site Investigation

Sun Oil's first Delta wildcat was selected for a site 4.8 km (3 miles) offshore in 1 - 1.2 m (3 1/2' - 4') of water. Winter construction was planned for this location so that it would be ready for rig up in the summer of 1974 and subsequent drilling to start in the early fall. The shallow water allowed for total freezing by January which was a significant plus for the construction crews, since they did not have to contend with free water.

In January of 1974 a field crew was sent out to obtain information relative to the seabed materials which would form the foundation for the gravel fill island. From a borehole survey it was found that a continuous permafrost layer existed 3.7 m (12') below the sea bed. The "active layer" was made up of a non-plastic silt with a trace of fine sand. The silt was firm to stiff and judged to be capable of supporting the overburden weight of the gravel fill plus the drilling rig, ancillary equipment and drilling supplies without fear of a bearing capacity failure or excessive consolidation.

Extensive data on storm surges, tides, currents etc. was not available but a review of available records was useful in preliminary design. Tides in the area do not exceed 0.3 m (1') which is significant to mariners since, in places, it is 30% of the water depth, but not particularly crucial to the performance of an artificial island.

Storms in the area or further out in the Beaufort Sea have raised the water levels up to 2.6 m (8 1/2') and with the waves on top of that, water at the 3.4 m (11') mark could be expected at the Unark site.

An examination of ERTS photographs revealed surface currents determined by prevailing winds. A counter clockwise gyre was produced due to prevailing NE winds and the lee of Richard's Island Peninsula. Site investigations and available literature
confirmed that neither surface currents nor subsea currents existed to any extent and so erosion of the island fill from these sources would not be a problem.

**Design**

Due to the logistics involved in moving in the rig, setting it up and drilling the well, it became necessary that the island remain as a competent structure throughout one full year.

Storm surge data indicated that water levels up to 3.4 m (11') could occur and after considering the probabilities and risks involved, a surface elevation of 3.7 m (12') above sea bed was selected.

Gravel was selected as the most practical economical fill material. The unavailability of suitable fill material locally necessitated that gravel be transported from quarries at Ya Ya Lakes, some 72 km (45 miles) south. The Ya Ya gravel was generally clean, well graded (100% passing 50.8 mm (2") sieve and less than 5% passing No. 74 sieve) and of low moisture content, making it ideal for winter construction. Winter construction was particularly suited for this since roads are plowed on the Mackenzie River and trucks of 18.9 t (42,000 lbs.) G.V.W. capacity can easily move on them. Borehole surveys and laboratory analysis showed that there would be sufficient bearing capacity to support the expected surcharge of 62.7 kPa (1310 p.s.f.)

Possible ice pressures exerted on the island are exceedingly difficult to forecast since they depend on the mechanism of failure. Surface dimensions of 61 m x 122 m (200' x 400') were chosen for the purposes of accommodating the rig and its support equipment. Shear strength analysis indicated that using Ya Ya's gravel the island could withstand ice pressures of 3,445 kPa (500 psi) with no problems.

The island was orientated in a N-S direction so as to expose as little as possible to the NE and NW winds. The sides of the island were sloped 1 : 3 (see Fig. 3) so as to induce the advancing ice sheet to fail in bending and prevent the full compressive force from being developed. This concept was sound but problems were experienced during the first days of freeze-up when winds caused thin sheets 2 - 3 cm (1") to ride up the slope and pile up against buildings, stockpiles, etc. A somewhat steeper slope (1 : 2) would be advisable for future islands.

Ice action was not a major concern for Unark since, with the shallow water, large floes moving in from the Beaufort Sea would be quickly grounded and with the ice around the island being bottomfast, it could do little damage. Water was the main concern since it would have one full summer to work away on the fill material. Having selected a 3.7 m (12') surface elevation, the sides then needed protection. Filter cloth, similar to that used for road stabilization and drainage works, was laid down on the side slopes. Sandbags containing 5% dry Portland cement by weight were laid in three layers thick on top of the filter cloth. The cloth then, would prevent the fines from washing out and the sandbags, as well as holding the cloth in place, served to break the action of the waves. The cement was added to the sand at the time the bags were filled. The intention being that after the sandbags were placed on the slope, the cement would get wet, set up and help to hold the bags together as a unit. The other consideration was that should the jute material wear thin, the cement would hold the contents intact. To tie everything together chain link fencing was laid overtop of the bags and anchored at the toe and the top of the island.

There existed a potential for consolidation of the foundation soil and of the fill as well as thaw subsidence. As mentioned previously, analysis indicated consolidation
Figure 3.

UNARK DRILLING ISLAND
SECTIONAL VIEW

- Ice Sheet
- Chain Link Rolls
- Sandbags
- Filter Cloth
- Gravel Fill

Sea Bed

+1.2 m
of the foundation soils would be minimal. Settlement of the fill was also judged to be minimal due to the low moisture content and good compaction properties of the gravel. Thaw subsidence resulting from heat generated by the drilling rig and exposure to the summer sun was the other possibility. To reduce the problems resulting from any differential settlement, the critical areas of the rig were provided with steel piles extending 6.1 m (20') into the sea bed.

An instrumentation program was instituted:

1. To determine what effect the placement of fill would have on the underlying permafrost (using thermistors).

2. To ascertain if significant consolidation settlement of sea bed soils occurred under the weight of the fill (using settlement sensors).

3. To determine if lateral displacement of the fill occurred due to ice action (using inclinometer tubing).

Island Construction and Performance

By the time construction began in February 1974, the water was frozen to bottom. The absence of free water allowed crews to excavate the ice by means of ripper-equipped bulldozers and front-end loaders. The dozers ripped the ice, pushed it into piles and the front-end loaders moved it out and away. Gravel was trucked from Ya Ya at 1,528 cubic metres (2,000 cubic yards) per day and spread. As the bulldozers and loaders worked, the gravel was compacted well enough that very little consolidation took place during the spring and summer months. Basically, the island was built up in three 1.2 m (4') lifts. As each lift was completed sandbags were placed on the slope on top of the filter cloth and the chain link rolled up. From the time that the equipment was first moved on to site until the finishing touches were put on, some seven weeks had elapsed. (Fig. 4)
Very simply stated, the performance of this island has been excellent. The sandbags have stayed on the side slopes remarkably well due to the cement content and the careful manner in which they were originally hand-placed. At places, near the waterline, the jute has worn through, but the sand-cement remains intact.

As mentioned earlier, the fill material showed only minimal consolidation and the sea bed sensors indicate that there was insignificant settlement of the foundation material. The temperature profile illustrated in Figure 5 shows that there has been no degradation of the permafrost and that, in fact, the continuous permafrost table has been elevated. This growth undoubtedly is the reason for the insignificant settlement observed to date.

During the summer of 1974, water levels (including wave run-up) were observed up to the 3 m (10') mark, whilst at other times we were able to walk on the natural sea bed without getting our feet wet.

Ice presented no problems other than the thin sheets running up the slopes as mentioned earlier. The inclinometer data showed no lateral movements other than some subsidence of the fill at the corners, due to differential thawing.

Unark island has existed for nearly one and a half years now and could remain intact for several more to come.

Pelly B-35

Site Investigation

The Pelly B-35 location was in 2.1 m (7') of water. Concurrent with the borehole work at Unark L-24, investigations were carried out at Pelly. The results of the work were similar to those of Unark except that permafrost was not encountered in any of the 24.4 m (80') test holes. It was, in fact, not until the 183 m (600') level of the drilling operation that it was found.

The shallow water, limited fetch, and the shelter of neighboring islands, led to a maximum design water level of 4.3 m (14').

Design

Construction of Pelly was slated for the summer of 1974. The transporting of gravel in sufficient quantities to sustain construction crews was a risky proposition in the shallow waters of Mackenzie Bay. The use of the silt sea bed material as fill material had been employed successfully on previous occasions and it was decided to use silt for the Pelly site. The silt, however, would not support a drilling rig and in an attempt to save time and provide a stable platform for the drilling operation, two railroad barges were purchased. These barges were to be tied together with a superstructure and the rig assembled on it in Inuvik. The barge would then be moved out to the site and set in the centre of the island. The barge was 2.4 m (8') in height and the superstructure added another 1.8 m (6') of elevation so that adequate freeboard could be provided. Once the barge was in place, the compartments were to be flooded with seawater and the barge grounded. This left a freeboard of 2.1 m (7') and the construction of the island could commence.

The sizing of the island was computed by analyzing the shear stresses imposed by ice pressures of up to 3,445 kPa (500 psi). Laboratory analysis of remolded silt provided a range of shear strengths varying with normal pressure (i.e. weight of fill). It was assumed that the barge would not contribute towards the shear strength of the island and that it would merely transfer the shear stresses. Two failure modes were
THERMISTOR STRING T3
UNARK ISLAND
TEMPERATURE (°F & °C)

Figure 5.
investigated, one being the fill itself and the other being through the foundation materials. The maximum forces would develop in January-February when the ice reached a thickness of 1.8 m (6'). To further enhance the shear strength of the silt, it was decided to promote frost penetration. By limiting the height of fill the frost action would achieve deeper penetration earlier in the season and it was judged worthwhile to sacrifice a few pounds of normal pressure. After considering these factors, an island 83 m x 157 m x 2.4 m (270' x 515' x 8') was developed. The minimal freeboard of 0.3 m (1') was acceptable since the drilling rig itself was at 2.1 m (+ 7'). Construction crews would work up to freeze-up in September so that any damage to the island by storms could be easily repaired.

A berm concept was developed to form a perimeter to retain the silt and protect the island from the erosive effects of water and ice. Gabions made up of wire mesh and sandbags and measuring 1.5 m x 1.5 m x 0.75 m (5' x 5' x 30") were to be stacked in pyramid fashion. The original configuration was altered somewhat, due to the possible bearing capacity failure of the sea bed under this 38.3 kPa (800 psf) line load. (Fig. 6)

The island orientation was selected so as to expose the smallest area to the prevailing NNE-SSW winds.

Island Construction and Performance

The gabions were built at the Ya Ya gravel quarry during the spring of 1974. Once the corners of the proposed island were located by survey crews in July, the gabions were moved out to site by barge. Using a crane located on a spud barge, the gabions were placed in the water. Crews quickly caught on to the placement of the gabions by "feel" since, for the most part, they were out of sight in the muddy water. The gabion construction continued until all but one end was complete and the drill barge was moved out from Inuvik. This allowed crews to complete rigging up while the gabion berm would break the waves and shelter the drill barge. Once the barge was in place and grounded, the south end of the berm was closed off. Silt was then placed in the area between the barge and the berm by means of clamshell. The clamming operation was restricted to an area 15.2 m (50') out from the berm and a slope of 1:3 maintained so as to prevent a failure at the toe of the berm.

From start to finish, the construction lasted some seven weeks including lost time due to low water and fog.

Instrumentation was installed to record the various design parameters and provide warning should the design conditions be exceeded. Inclinometer tubing with alarm modules were installed to provide a record of lateral movements and alarms for movements in excess of 10 cm (4"). The plot illustrated in Figure 7 shows typical movements to May 1975. It is interesting to note how the frozen silt floated on top of the unfrozen material. This movement can be correlated with storm conditions in some instances. These movements were preserved in the island as the frost zone progressed downwards. The "kinks" on the plot reflect these permanent records.

Thermistor strings were installed to monitor frost penetration. The data obtained was similar to that recorded at Unark L-24. As could be expected, areas covered by snow showed slower penetration. The southeast end of the island was kept snow free by the winds and the frost was 0.6 m (2') lower than in other areas. Future silt islands may be kept clear of snow to take advantage of this.

Ice movement around the island was monitored by conventional surveying techniques and although the results may not have been as precise as one may wish, movements in the order of 3.7 m - 4.3 m (12' - 13') were observed.
Figure 6.

PELLY DRILLING ISLAND

SECTION AA

SECTION BB

2.1m WATER

2.1-2.4m SILT

2.1-2.4m SILT

DREDGE Silt
Figure 7.

INCLINOMETER RECORD
PELLY B-35
HOLE # 5

DEFLECTION IN INCHES & CENTIMETERS

(NORTH)

(IN) -1.0 -0.8 -0.6 -0.4 -0.2 0 0 +0.2 +0.4 +0.6 +0.8 +1.0

(SOUTH)

(IN) -1.0 -0.8 -0.6 -0.4 -0.2 0 0 +0.2 +0.4 +0.6 +0.8 +1.0

(INCHES)

CENTIMETERS

INITIAL 04/11/74

05/01/75

12/03/75

05/05/75

NORTH SOUTH

DEPTBelow ISLAND SURFACE

(FM) 30 20 10 4 2 0

(M) (FT) 30 20 10 4 2 0
Very early in the season, as the first permanent sheet formed in the area, ice ridges appeared at the Pelly B-35 location. Figure 9 shows that the ridges seemed to build up roughly parallel to the sides of the island. Not having had an opportunity to observe this effect previously, we can only speculate that the...

Figure 9. Location of Pressure Ridges in the Eastern Mackenzie Bay, Winter 1974/75.
islands do have an effect on the ice field. If, in fact, this is a common phenomena and since the ridge comprises a weak spot in the ice providing a mechanism to redistribute movement around the islands, then perhaps future island designs should take this into account.

Ice pressure data was obtained from stress transducers set in the ice sheet. The data obtained has not been finalized as yet, but pressure less than 7.2 kPa (150 psi) was observed. The ice movement data was not recorded in real time but some correlation does appear to exist between pressure variations and ice movement.

During the summer of 1975, the drilling barge will be removed from the centre of the island and the elements will be allowed to work away at the silt fill, thus destroying the island.

LOGISTICS

Pre-planning is among the most critical of functions for both the construction of the island and the drilling of the well. The lead time required to obtain the necessary supplies and ensure that they are at the site when required is the biggest operational problem.

The most economical mode of transportation for the heavy, bulky materials and indeed, all materials, is by barge. The freight moves by rail and truck from southern centres to Hay River, N.W.T. Commencing in mid-June, it is transferred to barges and moved up the Mackenzie River following the ice as it retreats northward. Normally four trips are possible before the ice begins to form again in mid-September. If your equipment should miss the barge run, you must either fly it in or do without.

Once the materials and supplies reach the Delta area they are either stockpiled or relayed on to the job site. During the open water periods, barges are again useful, except that in many cases, the shallow waters limit their capacity and impede their progress. As an example, we waited nearly two weeks for the water levels to increase so as the crane and clamshell could be moved over to Pelly B-35. Helicopters and hovercraft are normally quite busy but they, too, can only operate under near-ideal climatic conditions. In the wintertime, the ice is sufficiently thick to support large trucks. Roads are plowed to various sites and, at times, from the number of vehicles, you would think you were on a major southern freeway.

Environmental restraints also play an important role in the logistics of the area. Each summer the beluga whales move through the Mackenzie Bay and all water traffic must be suspended for the first half of July to allow the whales to pass by. The Mackenzie Delta is an important nesting ground for the North American bird population and many areas are closed off for weeks at a time to protect the birds.

CONCLUSION

The construction of these temporary drilling islands has proven to be a viable method for the exploration activities in the shallow waters of the Beaufort Sea. The eventual design construction of permanent production platforms will benefit from the techniques and information obtained from these temporary exploration islands.
NORTH SEA OFFSHORE STRUCTURES

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The Norwegian Institute of Technology
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ABSTRACT

Existing and planned North Sea offshore structures for exploration (drillings) and structures for exploitation (production platforms) are mentioned. Introductorily the meteorological conditions, wave climate and bottom composition in the North Sea are reviewed briefly. They are determining factors for design as well as for operation. Furthermore, an account on present and planned activities on exploration for and exploitation of oil and gas in the North Sea is given.

NOTE: The author has requested this paper be read in conjunction with the related papers, "Safety Verification of North Sea Structures Practices and Trends", by J. Eri et al (p. 791) and "North Sea Offshore Structures Environment and Environmental Loads", by B. Pedersen (p. 875).
The North Sea is known to be one of the most hostile seas in the world combining high winds, high seas, low temperatures and often low visibility too. This situation is aggravated further by its hydrography exposing general shallowness and large shallow banks. Only the northermost part along the Norwegian Coast has a relatively narrow channel, depths to about 300 m.

The wind and wave climate in the North Sea are sometimes compared to conditions in the Gulf of Mexico. Almost every year one or more hurricanes with sustained wind velocities exceeding 70 mph (110 km/h) and much higher gustiness will pass through the Gulf accompanied by waves up to 12 to 14 m. The North Sea on the other hand has rough weather almost year round apart from a few late spring and summer months. The winds in the greater part of the sea may then be easterly, while during the rest of the year they are westerly with the typical storm starting in SW, veering W and finally NW, often with increasing velocities beyond "hurricane limits" and generating waves approaching 20 m maximum with 15-18 sec periods in the northern part. Wave heights decrease southward. The shallow irregular bottom topography of the North Sea furthermore causes crossing wave orthogonals which on shallow banks like the Dogger Bank in the middle of the North Sea may result in large spouts where wave crests of high steepness meet and separate in breaking. A storm may last 2-4 days and another storm may follow "before the end of the week".

Fig. 1, (1) is a quaternary map, and gives information about the bottom material in the North Sea. It shows the design wave height of the fully developed 50-year storm lasting 12 hours or more and the corresponding 50-year wave periods (paper by the National Institute of Oceanography Great Britain 1972 on "Extreme wave conditions in British and adjacent waters"). Comparing the North Sea to all other seas it is probably a fact that it is about the most hostile sea in the world.

The question is: How are structures for exploitation of oil and gas able to meet the challenge? Can presently available experiences and designs by extrapolation be used or has the hostile environment to be met by new and also considerably more expensive developments? Out of numerous boundary conditions four seem to be of major importance. They are:

1) Quantities and location of oil and/or gas presumably available.
2) Possibilities for piping versus marine storage/transport.
3) The stability of the offshore structures in the environment.
4) The overall economy of the project all factors included.

As mentioned below the present trend in the North Sea is to increase piping of oil and gas to shore e.g. from the EKOFISK field to Germany and England and from the FORTIES field to Scotland (Fig. 2). Other possibilities for piping of gas or oil to Scotland and Norway are being investigated at this time and may become realities in a foreseeable future, as they seem to be the most economical.

ACTIVITIES IN THE NORTH SEA

Giant oil recoveries have been made during the past few years. Locations are shown in Fig. 2 with the names of these fields (4). Many technical innovations have been necessary in equipment designs for North Sea operations and the development in the North Sea has already to some extent in-
Figure 1. The North Sea: bottom condition and "design wave". (50 year storm).
Figure 2. North Sea Oil and Gas activities.
fluenced world-wide producing operations as companies also in other parts of the world move farther offshore into greater depths of waters, as e.g. in the Gulf of Mexico, West Africa, and in the Western Pacific. At this time (1975) about 40 jackups and semi-submersible rigs are operating in the North Sea, in water depths ranging upward to 525 feet.

Since the middle of 1973, four major oil discoveries have been made in Jurassic sands in the East Shetland trough, in the northern North Sea east and northeast of the Shetland Islands. Reserves of 1.2 billion barrels are estimated for the large NINIAN structure southwest of BRENT field on the basis of two wells completed in 1974, with production start in 1978.

Major extensions have been made to THISTLE, DUNLIN and PIPER, increasing proved reserves of these fields. These recoverable reserves are estimated at: THISTLE, (blocks 211/18 and 211/19), 750 million bbls, with production start in 1977; DUNLIN, (blocks 211/23 and 211/24), 400 million bbls with production start in 1977/78; PIPER (block 15/17) 642-800 million bbls with production start early 1976 and BRENT (blocks 211/29 and 3/4), 1.8 billion bbls oil, 3 trillion cb ft gas with production start in 1976/77.

The Norwegian government granted licenses on two blocks east of the BRENT field and just east of the median line of the North Sea to a group of companies, including a 50% interest held by the Norwegian "State Oil" company. The initial well on the newly awarded acreage, with Mobil as operator, is a major separate discovery 6 miles northeast of BRENT, now called STATFJORD. This field (Norwegian blocks 33/12 - 33/9 - UK 211/24) has reserves of 3000 million bbls oil and 100,000 mill m³ associated gas with production start in late 1977 - early '78.

In the Norwegian part of the North Sea several EKOFISK group fields have had major extensions, and a gas discovery has been made east of the FRIGG field. The HEIMDAL field has been extended, and a possible commercial oil discovery has been made in Phillips' BRISLING field.

In the Dutch part of the North Sea, there have been two gas discoveries, and a gas field found in previous years has been extended.

Oil production from the British sector of the North Sea is expected to reach 2.4 million bpd, and from the Norwegian Ekofisk group of fields about 1 million bpd by the early 1980s. Because of the difficult conditions for operation there were construction delays on all projects in 1973-1974, and none of the North Sea oil fields went on production until 1975, except for loadings to tankers on a limited basis from the EKOFISK and DAN fields of Norway and Denmark, and the ARGYLL field in the British area.

 Pipelines with diameters ranging from 32 in. to 36 in. for oil or gas have been completed between the EKOFSK field and Germany and between the FORTIES field and other fields to Scotland (2 and 3). A number of problems connected with the laying of pipes in deep water are still unsolved. Projects performed within the framework of the Deep Water Pipeline Project Committee, set up by the Norwegian Ministry of Industry have contributed significantly to the understanding and solving of these.

The concern regarding pipelines safety against fishing gears like trawl doors seems to have been somewhat exaggerated. Comprehensive tests have been undertaken in Norway. So far pipelines do not seem either to have been damaged by the migrating sand waves of up to about 10 meters height occurring mainly in the relatively shallower waters of the British, Dutch and Danish zones, where tidal and other currents run to 2-3 knots.
STRUCTURES FOR EXPLORATION (DRILL/RIGS)

The question which arises is: Which type rig-structure is the most suitable and economical under the given conditions considering initial costs, risks of damage, maintenance and depreciation?

This question may be answered simply by considering the type of structures which have been preferred.

The December 1975 issue of "Northern Offshore" includes a table called "North Sea Rig status". The situation, with respect to types of rigs and location, was as shown below:

<table>
<thead>
<tr>
<th>TABLE 1</th>
<th>NORTH SEA RIG STATUS DECEMBER 1975</th>
</tr>
</thead>
<tbody>
<tr>
<td>Country</td>
<td>type</td>
</tr>
<tr>
<td>UNITED KINGDOM</td>
<td>31 SS</td>
</tr>
<tr>
<td>NORWAY</td>
<td>7 SS</td>
</tr>
<tr>
<td>HOLLAND</td>
<td>8 JU</td>
</tr>
<tr>
<td>IRISH SEA</td>
<td>1 DS</td>
</tr>
</tbody>
</table>

SS = Semisubmersible/rig  
JU = Jack-up  
DS = Drillship

The impression left by this information is that semi-submersible and some jack-up rigs, both designed specifically for conditions in the North Sea, presently are ruling here.

It should be noted that jack-up platforms so far have mainly concentrated in the shallowest and relatively less-exposed southern part of the North Sea like the Dutch sector, while the semi-submersible type is more popular in the deepest (northern) part, most heavily exposed.

The interest in jack-up platforms has been temporarily increasing. This may be caused by the introduction of jack-up designs being competitive to semi-submersibles, up to perhaps 350-400 ft water depth. Another reason is that the energy shortage has made a number of shallow water oil and gas fields of lower yield profitable.

Table 2 shows the different rig types ordered in 1973 all over the world compared to 1974 (until April). Here the number of jack-ups show a decreasing tendency.

<table>
<thead>
<tr>
<th>TABLE 2</th>
<th>NUMBERS AND PERCENTAGES OF DIFFERENT RIG TYPES ORDERED IN 1973 IN THE WORLD COMPARED TO 1974 (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>JU</td>
</tr>
<tr>
<td>1973</td>
<td>26 (35%)</td>
</tr>
<tr>
<td>1974</td>
<td>10 (42%)</td>
</tr>
</tbody>
</table>

Table 3 gives a recent account of the "Rigs under construction due to work in the North Sea after delivery" (Northern Offshore, No. 3, 1975).
TABLE 3 RIGS UNDER CONSTRUCTION DUE TO WORK IN THE NORTH SEA AFTER DELIVERY (Northern Offshore, No. 3, 1975).

| Mars/’75 | Dixilyn/Godager Venture One SS-Pentagone Conoco 2 years |
| Mars/’75 | Offshore Co Chris Chenery SS-SCP III Mk2 Shell 5 yearsX |
| Mars/’75 | Lauritzen Danwood Ice DS-Convertien Phillips 2 yearsX |
| Apr/’75 | Penrod Penrod 71 SS Sun/2 years Transocean |
| Apr/’75 | Saipem Scarabeo III SS-Tripod Aquitaine 2 yearsX |
| May/’75 | Penrod Penrod 72 SS Hunt 2 years |
| Sep/’75 | Penrod Penrod 65 JU-3-legged Mesa 2 years |
| Jun/’75 | Rosschavet Ross Rig SS-R-3 Statoil 5 years |
| Jun/’75 | Atlantic Pacific Maerskdrill 1 JU-3-legged DUC/Gulf Long term |
| Sep/’75 | Kingsnorth Kingsnorth 1 SS-h-3 Conoco 2 years |
| Aug/’75 | Deep Sea Drilling Deep Sea SS-h-3 Saga 2 years |
| Jun/’75 | Rasmussen/GLOBAL Polyglomar Driller 2 SS-h-3 Hydro 5 years |
| Jul/’75 | Sedco Sedco 705 SS-X1700 Shell 5 years |
| Dec/’75 | Sedco Sedco 707 SS-X-700Shell 5 years |
| Feb/’76 | Offshore Beige Le Petrel DS-Pelican Elf 5 yearsX |
| Apr/’76 | Sedco Sedco 709 SS-X-700 Shell 5 yearsX |

X=including other offshore areas. Complied by R.S. Platou A/S-Offshore Division.

The number and percentages corresponding to Table 3 are distributed as shown in Table 4. Comparison is made with Table 3.

TABLE 4 COMPARISON BETWEEN TABLES 1 (1974) AND 3 (1975/76)

<table>
<thead>
<tr>
<th>JU</th>
<th>SS</th>
<th>DS</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigs in the North Sea, by April 1974</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number and percentages</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Table 1</td>
<td>11 (27%)</td>
<td>41 (61%)</td>
<td>5 (12%)</td>
</tr>
<tr>
<td>Rigs under construction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number and percentages</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Table 3 1975-1976</td>
<td>2 (12%)</td>
<td>12 (76%)</td>
<td>2 (12%)</td>
</tr>
</tbody>
</table>

Table 4 reveals that semi-submersibles are taking the lead compared to Jackups while drillships are keeping their position in the North Sea. The increase in semi-submersible rigs reflects the increased development of the fields in the NW very exposed part of the North Sea between Norway and Scotland and the Islands north of Scotland (Fig. 2).

Table 5 gives an impression of the latest development.

TABLE 5 RIGS UNDER CONSTRUCTION DUE TO WORK IN THE NORTH SEA AFTER DELIVERY (Northern Offshore, No. 12, 1975).

<table>
<thead>
<tr>
<th>Delivery</th>
<th>Owner</th>
<th>Name of Rig</th>
<th>Type</th>
<th>Operator</th>
<th>Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan. ’76</td>
<td>Keydrill Co.</td>
<td>Key Gibraltar</td>
<td>JU 3-leg</td>
<td>BP</td>
<td></td>
</tr>
<tr>
<td>Mar. ’76</td>
<td>Penrod Drilling</td>
<td>Penrod 67</td>
<td>JU 3-leg</td>
<td>Placid</td>
<td></td>
</tr>
<tr>
<td>Jun. ’76</td>
<td>Offshore Beige</td>
<td>Le Petrel</td>
<td>DS Pelican</td>
<td>Elf</td>
<td>5 yearsX</td>
</tr>
<tr>
<td>Mar. ’77</td>
<td>Sedco</td>
<td>Sedco 471 DFDS</td>
<td>Sedco 470 type</td>
<td>BP</td>
<td>2 years</td>
</tr>
</tbody>
</table>

X=including other offshore areas.
The following gives brief description of typical designs.

Fig. 3 shows the very popular new design the H-3, (modification is called H-4). This platform has been designed with a system of fairly simple and slender transverse tubular trusses. Considerable effort has been put into the design of these members and joints. Alternative designs have been evaluated, including the introduction of additional truss members to obtain shorter truss spans. However, the indicated simple truss system has been maintained on the basis of favourable results from a series of quite extensive structural calculations. This truss arrangement also fits well into the design objective of avoiding undesirable structural complexity.

The main deck structure includes two longitudinal trusses which support the drill floor and the main deck below. Deck houses between the main and upper deck are designed as self-supporting elements using external and internal bulkheads and adjacent deck plating as principal structural members. Some versions of H-3 (H-4 etc.) exist.

Fig. 4 shows the "Ocean Voyager" (Northern Offshore No. 5, 1973) by ODECO, New Orleans. This platform of semi-submersible type has a drilling capacity of about 20,000 feet, and can operate in up to 600 ft depths. The hull of the rig consists of four horizontal pontoons of 28 ft diameter, 12 vertical pillars and large horizontal tubular constructions that bind the structure together. The tops of the pillars are held together by large horizontal caissons which also carry the main deck of 3,000 m². The top of the main deck lies 128 ft over the base line. The horizontal pontoons and parts of the pillars are used as ballast tanks, fresh water and fuel tanks. The two middle pontoons are 320 ft long and terminate aft in a conical portion with propellers and rudders for self-propulsion.

Propulsion is by two D.C. engines, each of about 3000 h.p. The engines, with steering engine, ballast pumps etc., are placed in a separate engine-room in the two after conical compartments. The caissons under the main deck also used as water and ballast tanks, and as cable runways.

Fig. 5 is the "West Venture" (Northern Offshore No. 5, 1973 which is a Norwegian design developed after the so-called NORRIG 5 dutch design for Dutch-Norwegian interests). A main feature of the NORRIG 5 design is the five columns arranged in a circle, giving the platform excellent dynamic behaviour under severe weather conditions. "West Venture" has been designed for operation in the North Sea and in Arctic areas up to the iceline. It is the biggest platform of its type in the world, with the following main dimensions: length 102.22 meters, breadth 106.50 meters, height from the base of columns to main deck 46.60 m, weight of steel 12,000 tons.

Fig. 6 shows the Norwegian drillship "Havdrill". It is a twin-screw vessel fitted with c.p. propellers, five c.p. transverse thrust units and a Honeywell ASK Dynamic Positioning System, incorporating a dual RS 5 acoustic position measurement system and two H-316 real time control computers. The vessel is equipped to work in conditions varying from arctic to tropical. The hull is strengthened for navigation and operation in ice and the vessel has the following principal characteristics:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length o.a.</td>
<td>149.00 m</td>
</tr>
<tr>
<td>Length b.p.</td>
<td>137.00 m</td>
</tr>
<tr>
<td>Breadth mld.</td>
<td>21.35 m</td>
</tr>
<tr>
<td>Depth mld.</td>
<td>12.50 m</td>
</tr>
<tr>
<td>Design draught</td>
<td>7.32 m</td>
</tr>
<tr>
<td>Displacement</td>
<td>15,500 t</td>
</tr>
<tr>
<td>Maximum speed</td>
<td>14 kn</td>
</tr>
</tbody>
</table>

The drilling installations include an IHC heave compensator and an automa-
Figure 3. The AKER-H3.

Figure 4. The Ocean Voyager.
Figure 5. The West Venture.

Figure 6. The Havdrill.
tic pipe racking system.

"Havdrill"'s design is based on the following criteria: high mobility and ability to operate in all climatic conditions ranging from arctic to tropical. Her hull is reinforced to withstand the pressure of ice, self-efficiency, operational flexibility by reducing the physical connections between the ship and the sea bed, complete mechanization of the handling operations on board.

The ship has a loading capacity of approximately 7,500 tons enabling the vessel to stay at sea without supplies for a period of about 100 days.

PLATFORMS FOR EXPLOITATION

The fixed pile type in typical designs of large pile-groups is found everywhere. The question is whether this type in the version designed for the ARK or FORTIES fields (Fig. 7) has reached its upper limit. In the FORTIES field each platform shall support 10,000 tons of equipment in about 400 feet of water. The structure shall withstand 94 feet waves with a simultaneous wind velocity of 130 miles per hour.

The height of each platform from the sea-bed to the top deck is about 550 feet with the top of the drilling derrick about 140 feet above this. 49,000 tons of steel will be used in each structure excluding equipment.

The platform is floated out as shown in Fig. 8 and ballasted to an up-right position. Its 16' legs are secured to the sea-bed by means of 48 (12 to each main leg) 54 inch diameter steel pipes driven into the sea-bed from above sea surface to a depth of more than 200 ft. The top part of the sea-bed section is soft, but a firm bottoming is to be found before 150 ft penetration. The platform "pins" are cemented to guides welded to the 72/92 ft platform legs and cut off about 20 ft above the sea-bed.

PLATFORMS FOR COMBINED EXPLORATION AND EXPLOITATION

Many North Sea operators, however, have during recent years turned to concrete gravity type structures like the three types shown in Fig. 9 the British "Sea Tank", Fig. 10, the Norwegian "Condeep" and Fig. 11, the Dutch "Andoc". Reasons for the increased application of concrete drilling and production platform include a number of factors.

Costs are reportedly lower for very large concrete structure systems compared to steel structures designed for similar exposed conditions. The need for oil storage in remote fields also makes concrete attractive - compared to combination steel and separate tank systems.

In the US Gulf of Mexico, however, and offshore California, where environmental conditions are less severe and where pipe lines are easily extended to new developments, steel structures continue to be favored. Shell and Exxon, for two examples, are actively pursuing plans for use of 800 to 1,000-foot steel towers in their deep water holdings. The Dutch "Heerema Steel structure" is claimed to be suitable for 1,000 ft depth for all North Sea locations. It consists of a jacket made up of four steel towers which fit into a steel base frame incorporating pile guides.

The construction of concrete gravity platforms for production and drilling started in both the UK, Holland and Norway following several years of proposals, investigations and comparisons with all steel, pile driven designs.
Figure 7. The Auk-Forties fields platform.

Figure 8. Placement of the AUK-Forties fields platform.
Figure 9. The Sea Tank.

Figure 10 a. The Condeep Tank.
Figure 10 b. The bottom section of the Condeep Tank.

Figure 10 c. Towing of a Condeep tank to the Beryl field, 1975.
and even hybrid, combination steel-concrete systems.

However, completion of the Phillips Group EKOFISK storage tank (Fig. 12) and recent studies of ocean floor stability, plus continued oil and gas development in deeper waters - where cost of steel structures skyrocket with depth - finally initiated construction in three countries.

Twelve concrete production platforms have been announced for North Sea waters as listed in Table 6 (1975).

<table>
<thead>
<tr>
<th>Field</th>
<th>Company</th>
<th>Builder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brent</td>
<td>Shell/Esso</td>
<td>McAlpine-Sea Tank</td>
</tr>
<tr>
<td>Brent</td>
<td>Shell/Esso</td>
<td>Condeep</td>
</tr>
<tr>
<td>Brent</td>
<td>Shell/Esso</td>
<td>Condeep</td>
</tr>
<tr>
<td>Statfiord</td>
<td>Mobil</td>
<td>Condeep</td>
</tr>
<tr>
<td>Comorant</td>
<td>Shell/Esso</td>
<td>McAlpine-Sea Tank</td>
</tr>
<tr>
<td>Dunlin</td>
<td>Shell Group</td>
<td>Andoc</td>
</tr>
<tr>
<td>Beryl</td>
<td>Mobil</td>
<td>Condeep</td>
</tr>
<tr>
<td>Frigg</td>
<td>Mobil</td>
<td>Condeep</td>
</tr>
<tr>
<td>Frigg</td>
<td>Elf-Aquitaine</td>
<td>McAlpine-Sea Tank</td>
</tr>
<tr>
<td>Frigg</td>
<td>Total Oil</td>
<td>F. Selmer (Doris)</td>
</tr>
<tr>
<td>Frigg</td>
<td>Total</td>
<td>Skaanska-Doris</td>
</tr>
<tr>
<td>Ninian</td>
<td>Burmah</td>
<td>John Howard (Doris)</td>
</tr>
</tbody>
</table>

The largest offshore platform yet to be ordered will be constructed by the Scottish-based combine of Sir Robert McAlpine & Sons Ltd. and the French firm, Sea Tank Co. at a cost of nearly $90 million (Fig. 9).

Tentatively scheduled for the Shell/Esso CORMORANT field, the SEA TANK structure will be 777 feet high including the 584-foot platform, drilling deck and derrick. It will weigh over 360,000 tons including 15,000 tons of reinforcing steel. A second $60 million SEA TANK platform for the BRENT field, 112 miles northeast of the Shetlands, is already under construction for installation in 465-foot water in May 1976. One million barrels of oil storage will be incorporated in the platform base. Both platforms will be constructed at the McAlpine site at Ardyne Point on the Firth of Clyde, on Scotland's West Coast.

Britain's first concrete gravity structure for the North Sea was due to be delivered by McALPINE-SEA TANK to the FRIGG gas field in early 1975, but it has been delayed. The $33 million treating platform designated the FRIGG field will have a 236-foot square by 115-foot-high base with twin towers rising 341 feet to support a 40,000 square-foot steel deck. The top module will stand 500 feet above the seafloor in 342-foot water.

The Norwegian firms of A/S HØYER-Ellefse, one of Norway's largest general contractors, Ingeniør F. Selmer A/S and Ingeniør Thor Furuholmen, joined to form the Norwegian Contractors. This consortium, combined again with the AKER GROUP, shipbuilding firm, has proceeded rapidly with first phase construction of two CONDEEP type platforms at a site near Stavanger (Fig. 10a). Three more will be delivered during 1976.

Construction of the lower part of the base or raft takes place in drydock. The upper part of the base and columns then are slip formed after the raft has been towed out and anchored in a deep water location. Steel decks and equipment are installed in sheltered waters.
Figure 11. The Andoc Tank

Figure 12a. The EKOFISK tank.

Figure 12 b. The EKOFISK tank in the North Sea.
Designed for 100-foot waves, the Mobil and Shell drilling and production platforms for the BERYL and BRENT fields are 600, 660 and 738 feet high, respectively, including deck and derrick. Base diameter is about 330 feet. One BRENT platform will contain 65,000 cubic meters of concrete. Oil storage capacities of the three platforms are 900,000, 1 million and 1 million barrels, respectively. The accompanying photo, Fig. 10 b, of one 20 m CONDEEP base cell under construction illustrated the tremendous size of these concrete structures. Fig. 10 c shows a 676 ft high 350,000 tons CONDEEP being towed (July 1975) from Stavanger to the BERYL field in the British sector of the North Sea. It was a journey of almost 200 miles assisted by eight tugs.

Another concrete structure called ANDOC (Fig. 11) to be constructed by the Anglo Dutch Offshore Concrete group is scheduled to be completed for installation in the Shell Expro-operated, unitized Dunlin field in 1976. Construction will be initiated in Rotterdam. The structure then will be towed to deeper Scottish water for completion.

The ANDOC structure base measures 338 feet square x 105 feet in height. Four concrete columns rise 365 feet and are extended by 107-foot steel columns. Total height with deck and derrick will be over 777 feet. Water depth at DUNLIN field is 500 feet. The base can store one million barrels of oil. Expected capacity is 100,000 bpd.

Another concrete platform designs proposed by the Norwegian firm Ingeniør F. Selmer A/S consists of three bottleshaped vertical towers joined at the bottom by large box girders made of concrete.

The Selmer TRIPOD can be designed for the deepest water presently under development or consideration, that means up to 300 m (Fig. 13). But efficient use of concrete also makes the design more competitive for water depths less than 300 feet. It has very satisfactory oscillation characteristics outside the range of occurring wave periods. Its structural configuration provides the rigidity required to prevent oscillations becoming a problem for stability.

Selmer is presently constructing a production and booster platform for Total Oil Marine Ltd.'s FRIGG gas field. The 330-foot diameter x 400-foot-high platform is built like the EKOFISK oil storage tank (Figs. 12 a and b), where the perforated outer walls take the brunt of breaking waves by eliminating shock pressures decreasing reflection, upwash and oversplash. A similar platform will be built by Scottish contractors on the NINIAN field for the Burmah Oil.

A Swedish-French company, Skaanska-Doris is building a Doris combined booster and production platform for the FRIGG field at 300' depth. Diameter is 375'. The advantages of the concrete type of gravity platform compared to the pile-type is their tremendous bottom weight making then very stable provided soil conditions are satisfactory to carry the heavy loads. This is usually the case in the North Sea with its fine sand and moraine bottom soils. They are relatively easy to place as no piles have to be driven in the bottom and they can carry relatively heavier loads on their deck than the conventional pile-group platforms. Their storage ability is a definite advantage for oil production. Scour does not seen to be any severe problem for the depths in question (70 m to 150 m). Another advantage is that they may be refloated and moved to another place. The newest developments propose a combination of concrete and steel. They include e.g. the Subtank and Sub Sea tank. They have a concrete bottom and steel towers. The Chicago Bridge and Iron proposed platform has in its first issue a triangular 550 ft sideline, 20 ft high concrete tank foundation and an approximately 735
Figure 13. The Selmer TRIPOD.
600 ft high steel structure with towers decreasing in diameter upward carrying the steel deck. It is proposed to be built in a dry dock and towed to the site floating on the more than 100,000 tdw. bottom tank section. A special construction harbor is under discussion for the NW coast of Ireland at Killala Bay. At present plans have been deferred, however, due to the general slow-down of activities in platform construction for the North Sea.

The three designs mentioned above are the first attempts to compromise between the slender and flexible steel and the massive and inflexible concrete utilizing either one based on its own merits. Concrete is mainly a compression material, therefore useful for bottom structures, steel is a tension material therefore useful where high tension forces occur. But steel is also useful as a bottom structure (the Dubai tank) and there is a possibility that one may see concrete semisubmersibles like the CONDRILL proposed by Norwegian contractors in the future.

A new light and flexible concrete platform called the "elementpillar system" has been proposed by a Norwegian group. The construction consists of three types of elements. The dimensions of the element types can be varied according to application. The overall material consumption of the construction is so planned that is should be economically favourable to standardize the production as far as possible and only make use of very few types, even if this should lead to an overdimensioning in some cases.

The design of the structure permits different levels of the three foundations. The sea bottom under the foundations need not be horizontal. Inclinations up to 50° can easily be overcome, and even cases of larger inclination can be mastered.

Concrete and steel both are exposed to destructive chemical attacks. They are, however, not supposed to last until eternity and concrete technology as well as steel welding techniques for marine structures have improved greatly. Both require good foundation conditions but while steel has to be pinned down, concrete does not need that kind of stabilization. Combinations of the two therefore seem to be the logical answer. The deeper the more steel in piles or towers. The bottom tank may still be concrete. Full concrete structures still seen to be competitive in the 150-250 m depth range, perhaps deeper. The next depth range may be occupied by articulated designs like the hinged French ELF proposed as flare platforms in the North Sea and various kinds of tension leg platforms like the TRITON, DOT (Fig. 14), and tripods with articulated joints like the HENDERSON TRIPOD (5). The AKER tethered production platform is economically competitive for depths > 500'. Various offshore loading systems have been proposed. One is the NORBUOY "Bottle" by a Norwegian group (Furuholmen et al.). The NOB F 120 consists mainly of a floating concrete body which carries a rotating steel deck for operation activities.

With respect to structures for deep water designers must be aware that deep-water structures are dominated by factors unimportant in shallow water. Dynamic response of resonance character for instance, becomes a significant factor beyond the 500-foot to 700-foot depths and very critical evaluation of future deck loads is necessary. The above mentioned Selmer TRIPOD has favorable oscillation characteristics.

It is claimed that conventional Gulf Coast fabrication yards have capability to construct steel platforms for water depths up to 900 feet, with moderate deck loads. If base widths of 350 feet could be handled, construction could be expanded to 1,200-foot depths. These dimensions may be scaled up.
for conditions in the North Sea e.g. involving 600 foot bases and 1,400-foot depths.

Several tower platform, e.g. the ARK have been proposed but oil companies seem to be somewhat reluctant with respect to the use of just one leg. The hinged ELF type may have a better chance as the dynamic response problem is much less severe.

A large spar-type platform of DRAVO (SPAR) type for combined storage and loading, 460' high, 100' diameter, 300,000 barrels capacity has been built and will be moored (1976) on the British BRENT field. The Norwegian TROSVIK Group has developed the deep water BIG BUOY for 500 m (1700') operation in "difficult climatic conditions". The rig has a storage capacity of 300,000 barrels. It is cylindrical in shape and will be built with its lower hull in concrete and upper part in steel.

The now almost classical DORIS in the EKOFISK field (Fig. 12) was designed as a combined storage and production facilities platform. It is mainly going to function for the purpose of the latter as a 34 inch pipeline to England will take over transport of oil. The DORIS tank is placed in 70 m depth and is able to hold one million barrels. It projects 20 meters above Sea Level. In plan, its shape is roughly circular inscribed within a circle 95 m in diameter. The structure includes an outer perforated protection wall, basically intended to withstand wave shocks and to reduce swell effects (according to the Jarlan patent) and an internal wall, forming the reservoir itself, the whole being fixed in a foundation raft.

The foundation raft is a cellular structure, in prestressed concrete, with a depth of 6 m, the bottom slab being 0.60 m thick and the upper slab 0.20 m, the cells being filled with sand or mass concrete.

The outer protection wall has a thickness of 1.35 m over the lower part and 1.83 m over the top 32 m. The perforations, intended to reduce shocks from breaking waves. The total force on the outer perforated wall is higher than the force on the inner solid wall.

Realizing the number of problems associated with these huge new concrete designs, comprehensive instrumentation program has been initiated to record all pertinent data needed for detailed evaluation on the integrity of the structure.

The very severe wave conditions and large depths in the NW part of the North Sea, however, may call for the development of bottom structures which are connected to the surface by articulated risers, towers or cable ways. This will probably be true when depths exceed 400 meters. Steel as well as concrete bottom structures have been proposed based on advanced prestressed concrete technology. Transportation to shore may be by pipelines or by tankers moored to or combined with a buoy, platform or any kind of hinged structure. The increase in prices for crude oil has also increased the possibilities for profitable recoveries even under the very hostile conditions of the North Sea and its surrounding waters towards the North and West.

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Table 4 Rigs under construction due to work in the North Sea after delivery (Plateau), December 1975.
Table 5 Comparison between Table 3 (75/76) and 1 (74).
Table 6 Concrete platform for the North Sea (5).
DETERMINATION OF ICE FORCES ON A CONICAL OFFSHORE STRUCTURE

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and

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ABSTRACT

Conical shapes have been used for a long time in building Canadian offshore lighthouses, but the actual mode of destruction of an ice sheet colliding with a conical structure is still not fully understood. The measured and theoretical values of the horizontal force calculated by the different methods have been compared. Calculations and field observations suggest that a failure mode transitional between that of crushing and flexure occurs for the steeper cone angles.
INTRODUCTION

Increased demand for inland winter navigation and the need to exploit offshore resources in the Arctic ocean has focused the attention of engineers on the problem of the determination of ice forces on offshore bottom-founded structures (Bercha 1975, 1974).

Approximately 20 years ago, the Marine Aids Division of the Canadian Ministry of Transport started a major program of building new lighthouses or reconstructing existing ones. Initially, a design value of 400 p.s.i. (28.1 kg/cm²) for the crushing strength of ice was used for the major lighthouses. It appeared that the design ice forces were very conservative and it was felt that measurement of the ice forces acting against the lightpiers were required for establishing the proper design standards. The desirability of more economical construction made an in-situ load measuring installation necessary. In 1972, a fully automated and remotely-controlled system to measure ice impact forces was installed in a conical lightpier of the Western Light Range in Lac St. Pierre about 65 miles (105 km) downstream from Montreal. The first force readings were obtained in the winter of 1973-74 (Danys 1975).

In 1971 a study of the actual ice forces which had caused some structural deformations of the lightpiers was started (Danys 1972) in order to compare the obtained values with the contemporary design values.

Although cylindrical and conical shapes have been used for a long time as the ice-load bearing component of offshore lightpiers, the precise nature of the ice-structure interaction does not yet appear to be fully understood (Danys 1971). Mathematical studies to improve the existent methods of calculations of the ice forces and to complement the in-situ measurement program were commissioned by the Ministry of Transport in 1973, and have been done by Acres Consulting Services Limited, Calgary, Alberta. These studies utilized modifications of some of the analytical approaches previously developed for the determination of bearing capacities of floating ice sheets (Nevel 1972), in conjunction with aspects of the conventional crushing theory approaches (Danys 1971) to develop mathematical simulators for ice sheets impinging on conical structures.

The laboratory model testing had been considered but not carried out. It seems that at present the artificial ice does not simulate adequately enough all physical properties of a natural floe.

ICE FORCE MEASURING INSTALLATION

The installation (Danys 1975) shown in Figures 1 and 2, consists essentially of four main components: (a) the ice force sensing panels, (b) the electrical integrating circuit, (c) the recorders, and (d) the remote control. The five load sensing panels were placed on the upstream side of the lightpier, exposed to the moving ice. Each panel was supported by four load cells. Continuous recording of data from each load cell was ruled out by the large volume of data which would have resulted; instead, an integrating circuit was used to combine the separate load cell outputs and to yield a total horizontal load.
Figure 1. Arrangement of Ice Force Measuring Installation at Yamachiche Lightpier

Figure 2. Instrumentation Diagram of Ice Force Measuring Installation
There are two recording systems. The time-accumulating registers record signal from the integration equipment of the net sum of the bending moments of the five ice force sensing panels caused by the ice thrust. Five time-accumulating clocks are adjustable and initially are set for the following levels - 20/40, 40/60, 60/80, 80/100 and 100/120 per cent of the calibrated load. Digital clocks register the moment of the ice thrust on the structure whenever it is within the preset load ranges. The cumulative duration of time in minutes of the ice thrust acting within the preset load range is recorded. The multichannel oscillograph recorder continuously registers the output and fluctuation of each pair of load cells and the resultant moment for a preset time interval.

The remote operation and sensing system consists of three main parts (Fig. 2): (a) a remote control station in Montreal, (b) a land radio station of Louiseville on the shore of the lake, (c) a radio station on the lightpier in the lake. A multichannel oscillograph can be turned on from the remote control station in Montreal. A telex connection between the remote control station in Montreal and the land and lightpier radio installations sends binary-coded data from the lightpier to a display box at the remote control station in Montreal. The remote control allows the oscillograph to be turned on at any time an ice movement takes place, or is anticipated for continuous recording.

FIELD PROGRAM

To date, due to operating difficulties, only a fraction of the ice force measuring capability of the installation has been realized. Operating problems have included damages to the submarine cable by ship anchors and breakdown of the diesel generator, resulting in loss of power. Further, because of jamming of the oscillator recorder and failure of the telex line, the actual time of the acting forces was not recorded; this made difficult to correlate the measured forces with ice thicknesses and water levels. Consequently, only the results measured by the time-accumulating clocks during part of the 1973-74 winter can be presented at this time.

Table 1 shows the five preset load ranges, calibrated values and the actually registered time for each preset load cumulative range. The range of the calibrated force per linear foot of the pier diameter is given for the average water level.
<table>
<thead>
<tr>
<th>Clock</th>
<th>Range in Percent</th>
<th>Calibrated Load Range</th>
<th>Duration of Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Total Load in 1000</td>
<td>Unit Load in 1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>lb</td>
<td>lb/lin.ft.</td>
</tr>
<tr>
<td>1</td>
<td>20 - 40</td>
<td>96-193</td>
<td>8.2-16.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(44-88)</td>
<td>(12.2-24.7)</td>
</tr>
<tr>
<td>2</td>
<td>40 - 60</td>
<td>193-289</td>
<td>16.6-24.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(88-131)</td>
<td>(24.7-36.9)</td>
</tr>
<tr>
<td>3</td>
<td>60 - 80</td>
<td>289-386</td>
<td>24.8-33.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(131-175)</td>
<td>(36.9-49.3)</td>
</tr>
<tr>
<td>4</td>
<td>80 - 100</td>
<td>386-482</td>
<td>33.1-41.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(175-219)</td>
<td>(49.3-61.5)</td>
</tr>
<tr>
<td>5</td>
<td>&gt; 100</td>
<td>&gt; 482</td>
<td>(b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(&gt; 219)</td>
<td></td>
</tr>
</tbody>
</table>

Notes: (a) and (b) - defective clocks

For design, the maximum ice force is important. It is estimated that the registered maximum total horizontal force on the lightpier was 434,000 lbs. (197,000 kg) for an ice thickness of 24 inches (61 cm), and that it acted at the water level approx 5 ft. (1.52 m) above the low water. Assuming the average coefficient for the pier shape and contact as 2/3, the actual crushing strength of ice was calculated to be 193 p.s.i. (13.6 kg/cm²). For design, a crushing strength of 250 p.s.i. (17.6 kg/cm²) was assumed.

Twenty mile long, Lac St. Pierre is a critical section of the St. Lawrence River to maintain winter season navigation as well as to protect Montreal against winter floods. Therefore, water levels, ice conditions and various climatological factors in Lac St. Pierre have been observed for many years. Measurements of ice thickness, ice sampling, laboratory testing and observations of ice movement have been carried out for several years. In 1971 an automatic water level gauge station was installed in another lightpier in the lake. All these observations allow a better and more complete evaluation of the measured forces. Lac St. Pierre is a very good place to compare the
calculated and observed or measured ice forces on the offshore structures because of the available data and frequent movements of ice.

**THEORY**

A vertical force, $F_z$, impinging on a plane inclined at an angle from the horizontal can be shown to generate a horizontal force component, $F_x$, given by

$$F_x = \xi F_z$$

where

$$\xi = \frac{\mu \cos \alpha + \sin \alpha}{\cos \alpha - \mu \sin \alpha}$$

is called the "resolution factor", and $\mu$ is the coefficient of friction at the interface of the inclined plane and the object through which the forces are applied. An average value of $\xi$ can be obtained by integration for the case of the vertical force distributed uniformly over half of a circular cone at the level of diameter $b$, and having a cone angle $\alpha$.

Using Nevel's (1972) theory, the total force, $F_z$, distributed in a specified manner, necessary to fail a floating ice sheet can be expressed in the form

$$F_z = F_z(a, \sigma_f, h, E, \rho)$$

where $a$ is the radius of the semi-circular line load distribution; $\sigma_f$, the flexural strength of ice; $h$, ice thickness; $E$, Young's modulus of ice; and $\rho$, the density of water. The explicit form of Eq. (3) is dominantly quadratic in $h$.

Now if we identify the vertical force, $F_z$, as the appropriate bearing capacity of a floating ice sheet given by Eq. (3), then the force obtained through Eq. (1) is identifiable as the approximate horizontal thrust which a conical structure must withstand in order to fail an impinging ice sheet in flexure.

In fact, a more exact value of this horizontal thrust can be obtained by compensating for the effects of the in-plane stresses and end-moments generated in the interaction. Say, the corrected force, $F_x^1$, is given as

$$F_x^1 = C_s & C_m F_x$$

then it can be shown that a first approximation for the two correction factors can be expressed as the two non-linear functions

$$C_s = C_s(\xi, h, a, E, \rho, \sigma_f)$$

$$C_m = C_m(\xi, h, a, E, \rho, \sigma_f)$$

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The above analytical expressions (Equations (1) to (4b)) were corroborated and found to agree very closely with the results of a linear finite element plate program which included the effects of cracking, buoyancy, and in-plane stresses and end moments. In what follows, calculations based on the above theory will be referred to as "flexural calculations".

On the other hand, calculations of the total force involving only the assumptions of a crushing type failure, and referred to as "crushing calculations", will be based on the crushing formula (Danys 1971), used for design of the Canadian lightpiers.

\[
\begin{align*}
R &= mn b h q_c \\
H &= m n_1 b h q_c \\
V &= m n_2 b h q_c
\end{align*}
\]

where

- \( R \) = resultant force on structure perpendicular to the surface,
- \( m \) = shape and contact coefficient,
- \( n \) = slope coefficient taken as \( \cos A \) for \( R \); \( \cos^2 A \) for \( H \); \( \cos A \sin A \) for \( V \); \( A \) is a slope angle with the vertical,
- \( H \) and \( V \) = horizontal and vertical components of \( R \),
- \( b \) = projected width of the structure, equivalent to the diameter for the circular shape,
- \( h \) = effective thickness of ice sheet,
- \( q_c \) = effective compressive strength of ice.

In Figure 3, coefficients \( k_1 = mn_1 \) and \( k_2 = mn_2 \) for the horizontal and vertical components of the ice impact forces are shown when the ice failure mode is "crushing" and there is no friction between the ice and the pier. For the design of Canadian lightpiers, the curves A and B have been used since 1959 (Danys 1971); the curves C and D were proposed by Carter in 1973 ("Action de la glace sur les ouvrages maritimes", Internal report to the Canadian Dept. of Public Works, 1973, unpublished) on the basis of the theoretical calculations. It is felt that all these curves give too conservative design forces for the large angles of the cone, say 30°-45°, because for such angles the failure mode is "flexure". But field observations have indicated that the ice failure mode against the conical structures with a slope angle up to 15° is basically a failure by crushing.

**COMPARISON OF MEASURED AND CALCULATED VALUES**

Figure 4 graphically compares the results of the calculations and the measured values of the total horizontal force at Yamachiche for various ice thicknesses.

Due to the uncertainty of the input parameters used, such as friction coefficient, flexural strength, crushing strength, and so on, attention should be restricted to the qualitative aspects of the comparison, rather than on the quantitative ones.
Figure 3. Coefficients for Failure Mode by Crushing

Figure 4. Comparison of Calculated and Measured Values at Yamachiche
The comparative calculations were made for the Yamachiche lightpier shown in Figure 1 at water level 5 ft (1.52 m) above the low water level. The crushing strength was assumed to be 250 p.s.i. (17.6 kg/cm²), and the flexural strength 100 p.s.i. (7.0 kg/cm²). Further assumptions were: \( m = 0.67, \, n = 0.94 \) for the "crushing" calculations in the formula (5a), and the maximum value of the "resolution factor". For a cylindrical shape, the effective value of the resolution factor would be lower; the minimum value being 0.64 of the maximum.

Only the maximum measured force of 434 kips (197 t) was plotted because other measured forces - 150 and 241 kips (68 and 109 t) could not be reliably related to the actual ice thickness and water levels.

CONCLUSIONS

The restriction of the amount of field measurements emphasized the operating difficulties associated with most cold weather operations, remote or manned. In this instance, several of the remote-control functions failed unexpectedly, necessitating previously unplanned and sometimes impossible maintenance trips to the installation.

In the mathematical work, a means to relate transverse flexural failure forces and horizontal in-plane sheet forces, with a first-order correction of the failure forces to account for in-plane and end moment effects was developed and utilized. Both the resolution factor which relates in-plane and transverse forces, and the force and moment corrections have not appeared previously.

The observed failure mode at the Yamachiche lightpier, and others of the same type, with a slope angle of approximately 15°, was predominantly crushing (Figures 5 and 6).

Although the numerical agreement should be regarded with some suspicion due to the low confidence in the input parameters, the qualitative corroboration and field observations suggest that a failure mode transitional between that of crushing and flexure occurs. Further work is required for the identification and analytical tractions of this transitional mode. In addition, the importance of the end moment and in-plane force effects, as suggested by the high values of \( C_s \) and \( C_m \), justifies further studies of those effects.

ACKNOWLEDGEMENT

The permission granted by Mr. G.L. Smith, Chief of the Marine Aids Division, to present this paper is gratefully acknowledged.
Figure 5. Failure of an Ice Floe on a Conical Substructure with a $15^\circ$ slope angle (Lightpier at Curve No. 2, Lac St. Pierre, 1971).

Figure 6. Failure of an Ice Floe on a Conical Substructure with a $45^\circ$ slope angle (Lightpier at Yamachiche Curve, Lac St. Pierre, 1971).
REFERENCES


DESIGN AND BUILDING OF TEMPORARY ARTIFICIAL ISLANDS IN THE BEAUFORT SEA

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Hydronamic B.V.
Port and Waterway Engineers
Sliedrecht, The Netherlands

ABSTRACT

During the past four years, Imperial Oil Limited has constructed six artificial islands in the Beaufort Sea to drill exploratory wells. Artificial islands were chosen over other drilling platforms because:

(1) The exploratory program is in shallow water (0-60 feet)
(2) No new technology was required
(3) Lower costs
(4) Minimum risk to the environment

Design of the islands has considered the following criteria: hydrography, tides and currents, winds and waves, strength, thickness and movement of ice, soil mechanical properties of fill and foundation, environmental restraints and the availability of construction equipment.

Calculation of particular concerns included:

(a) The limits of angle and elevation with respect to failures in the fill and/or slide failures in the foundation
(b) Foundation settlement
(c) Shear resistance of fill and foundation under ice loading
(d) Stability of slope protection under wave attack

Since the islands only need to last long enough to allow drilling of an exploratory well, slope protection need only be good enough to get through the construction season.

Two- and three-dimensional slope protection models were tested in the Delft Hydraulic Laboratory to determine what damage could be expected under various conditions of waves and wind.

All islands built so far have proven to be very resistant to wave and ice attack despite the fact that they were designed as temporary islands.
MAN-MADE ISLANDS USED AS DRILLING PLATFORMS

Since 1972, eight artificial islands have been built in the Mackenzie Bay area of the Beaufort Sea (Fig. 1). Two of these islands were built by Sun Oil Company and the other six by Imperial Oil Limited. The locations were up to 15 miles from the mainland and in water depths of up to 15 feet (Fig. 2). Three more islands are now under construction in 5, 23 and 40 feet of water, respectively, and later on this season, the building of a fourth island will begin in 22 feet of water.

A number of concepts for exploratory drilling platforms in the Beaufort Sea were considered, including submersible gravity structures that could be used throughout the year, such as the "Monopod" and the "Cone" (Ref. 5). Ice platforms were also investigated as were several types of floating rigs for seasonal operations.

Several of these concepts appeared to be feasible, but there were a number of reasons to select man-made islands instead of gravity structures or conventional floating rigs for Imperial's offshore exploration permit acreage in the Beaufort Sea. These include:

a. The permit acreage is in shallow water, extending from the shoreline to about the 60-foot contour.

b. The short working season (2½ - 3 months) results in very high standby costs for floating rigs during the winter.

c. Islands are considered to be the safest solution with regards to environmental risks and their resistance to ice forces.

d. The initial capital investment for structures is very high when compared to man-made islands. This is even more important when the number of prospective locations is small and very much dependent on the ratio of success (Fig. 3).

e. The building of artificial islands is a well-known technology and construction work requires only standard construction equipment.

Designing and building of islands in the Beaufort Sea and in the North Sea are, on the whole, alike, but in a number of aspects there are real differences, such as:

- the ice forces acting on the islands;

- the short working season;

- the absence of roads, railroads and ice-free deep waterways essential for the transportation of certain building materials; as a result, the islands in the Beaufort Sea have to be built mainly from silt, sand or gravel;

- the temporary character of the exploration islands.

Having been involved before in the design of artificial islands in the North Sea area, Hydromonic B.V. was asked to help design Imperial's exploration islands in the Beaufort Sea.

The present paper deals with the design criteria for islands in the Beaufort Sea, the design of these islands, the soil, mechanical and hydraulic model investigations and the actual building that has been carried out so far.
Figure 2

MAIN OUTLINE OF IMPERIAL ACREAGE

<table>
<thead>
<tr>
<th>NAME</th>
<th>DEPTH</th>
<th>YEAR OF CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 PULLEN</td>
<td>8 FT.</td>
<td>1973/74</td>
</tr>
<tr>
<td>2 SUNOCO IS.</td>
<td>7 FT.</td>
<td>1973</td>
</tr>
<tr>
<td>3 IMMERK</td>
<td>8 FT.</td>
<td>1972/73</td>
</tr>
<tr>
<td>4 SUNOCO IS.</td>
<td>7 FT.</td>
<td>1974</td>
</tr>
<tr>
<td>5 NETSERK</td>
<td>12 FT.</td>
<td>1974</td>
</tr>
<tr>
<td>6 ADGO I</td>
<td>6 FT.</td>
<td>1973</td>
</tr>
<tr>
<td>7 ADGO III</td>
<td>6 FT.</td>
<td>1974/75</td>
</tr>
<tr>
<td>8 ADGO II</td>
<td>6 FT.</td>
<td>1974</td>
</tr>
</tbody>
</table>

NICHOLSON PENINSULA

MACKENZIE BAY

TUKTOYAKTUK

SHINGLE POINT

HERSCHEL IS.
Figure 3

Principle of cost development for islands and structures

- Monopod and cone structures, drill vessels
- Deep water art. islands
- Shallow water art. islands

Total costs vs. number of wells
CONDITIONS IN THE BEAUFORT SEA

The availability of basic data on the sea-state, ice and sea bed conditions is essential in order to establish the criteria for island designs that stay within the framework of acceptable risks (safety factors, storm frequencies). Since 1969, oil companies with offshore interests in the Canadian Beaufort Sea have conducted numerous projects on environmental conditions, ice properties, etc. Many of these studies have been conducted through cooperative programs within the Arctic Petroleum Operators Association (APOA) which was established for this purpose. To date, over 90 projects have been completed at a total cost of around $6 million. The design criteria discussed in this paper are based on these studies and also other work conducted independently by Imperial Oil. From a meteorological point of view, the Beaufort Sea is a moderate area. It is the cold which gives rise to the exceptional conditions. The major area of interest for data is centered on the conditions existent in the area around the natural islands, Garry Island, Pelly Island, Hooper Island and Pullen Island.

The design conditions can be divided into three main groups:

- the sea-state conditions - water depth, currents and waves;
- the ice conditions;
- the soil conditions.

The Sea-State Conditions

These conditions are of interest for the purpose of determining the height of the island, the stability of the required sea defences, the erosion of the sea bed and the downtime during construction.

a. The design water depth is made up of the following three items:

- the water depth in relation to Chart Datum (in the area under consideration, Chart Datum is about 1.5 feet below mean sea level). Fairly recent sounding charts are available. The sea bed (Fig. 2) shows a very gentle slope in northerly direction;
- the tidal elevation above Chart Datum. The area under discussion, the tidal amplitude varies from 1½ feet at neap tide to 3 feet at spring tide;
- the wind set-up above the tidal elevations.

Little statistical information is available on storm tides. Hindcasting procedures have therefore been applied. Wind speed and direction were obtained from geostrophic weather charts and fetch from ice summary charts. The following tabulation shows the calculated storm tides for a number of Chart Datum depths. They include a 3-foot astronomical tide and 1-ft pressure effect.

<table>
<thead>
<tr>
<th>Chart Datum (ft)</th>
<th>8</th>
<th>25</th>
<th>40</th>
<th>80</th>
<th>120</th>
<th>160</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 x 50 years</td>
<td>8.5</td>
<td>7.5</td>
<td>6.5</td>
<td>5.5</td>
<td>4.5</td>
<td>3.5</td>
</tr>
<tr>
<td>1 x 100 years</td>
<td>9</td>
<td>8</td>
<td>7</td>
<td>6</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>
b. Currents are rather weak, generally less than 0.5 knots. Locally, higher velocities at contraction points and in the case of storms may occur. Figure 4 shows the pattern of the residual currents.

c. Waves - due to the fact that in the Beaufort Sea, direct wave measurements were limited and not sufficient for a statistical analysis, the wave height and wave period distribution were calculated using wind data and fetch. The hindcasting technique was checked against the limited wave measurements that were available.

Upon entering shallow water this calculated wave height $H_s$ (the average height of the highest one-third part of the wave field) changes according to the water depth ($d$) and several other factors. Where there is a gentle slope of the sea bed towards the shore the upper limit is approximately:

$$H_s = 0.40 \times d$$

On this basis, the following significant wave heights have been determined for a 50-year recurrence interval:

- $d$ (depth related to Chart Datum): 8 ft, 20 ft, 40 ft
- $H_s$ (significant wave height): 7-8 ft, 11-13 ft, 14-17 ft

The Ice Conditions

These conditions are important for the purpose of analyzing the forces of moving ice acting on the islands and for the purpose of determining the working season. Only when the ice is moving in relation to the island are ice forces being exerted. Several studies have been carried out to determine the extent of the ice movement, but the forces transmitted by this movement onto the island are not as yet fully known quantitatively (Ref. 1). Studies are now under way to determine the actual forces exerted on the islands by the ice movement. Specially designed sensors have been installed near several of the artificial islands. The recorded ice pressure readings indicate that the ice is in almost continuous cyclic movement, especially around islands in water deeper than 10 feet (outside the barrier islands). The failure mode of the ice appears to be a combination of continuous crushing, bending and shear and is very complex in nature. The actual measurements are being interpreted but, to date, it has not been possible to arrive at a statistically acceptable design load. For the time being, we are therefore using the crushing strength of ice as the ice design criterion. Factors influencing this crushing strength are:

- strain rate
- the temperature
- the crystalline orientation
- the $D/t$ value (in which $D$ is the diameter of the structure and $t$ the thickness of the ice)
- the thickness of the ice sheet
- the salinity of the water
PATTERN OF THE RESIDUAL CURRENTS

Figure 4

[Herschel Whorl, Komakuk Beach, Herschel Is., Mackenzie Bay, Shingle Point, Mackenzie Influence, Tuktoyaktuk, Nicholson Peninsula]
ICE CRUSHING STRENGTH RELATED TO D/t

Figure 5

D: DIAMETRE STRUCTURE
t: ICE THICKNESS

THEORY
Published data on the crushing strength of arctic ice indicates values in the range 500 to 1,000 psi for ice frozen to test piers, which were up to 5 feet wide. For wide structures and for ice in random contact, the average ice pressure across the structure will be less than the above values (Ref. 3).

The Soil Conditions of the Sea Bed

Information on these conditions is required to determine:

- the bearing capacity and the settlement of the island;
- the most suitable borrow area for silt or sand;
- the stability of the shore protection;
- the resistance of the island against ice forces.

Geologically, it can be said that, in general, east of the 132° W-Meridian, the sea bed consists of thick layers of medium to fine sand covered with a silty overburden of from 5 to 50 feet, decreasing in thickness in easterly and northerly directions (see Fig. 7). East of the 134° W-Meridian the silt cover is usually absent. Except for some isolated locations, no sand can be recovered economically west of 132° W-Meridian. However, this western region is of major interest, because the building of islands is concentrated in this area. The sea bed in this area consists mainly of thick layers of silt.

Geotechnically, the values of some critical parameters for this silt are as follows:

- the permeability (k); an average value of \(10^{-5}\) m/sec has been found;
- the unit weight of silt \(\gamma_s\) in situ varies from 110 pcf to 125 pcf, according to depth;
- the void ratio \(e\) generally varies from 0.8 to 1.1;
- the cohesion was found to vary from 0 to 2 psi. It has been assumed that, normally, no cohesion exists;
- the angle of internal friction \(\phi\) in weak samples varies from 28° to 32°;
- the compression index \(C_v\), a parameter governing consolidation and settlement, ranges from 3.5 to 20 ft²/day.

Figure 9 gives an envelope of the grain size distribution of the silty soils and sands.

DESIGN CONSIDERATIONS

Basically, man-made islands consist of two main parts:

a. The body of the island. It has to provide a sound base - with a minimum radius at the surface of 160 feet for the drilling operations and must, for this purpose, have adequate bearing capacity.

b. The slope protection construction. It has to protect the slopes of the island against wave forces in the summer season and should, itself, suffer minimum damage as a result of ice movement in the winter.
GENERAL GEOLOGICAL PROFILE OF THE SEA-BED IN THE MACKENZIE BAY

MACKENZIE CANYON

SILT

RICHARDS IS

SAND

TUKTOYAKTUK PENINSULA

SCALE:

WEST

EAST

133° W MER

60

FEET

6 16 MILES

Figure 7
Figure 8
ENVELOPE OF THE GRAINSIZE DISTRIBUTION FOR SAND AND SILTY SOILS

Figure 9
The design of the island body and its sea defence construction is determined by:

- The sea-state conditions (depth, water levels, currents and waves), the ice conditions and the sea bed conditions as previously discussed.
- The building materials that will be used. This is an economical decision.
- The construction spread that is available. Mobilization of new equipment takes time and must be economically justifiable.

Several islands have been designed and built. Each of these islands is different from the previous one. The variation in the designs can be explained by the following:

- the experience obtained during the building of the islands with respect to construction techniques and construction equipment;
- the boundary conditions are not the same for all locations;
- new developments in the design of slope protection.

Available Building Materials

For the island body, both silt and sand and gravel have been used. The disadvantage of silt is that its consolidation process is slow. The great attraction to use silt is that it is available in large quantities in the area of major activity (west of the 134° W-Meridian). In this area, sand and gravel is only found in limited quantities in isolated spots that are difficult to find. Sand in the area east of the 134° W-Meridian is available in large quantities, but in this case, the long haul distance constitutes a problem (Fig. 8).

Slope protection of islands provides protection mainly against wave forces, and normally consists of concrete blocks, quarry stone and bitumen mixtures (Refs. 3, 8 & 9). These materials are very expensive in the Beaufort Sea, due to long and difficult transportation to the site, and since the islands need only last long enough to drill an exploratory well the following temporary methods have been studied and used:

- filter cloth held down by wire netting;
- sand bags;
- gabions filled with sand bags;
- sand-filled plastic tubes.

The bag and filter-cloth materials were selected mainly on the basis of sand tightness, permeability to water and strength.

Construction Spread

As a result of the high costs of moving construction equipment to and from the Mackenzie Bay area and the fact that the building season lasts only 80 days, long term commitments must be made by the user. The equipment spread should, therefore, be as versatile as possible and be able to build islands in a number of different situations (Refs. 6 & 7).
Within this framework and on the basis of what was known some years ago, the first pieces of equipment were chosen. Gradually, new equipment was added as the requirements grew, and at the present time, the following equipment is under contract to Imperial Oil:

- 24-inch cutter dredge (Fig. 10).
- 34-inch stationary suction dredge (Fig. 11).
- five 2,000-cubic yard bottom-dump barges (Fig. 10).
- three 300-cubic yard bottom-dump barges
- four 1,500 h.p. tugs
- two 600 h.p. tugs
- one floating crane
- four 6-cubic yard clamshell cranes on spudded barges (Fig. 12).
- barge loading pontoon
- floating pipelines
- floating camps and repair shop
- sandbagging machines (Fig. 13).
- several other barges, launches and auxiliary equipment

Design of Slope Protection

The following designs have been considered for the slope protection of islands:

a. An unprotected artificial beach profile with gentle slopes. Calculations showed that considerable erosion would occur during heavy storms and that a large buffer zone of fill would be required above water to prevent erosion within

b. An artificial beach protected only by filter cloth covered with wire netting to keep the cloth in place (Figs. 14,15). Due to the gentle slopes of the beach, large quantities of fill are required. This type of construction demands a great deal of maintenance, especially in heavy storms. In shallow water it is a solution to be considered.

c. A defence construction consisting of small sandbags, with relatively steep slopes. This is a solution for shallow water (0 to 8 ft) only. In deeper water the weight of the bags has to be increased (Fig. 16).

d. A defence construction consisting of gabions filled with small sandbags. Gabions are wire netting cages normally filled with rock, but in this case, sandbags. During the filling and placing of the gabions, difficulties were encountered, and for this reason, it was decided to abandon this solution.
Figure 16. Placing 2 cu ft Sandbags on Slope of Island in 5 Feet of Water

Figure 17. 2 yd$^3$ Sandbags on Slope of Island in 15 Feet of Water
Figure 14. Filter Cloth, Chain Link Fencing and Torpedo Netting on Island Slope

Figure 15. Finished Slope of Island
Figure 12. 6 yd³ Clamshell Cranes on Spudded Barges

Figure 13. 2 yd³ Sandbagging Machine
Figure 10. 24" Cutter Suction Dredge, "Arctic Northern" and 2,000 yd$^3$ Dump Barges

Figure 11. 34" Stationary Suction Dredge, "Beaver Mackenzie"
e. A defence construction consisting of large sandbags of 5 to 10 tons instead of the gabions (see Fig. 17). These units can withstand significant wave heights of 10 to 13 feet, and therefore, apply to water depths of up to 25 feet.

f. A defense construction consisting of a system of sand-filled tubes. This solution is under development and will be used for islands in water deeper than 25 feet. This solution may turn out to be cheaper than large sandbags.

Figures 18a to 18f show the schematic drawings of the various solutions discussed above.

SOIL MECHANICAL INVESTIGATIONS

From a soil mechanical point of view, three main questions are of interest:

a. What should be the rate and the method of construction of the island to ensure the stability of the island and its slopes during the construction period.

b. What should be the dimensions of the island, in order to be able to withstand the ice forces.

c. What should be the extra height of the island to compensate for settlement of the sub soil and the fill.

Soil mechanical computations can answer all three questions.

Basic Assumptions

The following assumptions have been made:

- water depth in relation to Chart Datum: -7.5 ft, -13 ft and -23 ft;
- island surface elevation in relation to Chart Datum: +5 ft, +20 ft and +30 ft;
- island slope: 1 in 3 and 1 in 4;
- soil characteristics of island fill:
  \[ \gamma_f = 100 \text{ pcf} \]
  \[ \gamma_f = 122 \text{ pcf} \]
  \[ \phi_f = 35^\circ \]
- soil characteristics sea bed:
  \[ c = 0 \]
  \[ \phi_s = 25^\circ \]
  \[ \gamma_s = 115 \text{ pcf between 0 and 20 ft below the sea bed} \]
  \[ \gamma_s = 118 \text{ pcf between 20 and 40 ft below the sea bed} \]
  \[ \gamma_s = 122 \text{ pcf between 40 and 60 ft below the sea bed} \]
GENERAL DESIGNS OF SHORE PROTECTION

- **Figure 18**

  a) GRAIN SIZE DEPENDS ON WAVE CLIMATE
  b) USE OF NET WIRING
  c) MSL SILT SMALL SANDBAGS
  d) MSL GABIONS SMALL SANDBAGS
  e) MSL LARGE SANDBAGS SMALL SANDBAGS
  f) MSL TUBES SMALL SANDBAGS
\[ k = 10^{-6} \text{ cm/sec} \]
\[ c_v = 7 \text{ ft}^2/\text{day} \]

- ice forces based on ice crushing strength across full island width;
- increase of ice thickness according to Figure 6 (the average);
- island to be built in one year, in three phases.

The adaptation of the pore pressure as a function of time was initially calculated for a load of 1 kg/cm². The pore pressure for any other load can thus be calculated by multiplying with the actual load expressed in kg/cm².

Soil Mechanical Stability of Island Slopes

The critical sliding plane and the safety factor as calculated for an island in 13 feet of water is shown in Figure 19. Similar calculations show that for 1 in 3 slopes the safety factors (the maximum possible resisting force over the driving force) are 1.10, 1.08 and 1.01 for islands in 7.5 feet, 13 feet and 23 feet, respectively. For 1 in 4 slopes these values increase by approximately 20 per cent.

Because slide failure of the slope of an island slope does not seriously harm the safety of the whole island and as this failure only occurs during the construction period, a 1 in 3 slope can be considered a safe solution.

Soil Mechanical Stability of Islands Against Ice Forces

Failure due to ice forces may occur along three planes (see Fig. 20 & Ref. 4):

a. A plane through the frozen soil. Failure through this plane means that only part of the island will be damaged, without threatening the integrity of the island as a whole.

b. A plane along the boundary of the frozen and the non-frozen fill material.

c. A plane through the sea bottom. In case the island fill consists of sand possessing a good shearing resistance, the shear plane will most likely be through the sea bed as shown in Figure 22.

From the adaptation of pore pressure with time as a result of the total load, the final effective strength can be calculated.

Since \( \tau = \sigma \tan \phi \) in which \( \sigma \) is the overburden pressure

\( \tau \) is the effective shear strength as a result of the overburden,

and \( \phi \) is the angle of internal friction,

it follows that \( \tau = 0.70 \sigma \) for sand, and

\( \tau = 0.47 \sigma \) for silt

The calculated shear strength after construction as a function of time and depth is shown in Figure 21. From this graph, the minimum values of \( \tau \) for a number of points in time were selected without regard of the depth at which this value occurred. Curve c
CRITICAL SLIDING PLANE

DISTANCE ON X-AXIS

DISTANCE ON Y-AXIS

Figure 19
RESULTANT SHEAR STRENGTHS AS A FUNCTION OF TIME

INCREASE OF EFFECTIVE STRESSES DUE TO THE EXTRA LOAD $\Delta P$

INITIAL TOTAL AND EFFECTIVE STRESSES

RESULTING SHEAR STRENGTH AS A FUNCTION OF TIME AND DEPTH $S_{x} = 0.90 \phi$ RESPECTIVELY $S_{x} = 0.23 \phi$

$\sigma_{EFFECTIVE \ \text{AFTER} \ \text{X-MONTHS}}$

$\Delta \epsilon_{T} = 2.6 \times 10^{5}$; $11$ months

12 = $531 \times 10^{5}$

13 = $850 \times 10^{5}$

14 = $125 \times 10^{3}$

15 = $172 \times 10^{3}$

16 = $226 \times 10^{3}$

17 = $359 \times 10^{3}$

18 = $618 \times 10^{3}$

19 = $961 \times 10^{3}$

Figure 21
DEVELOPMENT OF TOTAL RESISTANCE FORCE IN SEABED UNDER THE ISLAND

Graph B: Driving force due to ice and resisting force due to shear strength as a function of time (F, below seabed). Figure 22
in Figure 22 shows the minimum resistance against time calculated from these values of \( \tau \).

On this basis, Figure 22 shows that where stability against ice forces is concerned, 6 to 7 months after the beginning of freeze-up the situation may become critical. At that time all drilling operations should be completed or else defensive measures must be taken. Recent ice loading measurements have shown that the total ice load on islands may be less than used in the calculations. If these measurements can be substantiated, a reduction in the design criteria may be warranted.

Settlement of the Island Body

Settlement of the island body due to consolidation will vary from 5" for an island in 8 feet of water to 25" for an island in 23 feet.

**SLOPE PROTECTION DESIGN FOR ISLANDS BUILT IN WATER DEPTHS OF UP TO 25 FT**

The weight of small sandbags measuring a few cubic feet, used for the slope protection of islands built in water depths of 5 to 10 feet, is not sufficient as slope protection for islands built in 25 feet of water depth. This led to the development of filling gabions with these small sandbags. The behaviour of these gabions under wave attack was investigated in the wind and wave flume of the Delft Hydraulic Laboratory.

The Model Facilities

The wind-wave flume is a rectangular basin 328 feet in length and 26 feet in width. At one end of the flume irregular waves can be generated and a wind force is added to maintain the irregularity of the wave field. In the test section at the other end of the flume various slope protection configurations were tested. These tests included 2-dimensional as well as 3-dimensional layouts.

Test Conditions

The design water depth was 25 feet below chart datum. Added to this was a storm tide of 5 feet.

Tests were carried out for peak wave periods of 8 and 10 seconds and the significant wave height \( H_s \) was increased in steps to 2 to 3 feet to the maximum wave height possible under these conditions.

Gabions and Model Scale

Prototype dimensions of the gabions are 3' x 3' x 12' (4 cu yd). Filled with sandbags the weight will be about 5.5 metric tons.

For the reproduction of the gabions in the model, the following properties were considered:

- dimension and weight of the gabion;
- porosity, with respect to current velocities and pressures inside the gabion;
- stiffness of the gabion;
- the shear resistance between the gabions and in relation to the layers under the gabions.

The scale used during these tests was 1 in 25.
The Tests

The investigations covered a number of different designs.

Items varied were:
- the angle of the slope: 1 in 2, 1 in 2½ and 1 in 3;
- the placing of the gabions (Fig. 23);
- the number of layers, single or double (Fig. 23);
- highest and lowest level of the layers (Fig. 24).

Test Results and Conclusions

The tests showed that for the most exposed sides of islands in a water depth of 25 feet the use of gabions of 5½ tons requires a 1 in 3 slope and a double layer, both layers placed perpendicular to the depth contours as shown in Figure 23.

The upper limit of the top layer of gabions should be extended as far as Design Water Level + 1.75 Hₜ. For the second layer DWL = 1.25 Hₜ suffices.

With regard to wave run-up and wave overtopping the tests showed that the crest of the slope protection should also extend to DWL + 1.75 Hₜ.

It was found that the top of the submerged berm (see Fig. 24) should be DWL -1.75 Hₜ.

Figure 25 shows a typical slope protection for the most exposed side of an island in 25 feet of water.

SLOPE PROTECTION FOR ISLANDS TO BE BUILT IN WATER DEPTHS IN EXCESS OF 25 FEET

According to the IRC report (Fig. 1) the 6-hour average, 50-year Hₜ, should not exceed 15 feet at any location in the Southern Beaufort Sea. Consequently, a slope protection has to be found for significant wave heights from 15 to 17 feet.

Sand-Filled Plastic Tubes

Mattresses consisting of a number of long 3' to 5' diameter sand-filled plastic tubes are under investigation in prototype as well as in the hydraulics laboratory. Preliminary results show that these tubes may provide a solution for islands in any water depth in the Beaufort Sea. Equipment to fill the tubes and to place the mattresses is being developed. Both 2-dimensional and 3-dimensional model tests are being carried out in the Delft Hydraulics Laboratory.

The Model Facilities

The 2-dimensional tests were performed in a wave flume of 125 feet long and 3.3 feet wide, in which irregular waves can be reproduced.

The 3-dimensional tests are being carried out in a basin approximately 70 feet in length in width, likewise with irregular waves.
TYPICAL SLOPE PROTECTION FOR THE MOST EXPOSED SIDE
OF AN ISLAND TO BE BUILT IN WATER 25 FT. DEEP

Figure 24
FINAL DESIGN OF AN ISLAND TO BE BUILT IN WATER 40 FT. DEEP

SECTION OF ISLAND PLAN

A TUBES 4-5 FT.

FLAP

BAGS 7x4x2 FT.

-33.5 FT.

-40 FT.

1.75 x H DESIGN

1.80 x H DESIGN

-25 FT.

-335 FT.

5 FT.

MSL 0.00

FLAP

SAND/GRAVEL

1:3

1:5

1:6

SAND/GRAVEL

SAND/GRAVEL

CROSS SECTION AA

Figure 25
The Test Conditions

The tests have been carried out for a design water level of 45 feet. Two wave periods of $T_0 = 8$ and 10 seconds have been investigated. $H_s$ was again gradually increased in 2 to 3 foot steps, each step lasting approximately 1.5 hours in the model which is 7.5 hours at full scale.

Preliminary Results

The tests showed that the way damage occurs for 2-d tests and 3-d tests is different. The 2-d test damage consisted of the mattresses completely sliding down at significant wave heights between 13 and 17 feet once the construction was destroyed. The tests performed on tubes of different sizes showed no marked difference. During the 3-d tests, the joints of the mattresses should be strengthened by sewing them together after placement. In spite of the fact that the investigations are not yet completed, it can be said that tubes of 5 feet in diameter are more stable than gabions and should eventually prove to be able to withstand 15 - 17 foot significant waves (see Fig. 25).

ISLANDS BUILT SO FAR

As stated previously, the actual building of man-made islands in the Beaufort Sea started in 1972. Since then, Imperial has built six islands (Fig. 2).

- Immerk B-48 in the summers of 1972 and 1973
- Adgo F-28 in the summer of 1973
- Pullen E-17 in the winter season 1973/74
- Netserk B-44 in the summer of 1974
- Adgo P-25 in the summer of 1974
- Adgo C-15 in the winter season 1974/75

Immerk B-48, the first island that was built, had beaches with natural slopes of 1 in 20 and a slope of 1 in 5 from sea bed to 15 feet above sea level. Sand and gravel were excavated by the cutter dredge and pumped through a floating pipeline directly onto the island site. The slope protection consisted of filter cloth held in place by chain link fencing and submarine netting. The water depth at this location is 10 feet (Fig. 26).

Adgo F-28 and Adgo P-25 were built mainly from silt placed by clamshells within a retaining wall consisting of 2-cu ft sandbags. These islands were strictly for winter use. Their elevation was less than MSL + 13 ft and they depended on the freezing of the silt to provide stable bases for the equipment. The water depth at these locations was approximately 7 feet (Fig. 27).

Adgo C-15 and Pullen E-17 were built during the winter season. Sand and gravel were trucked over the ice from a shore deposit to the proposed island sites. Ice was cut and lifted out in blocks and the sand and gravel were dumped in about six feet of water. The slope protection consists of small sandbags. The elevation of the islands was MSL + 10 ft so that they could be used during the summer (Figs. 28 and 29).
Figure 26. Immerk B-48 Nearing Completion

Figure 27. Adgo F-28 before Freeze-up

786 de Jong/Stigter/Steyn 34
Figure 28. Removing Ice Blocks at Adgo C-15

Figure 29. Pullen E-17 Near Completion
Netserk B-44 was built in 15 feet of water outside the protective ring of barrier islands. Bunds of 2-cu yd sandbags were used to retain sand hauled in by bottom-dump barges from a dredging site 20 miles from the island location. The sand was initially dumped inside the retaining ring, but once the level of the fill material reached a height of eight feet below sea level, the sand was dumped outside the retaining ring and rehandled by clamshells.

Construction has started on three new islands in water depths of 5', 23' and 40', respectively. The first island is now complete and was built using sand and gravel. The second island is being built in the same manner as Netserk B-44. The third island, in 40 feet of water, will take three seasons to build. Construction will also start later this year on an island near the Tuk Peninsula. The stationary suction dredge will be used to build this island.

CLOSING REMARKS

This paper is only a summary of the many aspects and problems involved in the design of islands in the Beaufort Sea.

The design problems for temporary islands have been solved where water depths of up to 25 feet is concerned. For deeper water a possible solution is being tested and shows promise of being a proper solution.

These temporary islands have been built for the purpose of exploring for oil and gas. This means that when oil and gas are discovered, permanent islands will be required. Their design will demand a completely new set of solutions. Design work for this phase of the operation is already in progress.

Although this is a technical paper, it would not be complete without mentioning some of the work that has been done to make sure that the construction work does not cause permanent damage to the environment. During the past three years, baseline studies have been prepared by independent environmental consultants on both the animal and the plant life in the area. The studies also cover the effect of our activities on the environment and wherever possible, follow-up investigations are made annually.

The siltation caused by the dredging operations is minor in comparison to and indistinguishable from the silt load dumped into the delta by the Mackenzie River.

The island construction work takes place within the migratory paths of the Beluga Whales but three years of study have shown that these animals are not concerned about routine boat traffic, and there is no indication that our operations have influenced the success of the native whale hunters.

The benthic organisms normally have a hard time surviving in the shallow near-shore waters where ice freezes to bottom every winter. The formation of burrow pits in this area by the dredges and the shelter that islands provide in deeper water combine to enhance the aquatic life in the offshore area.

The authors hope that by their paper they have shown that "Building islands in the Beaufort Sea is not a matter of simply dumping silt or sand in this shallow sea", but that thorough preparation behind drawing tables and in laboratories together with the practical experience gained during the construction work itself are essential for the successful completion of the islands and the safe drilling of the wells.
REFERENCES


ABSTRACT

Scope of the present lecture is to discuss certain experiences from DnV's involvement in safety verification of North Sea Structures.

The experience has been gained from the 60 fixed platforms in the North Sea surveyed by DnV.

DnV's general requirements to fixed offshore structures are given in DnV's "Rules for the Design, Construction and Inspection of Fixed Offshore Structures" (1974), to which reference is made. In the present lecture primarily new trends not covered by the rules are discussed.

Primarily, aspects concerning the special loading conditions of offshore structures are accentuated.

NOTE: The author has requested this paper be read in conjunction with the related papers, "North Sea Offshore Structures", by P. Bruun (p. 719) and "North Sea Offshore Structures Environment and Environmental Loads", by B. Pedersen (p. 875).
EXTERNAL SAFETY VERIFICATION

General

In spite of the fact that offshore oil production has been going on for some decades, there has been very limited legislation to ensure certain safety criteria be met, although such regulations have been rather common with respect to onshore structures, such as pressure vessels, pipelines, buildings, bridges.

Concern for pollution has increased markedly in recent years. Both air, land and water pollution and the degradation of our environment have become increasingly important social issues. In this respect as well as with respect to the safety of men, the consequences of failure of an offshore structure or installed equipment may be very considerable. Therefore, all offshore activities related to oil and gas should be carried out by the best possible technical equipment, and all possible precautions taken to increase safety. Advanced engineering techniques designed to increase reliability should be applied, as we are dealing with constructions exposed to heavy environmental loads with intricate mechanical, electrical, and hydraulic systems.

It should also be borne in mind that the offshore activities are expanding into more and more difficult areas with increasing problems related to environmental conditions, water depth, foundation and sea bed conditions, etc.

The North Sea bordering countries have realized the need for some kind of involvement in the activities going on on their continental shelves in order to make sure that reasonable safety measures are taken by all parties participating in such industry.

Regulations

In the United Kingdom the Secretary of State under Mineral Workings (Offshore Installations) Act 1971, has the power to make Construction and Survey Regulations requiring offshore installations to be certified as fit, for the purpose specified by the Regulations. The "Offshore Installations (Construction and Survey) Regulations 1974" lay down the relevant technical requirements and specify that each offshore installation, when in waters around the United Kingdom, must possess a certificate issued by a Certifying Authority stating that it is fit for use in those waters. The following five organizations have been appointed as certifying authorities: Lloyds Register of Shipping, American Bureau of Shipping, Bureau Veritas, Det norske Veritas, and Germanischer Lloyd.

Concerning fixed platforms, Lloyds and Det norske Veritas have been the dominating authorities.

In Norway, the government has the right to require all structures and equipment used for oil and gas exploration and production to have special approval prior to use and has practiced this right from the very beginning of the development in the Norwegian waters.

The practice of certification and requirements applied are very much the same in British and Norwegian waters and it includes a detailed scrutiny and inspection of all phases of design, construction and operation i.e., environmental conditions, structural analysis and design, fabrication of components, assembly and construction, soil investigations and foundations, installations as well as of process equipment, safety systems, etc.

Det norske Veritas has acted as the main consultant of the Norwegian Government but other consultants are also involved. Governmental requirements are also in existence, or under preparation in other North Sea countries, as well as in other parts of the world.

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As a basis for the quality assurance work on behalf of governmental bodies in different countries, DnV has issued rules for the design, construction, and inspection of offshore mobile units, fixed structures, pipelines, subsimmersibles and diving systems, which serve as supplements to any existing national requirements.

In our experience the oil companies' response to safety requirements in an increasing number of countries has in general been very positive.

LOAD CONSIDERATIONS

Load Conditions

For offshore structures as built today, temporary load conditions during construction are governing to a greater extent than usual for onshore structures. As an example the construction phases for a concrete platform of general tower and caisson type are sketched in Figure 1.

1. Construction in dock

The structure shall carry dead load, prestressing forces and possible cranes. This phase generally is not governing for main dimensions.
2. Structure as launched

Depending on the ballasting program, this phase can be governing for certain walls. If the bottom slab is cast in two steps the water pressure in this condition governs the dimension of the first step. If an air cushion is applied to reduce the draft, the skirts will be exposed to lateral forces.

3. Immersion for deck installation

This phase gives the maximum external compression on the tower and the roof and walls of the caisson. This means that even the inner walls should be checked for compression.

4. Towing

This phase is generally not governing for any structural element. If leak stability is a criterion, internal walls can be governed by this phase. The tower may be checked for extreme pitching.

5. Installation

Possible dowels obviously will be governed by this condition. Installation is generally governing the bottom slab. As described below considerable pressures can develop between bottom slab and sea floor. If suction under the slab is used to obtain complete penetration, this may be another governing load case. This suction can amount to 15 to 20 t/m².

If the interspace between bottom slab/sea floor is grouted with a grout layer of considerable magnitude, the bottom slab and adjoining walls should be checked for temperature stresses due to hydration heat during hardening of the grout.

The compression in the skirts during penetration is to be designed in combination with corresponding horizontal shear.

6. Operation

This final phase introduces maximum forces in the deck and the tower. For the upper part of the tower shock pressure and ship collision criteria are critical. The tower foot is governed by extreme wave or fatigue considerations.

In case of soft soil in the upper subsoil layers the horizontal force is to be transmitted to soil of sufficient strength at a certain depth. This means that the skirts should be designed to take the complete horizontal force.

The earth pressure at rest is greater beneath the bottom slab than outside the structure. During application of the wave force this pressure increases further. This means that we must design for an increased difference in earth pressure acting together with the wave force.

In case of oil storage the temperature effects will be governing for the necessary reinforcement of inner walls and roof.

If the caisson design is based on external water pressure the integrity of the structure is to be checked for an accidental removal of the overpressure.

However, the only parts of the structure governed by static strength capacity in the operating condition are deck, tower and possible shear transmitting skirts. This
means that for the long-term safety of the structure during operation, durability of
the structure and effect of the repetitive character of loads are decisive rather than
the direct strength capacity. Those responsible for the safety in this phase have to
pay special attention to the long-term effects.

Waves

For most offshore structures wave loads account for the major part of the environmental
loads.

On fixed offshore structures the load is normally evaluated by means of one of two basic
methods: (a) design wave method, and (b) spectral analysis method. The method to be used
should be considered in each specific case with due regard to the design and purpose of
calculation. As the spectral analysis is considered basically most correct, the trend is
in favor of this approach when applicable. For further description of different ap­
proaches for wave force determination, reference is made to a paper of the present con­

When determining the deck level DnV requires account to be taken of the tolerances ex­
pected in determination of water depth, long-term foundation settlements, subsidence due
to pressure release in the reservoir, increase in crest elevation due to reflected waves
from caisson and towers, and uncertainties in wave crest elevation. It is important to
note that the mentioned items also have to be considered in the calculation of overall
overturning moment.

For a concrete structure, too low deck elevation may cause local damage. For a steel
jacket structure, a wave hitting the deck would be disastrous because the overturning
moment caused by drag forces on the structure and on the deck would occur simultaneously.
Hence DnV tends to pay more attention to the air gap for steel structures.

Current

Adequate current velocity data are also to be selected from available statistics. Two
major components of current are to be considered; tidal current and wind driven current.

In open areas where statistical data are not available, the wind driven current at the
still water surface has been accepted as 1 percent of sustained wind at 10 m above still
water level. The combined loads exerted by waves and currents are to be computed on the
basis of a vectorial addition of current velocity and wave particle velocity.

Wind

Wind speed statistics are to be used as a basis for description of wind conditions as far
as such data are available. In general, wind conditions at sea are more regular than on­
shore.

Two kinds of wind speeds are normally considered:

a) Sustained wind speed which is defined as the average wind speed during the time
interval of 1 min. Sustained wind is to be used in conjunction with maximum
wave force.

b) Gust wind speed which is defined as the average wind speed during a time inter­
val of 3 sec. Gust wind is to be used if more unfavourable than sustained wind
in conjunction with maximum wave force.

DnV has encouraged a rational wind load analysis based on wind spectra (Fig. 2). An argu­
ment against this approach is that recognized spectra have been based mainly on onshore
data. However, similar arguments may apply to the "gust factors" etc., introduced in the traditional wind load analysis.

Shock Pressures on Columns

Shock pressure is impact pressure arising when a breaking wave hits a relatively large structure. This load case is considered to be of great importance for the local strength. Investigations have recently been carried out concerning the probability for breaking waves in deep water. Breaking waves were defined as waves with the relationship $H_b/T_b^2 > 0.27 \text{ m/s}^2$ (Fig. 3).

The results from this general investigation show that the most probable largest breaking wave height in a 100 year period is $H_{b100} = 1.4 \cdot H_{1/3}$ where $H_{1/3}$ is the significant wave height in the design storm spectrum.

The corresponding wave period is as given from the above relationship. The magnitude of $H_{b100}$ may of course vary from area to area, but the above figures may be used unless other documentation is provided.

The intensity and exposed area of the shock pressure will depend both on the structure and the breaking wave characteristic. Some information about this may be obtained from theoretical evaluation and in situ measurements on breakwaters and lighthouse. Unless more sophisticated calculations are provided, the basic equation for pressure may be used.
The shock pressure coefficient, $C_s$, should be chosen according to shape and dimensions of the exposed structure.

For platforms recently certified in the North Sea shock pressures of 30 t/m² have been accepted by DnV on towers of 10 to 15 m diameter. This corresponds to a value $C_p \approx 3.0$.

**Slamming**

Slamming is defined as wave impact loads on bracings at, or penetrating the waterline. Horizontal bracing and conductor framing should be located so as to minimize the effect of slamming.

The vertical bending stress component in horizontal bracings may be several times greater than the stresses resulting from the environmental loads normally allowed for in the design. Besides the impact, slamming causes vibration in the bracings. The result may be a short fatigue life and fatigue cracks.

Much research work is still to be done in this field. However, DnV has accepted the following approach (Fabula et al., 1955);

For the slamming load the same basic pressure equation may be used:

$$P_s = C_s \gamma \frac{V^2}{2g}$$

Where:
- $P_s =$ vertical slamming pressure
- $C_s =$ slamming coefficient
- $V =$ vertical water particle velocity at centerline

The slamming coefficient should not be chosen less than 3.0. This slamming load is to be added to the local vertical waveload.

**Soil Reaction on Base Slab**

For the large foundations of gravity structures resting on the sea bed the consequences of a non-uniform stress distribution against the base slab need careful investigation. Some reasons for a non-uniform distribution are:

a) for moderate loads the soil behaves like an elastic medium, which indicates stress concentrations at the exterior and relief at the interior of a footing;

b) for foundations on sand the bearing capacity is zero at the edge of the footing; and

c) when the sea bed is not entirely flat and a rigid foundation is placed against it, larger stresses will develop at the highest contact points as these are pressed down under the weight of the structure.

DnV's experience is that the third case normally causes the highest local stresses.
A rigorous elasto-plastic analysis based on assumed shape of sea floor projections results in extremely high pressures (Fig. 4). Experience from the installation of the Beryl A platform indicates that the calculated values are realistic. A common method to avoid extreme pressures, is to monitor the soil pressure or the stresses in reinforcement during installation and stop the penetration for grouting when design pressure is reached. Otherwise the high pressures have to be designed for.

Figure 4. Local soil pressures at conductors.

Conductors and Risers

When settlements occur in the soil under a gravity structure after the conductor pipes have been installed, downward (or negative) friction forces may develop along the conductors. If the conductor is not restricted against downward movement the friction forces have to be delivered as positive skin friction and point resistance at the lower end of the conductor pipes. The stresses set up in the conductors may approach yielding.

On the other hand the conductors reinforce the soil under the base slab. This means that the area around the conductors will be a hard point with considerably increased soil pressure (Fig. 5). It is to be documented that these stresses do not exceed the design stresses for the slab. Similar pressures will be caused by thermic expansion when the conductor is filled by hot oil.

The shear strain caused by wave loading sets up stresses in risers and conductors, being restrained by the soil under or around the platform. Unless special measures are taken, similar stresses will be introduced in the platform bottom section. These stresses are to be determined and considered in the design.

Impact Forces from Floating Objects

Although the activity in the North Sea has lasted for a relatively short time, several platforms, both mobile and fixed, have been damaged due to impact loads from boats.
In order to pursue the design philosophy one should try to obtain the actual risk for impact loads on platforms and design according to this. Due to lack of data, no such probability method can be developed for some time. However, DnV is at present intensively collecting and interpreting data on relevant accidents.

Until results from these investigations are available one has to agree on reasonable load-cases based on present knowledge and experience. This means to settle on a design boat size, strength and impact velocity.

The design boat size is to be chosen on the basis of expected ship traffic in the vicinity of the platform. If ships in general are not allowed to approach the platform and the restricted zone is sufficiently wide an accident will most probably be caused by a supply boat. An estimate of the size of future supply boats indicates boats of about 2500 tons displacement.

Under the assumption that pipe-laying barges etc., are sufficiently protected, 2500 tons have been accepted as design value.

In order to obtain realistic load conditions two cases may be considered:

a) An operational load with an impact energy which the platform should withstand without damage, by use of fenders or by its own structural strength.

This load may, for instance, be caused by a supply boat when approaching or leaving the platform. Based on available data for velocity of berthing ships a design velocity of 0.5 m/sec has been accepted by DnV for the operational load case.
b) An extreme load for which some damage may be allowed, but the platform as a whole should be constructed so as not to collapse. Such a situation may for instance, occur if an out-of-control boat is drifting against the platform.

The impact velocity may be determined by adding the effects of wind, current, and waves on the boat and by taking into account the actual response of the boat in these load cases. The weather conditions could be those for which the supply boat has to stop operating and move away. The impact velocity resulting from such a condition has been accepted in the order of 1.5 to 2.0 m/s.

The sector to protect should be based on the intended approach direction to the supply boats with a safety sector on each side. The necessary height of the fendering system will depend upon factors such as wave crest and wave trough, mean tide elevations (fixed structure), relative motion platform/boat (mobile platforms), depth of fender beneath minimum waterline and height above maximum waterline. For safety, an extra length should be added at the top and bottom.

As a possible alternative design approach DnV has studied which force would cause crushing of the ship. These investigations are still not final, but the evaluation seems difficult and results in high loads, e.g., for a ship filled with bulk material.

Temperature Stresses

If the caisson of a concrete gravity structure is utilized for storage of oil this generally sets up temperature stresses in the structure. Similar temperature differences are also introduced around the conductors with hot oil or gas heating the surroundings. The magnitude of the stresses depends on the oil temperature which, depending on the degree of cooling, is in the range from +20°C to +50°C. The design temperature of water is in the range of +4°C to +10°C. The heat flow from the hotter to the cooler parts of the structure tends to reduce the temperature gradients. This means that the most unfavorable conditions are associated with changes in the oil level. When overstressed in tension, the concrete cracks. The cracking changes the stiffness of the structure and partially relieves the stresses. Altogether, a rigorous stress analysis would be an extremely complex investigation based on thermodynamics and a step by step iteration of the cracked structure.

Much development work is anticipated before this type of analysis can be performed with high accuracy.

So far, accepted practice has been to perform a structural analysis disregarding heat flow and based on the assumption of an elastic material with instantaneous modulus of elasticity. The section analysis is based on forces and moments reduced in accordance with an estimated reduction in stiffness due to cracking. The correctness of this procedure is dependent on the formation of a systematic crack pattern.

If the amount of reinforcement is small and the thermal forces dominating in an element subject to almost pure tension, a limited number of cracks of unacceptable width may occur. To prevent this situation such an amount of minimum reinforcement is required that a force corresponding to the capacity of the concrete in tension may be transmitted by acceptable steel stresses (i.e., crack widths).

**DYNAMIC EFFECTS**

**General**

The importance of the dynamic effects is rapidly increasing by increasing water depth. The effects concern the design of all parts of the structure. In fact the dynamic
amplification of the quasistatic member forces is more pronounced for the upper part of the structure than for the overall forces. Amplification factor as high as 1.6 on the design wave has been calculated for concepts evaluated by DnV. For resonance amplification up to 30 is not uncommon.

In practice the dynamic analysis has to be performed by sophisticated computer programs, e.g., Stadyne, Sap, Condyn. Several programs have been tailored for offshore structures.

The details of the analysis model are determined by the complexity of the actual structure (Fig. 6). Besides the determination of the forces in the primary structure, it is also imperative to estimate the dynamic behaviour of deck structure, conductors, risers, etc. All locations, where such secondary structure parts are attached, are to be investigated for dynamic effects.

![Figure 6. Dynamic analysis model.](image)

In principle, all main members of the deck structure shall be included in the analysis. However, in practice the deck structure may be included in the analysis by somewhat simplified methods. A simplified static analysis with forces and moments in the lower end of the deck supports taken from the total dynamic analysis has been accepted by DnV.

The foundation soil may be modelled either as an elastic continuum or by three-dimensional finite elements. In the first case an elastic half-space or a layered system is used,
depending on the actual soil profile. In finite element models particular attention should be paid to the representation of the boundaries and to the non-linear stress-strain behaviour of the soil.

Stiffness Properties

If more accurate investigations are not carried out, the stiffness of the concrete structure is to be evaluated on the basis of net concrete section and short times modulus of elasticity as given in the applied design code or as determined by tests of the concrete on the site.

The shear modulus of the various soil layers identified in the foundation of the structure may be evaluated on the basis of; (1) laboratory tests on undisturbed samples of the actual soil, (2) empirical correlations between the shear modulus and index properties such as void ratio, shear strength, etc., and possibly (3) in situ measurements of the shear wave velocity. Particular attention should be paid to the variation of modulus with strain level (Fig. 7), and to the effects of a large number of repeated loads on the soil properties.

![Figure 7. In situ shear moduli for saturated clays.](image)

Due to the uncertainty in the assessment of the soil properties a parameter study should be carried out to establish the influence of each parameter. In the final analysis conservative values should be used for the critical parameters.

Damping

Energy dissipation may be represented by equivalent viscous dampers or modal damping factors. Evaluation of the damping coefficients are to be based on material, stress level and the type of connections used in the structural system.

The composite damping is to be evaluated on the basis of recognized principles as a weighted mean value of the contributing materials. If the difference in material damping for the different materials (or soil) is significant, this procedure may lead to erroneous results and the applicability of the approach is to be verified from case to case.
The damping properties of the materials is strain dependent. In practice this fact can be taken into account by choosing one set of factors for the design wave and another for smaller waves investigated in connection with fatigue.

When structure-foundation interaction is considered, the foundation damping essentially consists of two types of damping — radiation damping and internal material damping of the foundation medium.

The radiation damping may be calculated theoretically for foundations on an elastic half-space but for actual soil profiles considerably smaller radiation damping than this theoretical value should normally be used.

The magnitude of the hydrodynamic damping may be calculated by considering the drag force (also called damping force) on the structure. The quadratic drag force may be replaced by an "equivalent linear" drag force, defined as the linear drag force which dissipates the same energy per cycle as the quadratic force.

Forcing Functions

The forcing functions should include all loads due to waves, current and wind. In this way the dynamic forces and moments may be used directly for design avoiding the concept of "magnification factor".

If the drag force is of minor importance compared to the total force the drag force may be linearized and a stochastic process used to determine the dynamic response.

If a deterministic approach is chosen the response due to design wave condition should be based on an assumed realistic time history, i.e., also the transient part of the dynamic response is to be included.

LONG-TERM SAFETY OF THE STRUCTURE

General

The combination of aggressive sea water and repetitive loads may raise the question whether the structure can maintain the required safety level during its planned life time.

For steel the fatigue and corrosion properties are well established. In general even the possibilities for inspection are fairly good.

For concrete, however, the approach for durability control has not achieved the same general acceptance. Different codes express their SLS-criteria in different ways. The criteria shall be controlled for some apparently arbitrarily chosen "operating wave" as a substitute for a rigorous analysis of the effects of repeated loading. The uncertainties regarding the long-term behaviour of the structure is reflected by the difference in SLS-criteria laid down in different codes.

Fatigue of Concrete

Concrete in direct compression. Wöhler and Goodman diagrams for direct constant compression are very well established, however, based on test results of considerable scattering (Figs. 8 and 9). An important side-effect is that if the maximum stress remains below a certain value, the cyclic loading seems to increase the strength in subsequent short time compression. During the repeated loading the concrete is subject to a plastic strain increasing with number of load cycles. The rate of strain increases rapidly by increasing maximum stress level.
Concrete in bending compression. If the stresses are varying linearly over a section, the maximum edge stress does not seem decisive for the fatigue life on the beam. This finding probably is associated with a stress redistribution due to the above mentioned plastic strains. DnV has accepted that edge stress is disregarded in the fatigue analysis and accepted a stress distribution over the section as laid down for the ultimate limit state.

Shear. The amount of research work on fatigue properties of concrete in shear is modest. However, it can be concluded that the reduction in shear strength is more important than for compression strength. For shear reinforced members subject to pronounced repetitive loading the shear capacity contribution from the concrete may, as a conservative assumption, be ignored. As a guidance to the importance of the problem Wöhler diagrams similar to the ones for concrete compression could be constructed. These diagrams will have a more rapid declination by increasing number of cycles than for concrete compression.

Bond. The reduction in bond under repeated loads is not less pronounced than for shear. This fact is established by several tests, but the research work available is not sufficient to draw conclusions quantitatively. Possible Wöhler diagrams would have about the same declination as for shear. DnV has accepted the thumb rule that anchorage and lap lengths are doubled for fatigue loading compared to static load. Naturally the resistance depends on the shape of the bars and the state of their surface. High bond bars have relatively lower loss of resistance than smooth bars, and too great corrosion can entail an important reduction in bond stress due to fatigue.
Reinforcement. In general DnV requires special fatigue test of tendons and reinforcement with high repetitive stresses. The fatigue characteristics of reinforcement bars are fairly well known qualitatively, but the details call for special investigations in each case. The fact that bending the bars reduces the fatigue life considerably is to be taken into account. This reduction is of special importance for stirrups with small corner radius. Prestressed concrete designed to class 1 or 11 fatigue of steel does not seem to be a problem.

Cumulative damage. The criteria of Miner relative to cumulative damage does not seem to be directly applicable to concrete. Tests have shown that ruptures may occur after load cycles corresponding to a Miner factor of 0.4 to 2.0 depending on the load sequence (Fig. 10). As a conservative approximation DnV has suggested a Miner factor of 0.2 as design criterion for concrete structures. It is difficult to reveal incipient fatigue of a concrete structure by inspection. For this reason DnV requires the Wohler curves obtained by tests reduced for design by a material factor of same magnitude as for static loads.

Figure 10. Cumulative effect of repetitive loads on concrete.

Effect of cracks. When a beam is subject to repeated load in air, it can be noted that the cracks open and close at each loading cycle without any deteriorating effect. On the other hand, if we pour water on the cracked face of such a beam, we quickly note a deterioration of the surrounding concrete. This effect seems due to the fact that, when the cracks open, water can penetrate into them but when they tend to close again capillarity...
occurs which prevents the evacuation of the water. This water then plays the role of a wedge which tends to make the surrounding concrete split. Naturally this phenomenon only takes place in the case of cracks open enough for the water to penetrate into them.

So far we have only a qualitative knowledge of this effect; systematic tests are presently carried out by DnV to determine the dangerous limits of cracking. In this situation DnV has promoted a conservative attitude to this problem, accepting only minor bending tension in members exposed to pronounced repetitive loading.

**Design considerations.** Finally, it should be mentioned that the most important factor to prevent fatigue problems is proper reinforcement and design detailing. This means that changes in sections, anchorage and other reinforcement details, clearance between bars securing adequate bond and complete grouting etc., should receive much attention.

**Corrosion of Reinforcement**

The general principles for corrosion of steel in concrete are recognized, whereas the quantitative effects of the different aspects are not completely understood. Recognized design procedure is to aim at sufficient cover and limited crack widths. DnV feels that the attention of the designer should also be turned to other important aspects such as, the effect of stray currents, and inhomogenous concrete giving local galvanic cells.

Under the water pressure in question sea water will penetrate to the reinforcement after some time irrespective a cover of 5 to 10 cm. However, this is not likely to promote corrosion provided the concentrations of chlorids and the accessibility for oxygen is low. Experience has shown that sufficient cover of concrete of low permeability, e.g. $<10^{-12}$ m/sec is the most effective remedy to prevent reinforcement corrosion.

The significance of crack widths as a criterion for danger for corrosion has been discussed. However, recent tests on reinforced concrete stored in the splash zone in the North Sea and in NaCl solution indicate a crack width of 0.15 mm as a limit. Crack widths below this limit are normally not harmful and even certain healing effects can be observed in a moist environment (Figs. 11 and 12).

![Figure 11. Corrosion as a function of crack width.](image-url)
Figure 12. Corrosion as a function of crack width.

The problem is more complicated for concrete subject to cyclic loading with repeated widening and closing of cracks and possibilities for fatigue corrosion of the steel. However, experiments on the fatigue performance of reinforced concrete saturated with sea water are indicating that provided recognized design practice as regards cover and crack widths are applied, no corrosion fatigue should occur. These tests are as yet incomplete.

No reliable procedure to predict the crack width in concrete members of the present dimensions and cover exists. If the generally accepted formulae were applied, they would have promoted application of bars of small diameters and reduced cover. This result would not be in accordance with good engineering. Therefore, DnV has taken the attitude that a limitation of the steel stresses, which seems to be the governing factor, is the most reasonable criterium. The stress limits have been chosen so that the crack widths should be kept below 0.1 to 0.15 mm.

Coating

The most severely exposed part of the structure is the splash zone exposed to repeated wetting and drying. It has been suggested that this part of the structure should be given extra protection by application of some surface coating. The same remedy has been suggested when, for some reason, the required cover is insufficient. The coating is intended to keep the concrete water-proof and in some cases, to protect it against possible erosion due to streaming water and possible floating particles. It is important to note that holidays would admit uneven penetration of oxygen thus promoting corrosion rather than preventing it. Practice has shown that coatings recommended for this purpose by the suppliers spalled after relatively short time. Therefore, DnV requires a comprehensive documentation of the bond to concrete properties and the mechanical resistance. The coating must have sufficient ductility to remain intact after cracking of the concrete, and great resistance to aging. Preferably the coating material should be absorbed into the pores of the concrete. If the coating is intended to protect the splash zone, the coating should cover the complete splash zone up to the concrete/steel joint.
REFERENCES


Det norske Veritas. 1975. Rules for the construction and classification of mobile offshore units.


Whitman. 1972. Analysis of the soil structure interaction. MIT.
WAVE LOADS ON GRAVITY PLATFORMS

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ABSTRACT

This paper presents theoretical calculations and experimental values of wave induced pressure, force and moment on a gravity platform. Further, theoretical values of free surface profile and theoretical and experimental values of wave induced velocities above the caisson are given.

A study of minimalization of overturning wave moment for different water depths is presented.
Theoretical methods

Different methods with different form of accuracy and simplicity exist to calculate wave induced velocities, pressures and forces for gravity platforms.

For the force distribution on columns the simple Morison's equation (Morison et al., 1959) is usually employed. For this purpose it might be considered an accurate method. However, for brackets where drag forces are important, the accuracy is questionable. On the other hand there exists no other methods that take proper account of the viscous effects. For the caisson of gravity platform the accuracy of the Morison's equation is also questionable since it does not take proper account of the reflection and diffraction of the incident regular waves by the caisson. The most accurate method for the caisson is the three-dimensional sink-source technique. It seems to be generally approved that the accuracy of this method is comparable with the accuracy of model tests.

The method consists in distributing Green's functions with different densities over the wetted surface of the caisson. The density function is found by solving a 2-dimensional Fredholm integral equation of the second kind over the surface. This equation results from satisfying the body boundary conditions. In the numerical calculation the body surface is approximated by a finite number of surface elements.

To our knowledge Lebreton and Cormault (1969) were the first to apply this kind of method to offshore structures. But the knowledge of the method and the appropriate Green's function to be applied has existed for a much longer period of time. The reason why the method was not applied before was mainly limitations in computer capacity, but of course also due to the lack of any important need for such a method. It is really the offshore activities in the North Sea which has provided the need.

In connection with this paper we have used the three-dimensional sink-source program which is available at Det Norske Veritas and is described in Faltinsen and Michelsen (1974). This program handles fixed objects, like a gravity platform, as well as floating objects and it calculates both forces, moments, water velocities, pressures and free surface elevations in regular sinusoidal incident waves.

The three-dimensional sink-source method is based on potential flow theory and small amplitude waves. For the caisson of a gravity platform these restrictions are not of any significant practical importance.

The three-dimensional sink-source technique is timeconsuming compared with other methods. Typical calculations for a gravity platform will, on a UNIVAC 1108-computer, be of the order of one minute for each wave period.

The method presented by Gran (1973) is a much less timeconsuming method. It is not as accurate as the three-dimensional sink-source technique, but for preliminary design purposes it should be accurate enough.

The method is based on potential flow theory and incident regular waves of small amplitudes and is applicable for a truncated vertical circular cylinder standing on a horizontal sea-floor. It is a long wave approximation but it is applicable for smaller wave lengths than the Morison's equation.
Another method which may be used is a semi-empirical method using correction factors to the Froude-Kriloff force and moment.

Comparison between theory and experiment

In order to check the three-dimensional sink-source technique model tests were run in scale 1:100 with the caisson shown in Fig. 1. The tests were done at The River and Harbour Laboratory in Trondheim and was started as a master thesis by Gravvold and Haugsoen (1973) and later continued by Gravvold.

Regular waves with periods ranging from 1.2 sec to 2.0 sec and wave heights ranging from 5 cm to 24 cm were used. Measured values were taken as half the peak to peak reading of the output. In the force calculation 48 plane elements were used to approximate the wetted surface of the caisson and in the pressure and velocity computation 108 plane elements were used.

Figure 1. The caisson
Pressure

Pressure was measured at the 12 points shown in Fig. 2. For each run 6 oscillations assumed to be nearly steady state oscillations were used as a basis for estimating pressure amplitudes. Due to nonlinear effects, reflection in the wave tank, inaccuracies in the measuring technique it is impossible to measure 6 oscillations of constant amplitude. The standard deviation of the amplitudes was found to be about 4%.

To measure the difficulty in setting the right period and wave height on the wave generator the tests were repeated four times for each pair of wave period T and wave height H. As a measure of this difficulty a standard deviation $\sigma$ was calculated and is presented in this paper.

The results of measured and calculated values for the two wave periods 1.5 sec and 1.8 sec corresponding to full scale values of 15 sec and 18 sec are presented in Tables 1 and 2. The wave heading is $0^\circ$ which means that the waves are propagating along the positive x-axis (See Fig. 1). The numbering of the pressure points refer to Fig. 2.

![Figure 2. Pressure were measured at the points 1 to 12](image)

The pressure values are non-dimensionalized by $\rho g H/2$ where $\rho$ is mass density of water and $g$ is acceleration of gravity. Since the three-dimensional sink-source technique is a linear theory based on a small wave-height assumption, the calculated values $p_c$ are independent of the wave height.

Taking into account the inaccuracies in the modeltest due to difficulties...
in reaching steady-state oscillations and repeating experiments, one may in general conclude that there is good agreement between theoretical and experimental values.

**Force**

Calculated and experimental values of the horizontal and vertical amplitudes $F_h$ and $F_z$ are presented in Table 3. The forces are non-dimensionalized by $p g V H/2a$ where $V$ is the volume of the caisson and $a$ is explained in Fig. 1. Regular sinusoidal waves of periods $T = 1.3, 1.5, 1.8$ and $2.0$ sec were used and the wave headings were $0^\circ$ and $45^\circ$ with respect to the x-axis.

Due to the measuring technique, forces acting on the lower 2.5 cm of the model was not registered. In order to make comparisons the calculated values were adjusted for this effect. It is seen that the agreement between theory and experiments is good.

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TABLE 1. Local pressure nondimensionalized by $p g H/2$. Wave period 1.4 sec.
TABLE 2. Local pressure nondimensionalized by $p_g H/2$. Wave period 1.8 sec.

<table>
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<th>% of $P_c$</th>
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*These are the arithmetic mean of only three tests.
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<th>Heading 45°</th>
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</tbody>
</table>

**TABLE 3 Forces on the caisson. Nondimensionalized by $pg \frac{V}{H}/2a$**
Velocity

Horizontal water velocity \((V_x)\) was measured by an ultrasound current meter at the eight points shown in Fig. 3. Results of measured and calculated values for wave heading \(0^\circ\) and wave period \(T = 1.5\) sec are shown in Table 4 and Fig. 4. The values are nondimensionalized by \(H/2 \sqrt{g/a}\). The calculated values in Table 4 are by three-dimensional sink-source technique or what is called diffraction theory in Fig. 4. Calculated values of undisturbed velocity profiles of regular sinusoidal waves on water depths \(d=80\) cm and \(40\) cm have also been plotted in Fig. 4 to try to illustrate the effect the caisson has on the velocity field. The case of \(d=40\) cm would correspond to letting the sea floor coinciding with the top of the caisson.

It is seen that the measured values are generally quite close to the values calculated by diffraction theory with the exception of the \(24\) cm high waves. In the latter case the waves has a tendency to break on top of the caisson and good agreement should perhaps not be expected. The measured values are closer to the calculated undisturbed velocity profile for water depth \(40\) cm than the calculated undisturbed velocity profile for water depth \(80\) cm. So one might in this case be tempted to say that the caisson's effect is very much like changing the waterdepth.

Figure 3. Sketch showing where the velocities were measured.
TABLE 4. Measured and calculated velocities.

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<th>pkt.</th>
<th>H</th>
<th>Meas. $V_m$</th>
<th>Calc. $V_c$</th>
<th>Diff. % of $V_c$</th>
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Figure 4. Measured velocities plotted against different theoretical velocity profiles.
Forces on columns

Having obtained an indication that the three-dimensional sink-source technique calculates velocities and hence acceleration very well above the caisson, we can have some faith in using those velocities and accelerations together with Morison's equation to calculate forces and moments on columns on the caisson. To further check this procedure model tests were performed with a column of diameter 20 cm placed on the top of the caisson (see Fig. 5). Both horizontal force and overturning moment with respect to an axis of the sea floor were measured. The wave heading was 0°. In the calculations a $C_m$ value of 2 and $C_D$ value of 1 was used in Morison's equation. Results of the calculations and the experiments are shown in Table 5. For the lower wave height the agreement between theory and experiments is very good.

Figure 5. The caisson with surface piercing column.
TABLE 5. Forces and moments acting on the column.
Forces are divided by \( pg \ VH/2D \), moments by \( pg \ VH/2/10^3 (V = D^2/4) \).

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<th>Moment</th>
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Free surface profile

Diffraction effects caused by the presence of a caisson may result in a noticeable increase in the wave crest elevation compared to the incident wave. This increase may be important to consider when calculating forces on the columns, and also when determining the deck elevation above the still water level.

Among the various parameters determining the wave crest increase, the wave period and the ratio caisson height to water depth are the most important. Within a certain range of periods, the wave crest elevation will increase significantly with increasing period, reach a maximum and then decrease slowly. Keeping the water depth constant and increasing the caisson height will also result in increasing wave height above the caisson, until the wave will finally break.

The wave crest elevation may be calculated using the three-dimensional
sink-source technique, but in this paper Gran's method (Gran, 1973) has been used. A comparison between those two methods have been done by Fines and Flageland (1975), showing relatively good agreement, especially for long waves. Because of its simplicity Gran's method is advantageous to use when making parameter studies. To illustrate the effect of various caisson height to water depth ratios, results of wave crest increase for truncated vertical circular cylinders of different heights \( h_1 \) are presented in Fig. 6. The wave crest increase is given in per cent and is defined as

\[
\frac{a_c}{a_i} \times 100
\]

where

- \( a_c \) = amplitude of wave taking into account the influence of the caisson
- \( a_i \) = amplitude of incident wave

The calculations were done for seven points along the wave direction, over a cylinder diametre.

The form of the horizontal cross-section (i.e. if it is square, circular, hexagonal etc.) has been found to be of small importance for the wave crest increase (Olsen & Hessen, 1974). Gran's method may therefore also be applied on a caisson of square form to determine the wave profile. In doing so, it is suggested that the volume of the equivalent cylinder is set equal to the volume of the square one, i.e.

\[
a = \frac{L}{\sqrt{\pi}}
\]

\[
h_c = h_s
\]

where

- \( a \) = radius of cylinder
- \( L \) = length of square caisson
- \( h_c \) = height of cylinder
- \( h_s \) = height of square caisson

This method was used to find the wave crest increase over the platform shown in Fig. 7. This was done for the periods 9, 12, 15 and 18 seconds. It is seen that the maximum value of the increase occurs for \( T = 15 \) s, and it can also be noted that the peak tends to occur nearer the forward wall of the caisson at increasing periods. This effect is believed to be a general conclusion (Olsen and Hessen, 1974). The maximum value of the increase for each wave period has been plotted as a function of period in Fig. 8.
Figure 6. Wave crest increase for truncated circular cylinders.
Figure 7. Wave crest increase over a square caisson.
Figure 8. Maximum value of the wave crest increase as function of the wave period.

Minimalization of overturning moment

The typical gravity platform designed for use in the North Sea, consists of a large concrete caisson and a number of columns piercing the surface, and a large deck for drilling and production equipment on top. The caisson shall provide stability with regard to foundation as well as the floating condition during building, towing and immersion.

To take the maximum benefit from the advantages in foundation system, such a structure should be designed so as to minimize the total overturning moment and the horizontal force. In that way the construction most likely have a large safety factor against bearing capacity failure, even in cases with poor foundation conditions.

Fines and Flogeland (1975) made a study of minimalization of overturning moment. As this study resulted in a relatively low caisson, the horizontal force is also low compared to conventional gravity platforms.
The idea behind their study can be illustrated by Fig. 9 where a typical gravity structure and an example of variation of force distribution with wave phase is shown. (Drag forces are neglected). The contribution to the overturning moment may be divided into two parts. It is

\[ M_{yx}: \text{moment due to horizontal forces on columns and caisson} \]
\[ M_{yz}: \text{moment due to distribution of force on the top of the caisson} \]

Using that \( M_{yx} \) and \( M_{yz} \) counteract and by systematically varying the main dimensions of the platform, it is then possible to minimize the overturning moment for a given incident regular wave.

Fines and Flogeland started first out with minimizing the overturning moment for a hundred year wave using a regular sinusoidal wave with wave height equal to the most probable largest wave height in hundred years.

![Figure 9. Example on force distribution at various phase angles.](image-url)
They used Walden's data (Walden, 1964) together with the long term statistical approach described by Nordenström (1971) to estimate the most probable largest wave. The wave period was selected according to DnV's rules (1974) which specify the following range for the design wave period:

\[ \sqrt{6}H < T < 20 \text{ sec} \]

where \( H \) is the wave height in meters. Normally the design period is in the range of 14 to 18 sec. However, Fines and Flogeland found that a variation of the design period from 14 to 19 seconds will not alter the computed value of the horizontal length of the "optimal" caisson with more than 1 to 2 per cent.

Fines and Flogeland used also the method of long term distribution of response (Nordenström, 1971) and an "optimal" platform was defined as the one having the smallest value of the most probable largest overturning moment in hundred years. The method of long term distribution of response assumes that the overturning moment in regular sinusoidal waves are linearly connected to the wave amplitude. This is satisfied since drag forces are neglected. The method of long term distribution of response is considered to be a more realistic approach than the design wave method.

Both approaches were used to obtain gravity platforms of minimum overturning moment for two different water depths, 80 and 160 meters. In both cases the two approaches gave the same results for the main dimensions of the substructure. It is thus believed that the design wave approach can be used to make a reasonable estimate of the main dimension instead of using the more complicated method of long term distribution of response.

The value of overturning moment obtained by using the method of long term distribution of response is considerable higher than the value obtained by using the design wave approach. The reason for this is as follows. The long term value of the overturning moment of the "optimal" platform will mainly be determined by the short waves with periods less than ten seconds. These short waves do not "touch" the caisson, and consequently there will be no moment due to pressure distribution on the top of the caisson to counteract the moment due to horizontal forces on the columns. The magnitude of the overturning moment in the range of periods considered when using the design wave approach is small since the moments due to horizontal and vertical forces will tend to cancel each other.

Fines and Flogeland found that for water depths up to 160-200 meters, gravity platforms can be well designed so as to minimize the overturning moment. These platforms are favourable in transferring wave induced loads to the ground. They do also provide a large foundation area, and good stability in floating condition. Problems may arise, however, in designing such platforms for deeper water, as the caisson will be lower and wider with increasing water depth.
REFERENCES


CELLULAR SHEET PILE ISLANDS IN ARCTIC WATERS

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Forssen Engineers
Anchorage, Alaska
Los Angeles, California
United States

ABSTRACT

Assuming acceptable soil conditions, large working areas can be constructed utilizing cellular sheet-pile construction techniques in water depths of up to 10 m. A combination of frozen and unfrozen soils and ice can resist varying raft and pack ice conditions, with required diameter a function of driving forces, foundation competence, available construction time and economics (induced freezing versus greater mass through earth fill). Greater depths of water may be accommodated under certain conditions.

The "cells-in-a-cell" concept provides a clean structural interface with sea ice, while adding redundancy to the system in the event of local failure. The small cells are self-bracing, once completed, and when filled, provide a working platform to supplement marine construction operations. In exposed locations, the outer zone of all cells, and the circumferential space between cells and outer bulkhead, would be artificially frozen to resist local buckling of sheet-piling and to enhance stability. Expansion forces due to subsequent freezing would thereby be directed inward and upward.

Materials import and construction logistics are acceptable in the short working seasons. Skirt containment systems, if dredging is permitted, would minimize adverse environmental impact. Heat injection into the freezing system would thaw the soil to enable demounting of the structure. Metallurgically acceptable sheet piling for cold conditions can be obtained.
INTRODUCTION

Several types of islands or structures have been proposed for construction in waters of the Arctic Ocean to serve as exploration and development facilities or as marine terminals. Steel and concrete structures, artificial or natural ice islands, and artificial earth islands have all been considered, or, in some cases, constructed. This paper examines the use of "off-the-shelf" components - sheet piling, earth and ice - and traditional construction techniques to build such facilities. The potential for reducing exploration risk and development cost is substantial where such construction is feasible.

At this writing, industry support for further analytic, model testing (especially ice mechanics on large-diameter cylinders) and possibly prototype construction are apparently forthcoming to confirm the conclusions in this paper, which are based on routine engineering calculations and judgement, and previously published related work.

BACKGROUND

Several years ago, in connection with oil exploration programs proposed for the Lower Cook Inlet of Alaska, U.S.A., the author examined the use of "sand islands", hydraulically filled cellular sheet pile structures as an alternative to structural steel towers. In waters less than thirteen meters deep at low tide, piling would be driven into the existing bottom. For greater depths, up to twenty-seven meters at high tide, a submerged rock fill would be constructed as a base for the cellular islands. No actual designs were completed in light of subsequent delays of leasing in that area, but preliminary analysis appeared to support the technical feasibility of the concept.

With the imminent leasing of shallow water areas of the Alaskan Beaufort Sea a possibility, and the general need for ice-safe marine facilities in many areas of Alaska, the sheet-pile cellular construction concept has been re-examined. In the meantime, similar construction has been accomplished in arctic Canada and Alaska. The Canadian facility utilizes cells driven off fast ice as part of a marginal pier, while the Alaskan project represented a sheet pile retaining bulkhead for fill placed on a natural island. The concept presented herein is a philosophy of total design for islands capable of resisting worst-case ice loadings within the five-fathom line; construction is feasible in open water or from fast ice.

Each site requires specific design, considering such variables as soil conditions, water depth, fill type and location, freezing requirements - thermal analysis, and worst-case ice conditions.

ASSUMPTIONS

For preliminary analysis, the following assumptions were made:

a.) The full projected area of the structure is loaded from just above the sea bottom to +3 meters uniformly at 400 p.s.i. No increases for rate of loading, or decreases for failure mechanisms were taken. 1)

b.) The exterior piling may be loaded locally to 1,000 p.s.i. by dense pressure ridge ice. 1)

c.) Ice loading may occur from any direction.
d.) Depth of water is ten meters

e.) Sheet pile section is 1/2 inch thick, strength equal to A 572.

**CONFIGURATION**

It is logical that the optimum streamlined configuration due to ice loading potential from any direction, is a plan-view circle. Although conical structures or ramp-sided islands will significantly reduce gross ice loadings under dynamic conditions, the following should also be considered in analyzing design loads for such facilities.

a.) Incipient movements of thick fast ice when adfreeze has occurred can produce loads equal to those on vertical-sided structures.

b.) Ramp-sided islands in water depths too deep for grounding of ice rafts will be subject to override by ice.

c.) Flared or conical structures require offsite prefabrication, difficult transportation, and pin-piling to resist overturning or translation.

The vertical sided cylindrical configuration proposed must resist full direct crushing strength levels of ice loading. Insofar as ice failure will be by crushing, bending and buckling, sufficient mass and dimension must be provided to resist very large overturning and sliding forces. This mass is most readily available in the form of local earth fill, or, in part, ice frozen in place.

Given these criteria, several types of construction may be feasible, including flat or box sheet piling, steel ring segments, or concrete ring segments. Exterior structures which lack flexibility must be able to structurally withstand full ice loading due to the significantly lower rigidity of frozen or unfrozen fill. Further, the latter, non-piling types of construction must be either barged to the proposed installation areas as sub-assemblies, assembled on shore and floated on site, or totally constructed in a yard or graving basin and towed on site; a total sequence of lead-time activities for custom design, fabrication, transportation and installation must be accomplished. The sheet-piling cellular approach, on the other hand, allows design for specific sites to occur simultaneously with transportation, since basic lengths and an approximate pile-count can be determined as soon as a site is identified. Piling and fill may be stockpiled a season in advance so that transportation risk would be limited to construction equipment. Standby charges for such equipment during full-scale development activities should not represent an unduly large percentage of total development facility costs. A further advantage of the sheet-pile system lies in its relative flexibility due to joint rotation and sheet bending, which should enable the fill to perform as the principal structural element. Fill volumes are reduced, and fill operations are fully contained to minimize adverse environmental impact.

Therefore, for this paper, only cylindrical islands utilizing sheet piling construction have been examined.

**EXPLORATION ISLANDS**

In discussions with various oil companies, it is obvious that the plan dimensions of an island will be a function of operations. In arctic locations where drilling mud, pipe and other bulk supplies cannot be reliably transported by supply vessel, over ice, or by helicopter year around, a much larger deck area must be available for storage than was provided on Cook Inlet platforms. However, to set the structural
MINIMUM SIZE - EXPLORATION

MINIMUM SIZE - DEVELOPMENT

ALTERNATE ISLAND CONFIGURATION
parameters for sizing of minimum island diameter, an analysis was made utilizing previously stated loading criteria, which set 60 meters in diameter as the minimum acceptable to resist overturning, sliding and internal shear failure. The fill must be frozen prior to maximum ice loading. Soil strengths required are two to four times those for competent sands, which is reasonable for frozen soils according to the Corps of Engineers, which cites typical strength increases of frozen over unfrozen soils of four to nine times. Plastic flow effects should not be severe due to non-uniform loading and short-term peak loading which can be expected. Ideally, fill material would be cohesionless, clean sands or gravels. Clean, uniformly graded sands will produce the greatest frozen strengths, with clays producing the poorest results.

The 60 meter diameter island would consist of a square arrangement of four cells, each approximately 23 meters in diameter, encircled by a smooth, round sheet pile bulkhead 60 meters in diameter as illustrated. Perimeter systems to induce freezing are contemplated, thereby producing sufficient strength in the annular fill to resist local pile buckling from pressure-ridge ice with crushing strength of up to 1,000 p.s.i., and eliminating the potential for hoop-tension failure due to internal expansion during freezeup; gross stability is enhanced during early raft ice loading. Additional heat pipe (thermal pile) systems with artificial chilling may be required to freeze the interior mass depending on thermal calculations and estimates of the timing of critical ice loading relative to natural freezeup. Base heave due to freezeup is expected to be accommodated due to the overall flexibility of the pile-soil structure. Once frozen, the entire system is expected to act as a unit through its frozen base to resist pack ice loading.

Procedurally, the island would be constructed in one continuous operation of 40-50 days duration. The self-bracing cells would be driven utilizing a bi-level template and filled by barged-in materials stockpiled on shore or dredged at a sufficiently remote area to avoid influence on base-scour of the structures. If necessary, dredging could be accomplished within a bottom-contact floating containment skirt which would confine suspended cuttings until settlement occurs. Dragline operations may be feasible, but clamshell and barge operations are assured. Hydraulic filling of exterior cells is only feasible where clean sands are available due to productivity and strength requirements. Upon completion of the first cell, it would be feasible to proceed with additional driving and filling operations utilizing both surface and floating equipment. The perimeter bulkhead would be tied back to completed cells. Fill would be placed around in-place thermal tubes. Thermo-couples would be installed to monitor freezeup; no personnel would occupy the island until freezeup occurred. Armor mats or rock would be installed at the base where scour is considered likely. Drilling operations and construction on the completed islands would be accomplished utilizing proven arctic techniques as for on-shore permafrost conditions. Inducing and maintaining the frozen condition requires different equipment depending on the type of material, ambient air temperature as well as that of surrounding sea water, and the time available to achieve freezeup. Natural temperatures vary with depth and time, depending on surface conditions and thermal properties of the soils. In the summer, soil temperatures in permafrost will increase from the annual mean (below freezing) at 7 meters to ten meters below the surface to above freezing at the surface. The freezeup diameter varies as the square root of time. Construction would require a work barge and supply barge, as well as a camp to serve a work force of fifty. A second crane for operations on completed cells would be required to complete basic construction in less than the maximum 80 days assured available. Time schedules and costs for the smaller island assume stockpiling of fill material, barging, and clamshell placement.

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Demounting would be facilitated by injecting heat into the thermal piles, which would be a prerequisite to removing sheet piling. Under these conditions, good re-use of the piling can be expected. Such fill as is thawed can be dredged within containment skirts if required.

As diameter is increased, strength requirements through freezing are decreased; a trade-off analysis will therefore determine whether cantilevered structure or additional filled dimension is preferable. Many soils in the Beaufort Sea are incompetent to support large loads in the unfrozen state, which would favor the sequential cell-freezing process as opposed to gravity structures for these locations. 4)

**DEVELOPMENT/PRODUCTION ISLANDS**

In sizing long-life islands for development, soils strength increases due to freezing were not considered, except in the peripheral cells and the annular space between the streamlined bulkhead and the perimeter cells. The frozen condition which will ultimately maintain in the interior fill will substantially increase the factors of safety against various failure modes. The minimum diameter required, therefore, in well consolidated sands, with peripheral cells frozen, is 150 meters, assuming previously stated loading criteria. Adequate strength can be developed in structures of a larger diameter by freezing ice in place in the central zone, which may be either a final structure (with adequate depth of fill as insulation), or to maintain the integrity of completed exterior cells if fill of the center cannot be completed prior to loading by sea ice.

The proposed design would utilize 23-meter diameter interlocking cells within the perimeter bulkhead on a radius which would provide a two-meter separation between the perimeter bulkhead and cells. The filled cells would constitute a containment bulkhead for the large internal fill, and would desirably be filled with well graded, granular, material, while the internal fill could be hydraulically placed material of lower quality, or ice covered with sufficient fill to prevent melting. Heave during freezeup would be less severe in coarse, porous materials. Substantial testing should be conducted on a full scale prototype to confirm design for such factors as creep, relaxation, constant strain rate, and direct static and dynamic ice loading. 5)

If ice is designed as a permanent component of the structure, insulation design should include consideration of the absorptivity of the surface, which, in natural conditions, can vary from 0.2 - 0.9. 6), as well as the thermal tube or circulating brine systems necessary to induce or maintain freezing.

Freezing of the zone between the external cells and streamlined bulkhead is mandatory to prevent local pile buckling under dense ice loading, and to prevent washing of fines at joints.

Although the preliminary calculations in support of this paper dealt with each element separately and utilized traditional soils and structural evaluation techniques, matrix methods may be valuable in examining the dynamic action of ice loading and the response of the non-homogeneous structure and foundation. In order to effectively accomplish such analysis, substantial physical or mathematical modeling of ice-failure mechanisms for various diameter islands would be desirable, and a means must be established to develop confidence in modeling ice-steel-soil boundary conditions.

Construction of the larger islands will require two seasons if totally earth-filled, even assuming prior stockpiling of fill material. The work effort would require two twelve-hour shifts utilizing six crews each, with six driving templates and cranes. As much work as possible would proceed from completed cells. Each location requires different considerations of construction techniques; fast ice can extend fifty miles
or more from shore and may allow winter construction; shore leads in summer may occur up to 50 miles wide; pack ice in motion will crush fast ice. 7) These and other factors will determine whether to complete the internal fill with soil or ice.

The long-life development/production island should be protected against corrosion. Experience indicates that coatings are not suitable in abrasive ice and that sacrificial metal is preferable. 8) Cathodic protection in ice-laden, fast flowing sea water is not wholly reliable either, but should be considered if insufficient steel thickness would remain at the end of the design life, after allowing for the relatively free-standing nature of the frozen soil mass. Sheet piling will not be subject to the cyclical loading and deformations found in Cook Inlet platforms, and normalizing is not feasible due to warping. Use of ASTM Grade A 537 steel is recommend

**MARINE TERMINALS**

In ice-prone areas such as Nome and Kotzebue, for drafts of thirteen meters or less, sheet-pile cells may be considered as the vertical elements in pier construction, with deck structures spanning between two or more cells. Alternatively, they may serve as dolphins or staging areas, with aerial cableways, conveyors, or pipelines transferring cargo to shore.

**ICE FAILURE MECHANISM**

It has been well documented that raft ice failure against vertical surfaces is initiated downward until ice-block buildup at the bottom creates ramping upward. Further, random block stacking indicates that freeboard of eight meters should be adequate to resist overtopping for even a planar vertical sided structure. 9) In the case of these round vertical sided structures, it is felt that the blocks forced downward will slough off laterally at the base forward of the tangent impact point, forming a trapezoidal mass with a spine sloped downward from the structure to a point under the ice raft. If this occurs, the raft would be failed in upward bending, and blocks would be diverted laterally, thus preventing raft ice from riding over the structure, and reducing gross loads from dense ice.

**COST**

Insofar as the designs presented above are conceptual in nature, and many unforeseen cost factors may emerge in research and design phases, cost estimates should be treated with great caution. Long experience in arctic construction also has taught us that the most carefully conceived construction programs can break down due to logistics or the perversity of nature.

The proposed designs have been separately reviewed on a preliminary basis by Wright-Schuchart-Harbor Construction Company and Swalling Construction Company, both experienced in arctic construction. Based on all required supply and construction costs, including camps, approximate budget figures for the two configurations discussed above would be U.S. $6 million and U.S. $20 million, respectively, for sheet-pile and earth fill operations only.
REFERENCES


INSTRUMENTATION FOR OFFSHORE STRUCTURES

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ABSTRACT

Actual experience with instrumentation on offshore structures is presented. Structures were instrumented to monitor such items as vessel motion; marine riser stresses; stresses and loads in critical structural members; wave, wind and current conditions; and operational parameters. Types of structures include platforms, semi-submersibles and ships, and geographic areas include the North Sea, Cook Inlet (Alaska), the Gulf of Mexico, the Santa Barbara Channel (California), and the Bass Strait (Australia).

Types of instruments are presented, as well as read-out methods, including techniques for automated recording and processing of data. Also described are methods for using instrumentation to obtain critical design information, and to monitor the safety of an offshore structure.
INTRODUCTION

At the present time many structures used in offshore exploration are instrumented to assist in the design and safety of these structures. The more recent structures have onboard computers which are used to collect data and control the operation of the rig. Many structural failures caused by the severe offshore environments have created a need for obtaining data to improve the design and try to eliminate these very costly accidents. For example, since 1955 there have been almost one hundred accidents, causing a loss of over 200 million dollars (Ref. 1). Half of these accidents were caused by the severe offshore environment. Thus, there is a great need to improve the design criteria for the offshore structures. There is also a need for hazard warning systems. Instrumentation can provide the type of information needed to minimize accidents.

The following can be a direct benefit from a well instrumented rig:

- Better rig design criteria
- Control during operation
- Fatigue information
- Improvement in safety
- Great cost savings

In this paper, various instrumentation techniques will be discussed in the many severe environments throughout the world.

TYPES OF STRUCTURES INSTRUMENTED

The mobile rig construction has increased significantly. As an example, 172 rigs were under construction in February 1975 (Ref. 2). One-third of these were semi-submersible. Many of these rigs will be instrumented. An example of a rig which had instrumentation is shown in Figure 1. This rig will be utilized in the North Sea.

![Semi-submersible in the North Sea](image)

Fixed platforms have also been instrumented in many different parts of the world. Figures 2 and 3 are examples of fixed platforms which were instrumented to measure the environment and structural response to the environment.
Many floating vessels have been instrumented. Figures 4 and 5 are examples of floaters which had an instrumented riser during exploratory drilling for load and angle measurements.

Instrumentation has been placed on many models tested in model basins to gain information prior to design and fabrication of the full scale unit. Figure 6 shows a scale model built and instrumented for tests at a model basin test facility.
Many laboratory and shipyard tests have used instrumentation in order to optimize a design. Figure 7 shows a test conducted in the shipyard. Later, the same unit was tested and instrumented during an offshore test.

As previously stated, the severe offshore environment causes approximately 50% of the rig failures. Thus, it becomes imperative to measure these environments.

An example of a typical structural load caused by an environment which has caused many failures is the ice loading on a structure. Ice floating at Cook Inlet tends to pile up against the structure. This type of loading (Fig. 8) is quite severe, and measurements were taken to establish its force.
Rough seas in the North Sea, Gulf of Mexico and Bass Straits are the types of environments (Fig. 9) which have caused structural damage and often failure. These forces acting on the structure have been and must be measured.

Seismic loading on a structure is another important parameter which is measured. This loading is one of the criteria used for design of structures off the coast of California and in the Gulf of Alaska.

**TYPES OF MEASUREMENTS**

The type of measurements that have been performed and that are required are discussed in this section. In general, these can be grouped in the following manner:

- Forcing Functions
- Structural Response
- Vessel Conditions
Forcing Function

The forcing function on a structure is the environment which causes the structure to fail. Measurements must be made in order to establish a relation between the input forces and the structural response. The forces acting on a structure are wave and current loads, wind loads, seismic loads, and other external forces (i.e., ice, axial load).

The forces created by wave loads are difficult to measure. A wave force transducer* has been designed and fabricated only for the measurement of wave forces. This transducer measures the magnitude and direction of total wave force on a selected shape object immersed in the path of a wave.

The design incorporates several active element sections. The entire force against the active elements are resisted by hydraulic load cells. The main structure of the wave transducer is a central 60 cm O.D. pipe to which the active and inactive (dummy) elements are attached (Fig. 10).

The sea state measurements are made by off-the-shelf instruments. To measure wave height, a wave staff or a pressure transducer is utilized. The wave staff (i.e., Baylor Company, a wave profile recorder), consists of two stainless steel wire ropes (Fig. 11, 25 m in length, 1.2 cm in diameter), spaced about 23 cm apart, which are secured to a platform below the water. The transducer element electrically senses the length of wire rope above the water. Thus, the wave height is sensed as a function of time.

Sea state measurements are often made near the structure (Ref. 4) to better define the sea environment. A waverider buoy (i.e., Datawell Waverider Series 600) is a reliable instrument which has been used in many countries. The basic system consists of a spherical stainless steel hull 0.7 meters in diameter and a receiver, which are tied together by a radio link. In the hull is mounted a vertically stabilized accelerometer, integrating circuits and battery pack, and a radio transmitter in the 27 megahertz range which radiates a signal representing sea surface elevation.

A data processor and recording system have also been placed inside the hull of each buoy. A cutaway view of the buoy is shown in Figure 12. The system can analyze the wave record for several minutes of each hour. The analysis results in a set of numbers which gives the average height and average period of the waves. At the end of the sample period, the numbers which describe the waves are written on a digital magnetic cassette data logger. Wave data are received at shore stations with a receiver and signal processor.
A significant factor in the survival of the buoys is the mooring design. The wave-profile following the buoy must remain afloat while riding over the maximum wave during the extreme current that may be occurring. Strong currents expected in a severe storm limit the mooring design alternatives.

Another important parameter is the measurement of water velocity accomplished with the use of a water current meter (i.e., Engineering Physics Company). The meter measures water particle velocities, both transient and steady state. It provides two voltage outputs proportional to the water flow orthogonal vectors by use of an electromagnetic device which senses the component of water flow perpendicular to the sensor.

Wind velocity and direction are measured by an aerovane (i.e., Bendix Corporation). The aerovane system consists of a transmitter, a support pipe, an indicator, and connecting cables. The transmitter is a three-bladed rotor driven D.C. generator which produces a voltage proportional to wind speed. It also has a streamlined vane which holds the rotor into the wind and reacts with each change in direction. A small synchro motor transmits the vane position to the recording system.

Other environmental parameters which are measured are barometric pressure and temperature.

Loads caused by seismic inputs are required to identify input forcing functions. Seismic transducers are installed offshore in hard rock or in soft clay. A typical system for seismic measurement (i.e., Kinematics, Strong Motion Accelerograph) consists of triaxial electromagnetic accelerometers which record the ground motion acceleration response at a specific location. A difficult problem in seismic measurement is the installation of the transducer system (i.e., under the water in hard rock).

Structural Response

Structural stress is the most important response measurement. Axial stresses (both tension and compression), bending stresses, and torsional stresses are the desired parameters. Strain gages are used to obtain the stress in structural members, member joints, riser conductors, and piles. As most of these members are under water, a waterproof strain measurement system is mandatory. The waterproofing of systems for a
long period of time in an underwater environment is a difficult but not unsurmountable task. The test results have been obtained by the use of an integral lead weldable strain gage. A strain gage configuration is shown in Figure 13. Shown is a half-bridge strain gage that has one active element, which yields strain output, and one dummy element, which cancels the undesirable environmental changes. The gages are used for dynamic and static measurements to ±6000 microinches per inch. They are attached to the structure by spot welding. The wires are encased in a stainless steel tube and are transmitted to a water tight manifold. A cable is secured to the manifold and is used for transmission of the signal to the recording system at the top of the rig.

Figure 13. Strain Gage Configuration Used for Underwater Stress Measurements

Typical strain gage installations are shown in Figures 14 through 16. Many offshore structures fail in the joints, stresses at or near a joint are often measured (Fig. 14).

Figure 14. Typical Strain Gage Installation at a Structural Joint on a Fixed Platform
Axial or bending stresses of structural members or risers are commonly measured. Figure 15 shows strain gages on a riser. Often strain measurements are desired to obtain the bending moments in a pile. Gages have been installed in piles as deep as 16 meters below the mud line. Figure 16 shows the gage installation inside the pile.

![Figure 15. Strain Gage Installation on a Riser Used in the Bass Straits](image)

![Figure 16. Strain Gage Installation Inside a Pile Below the Mud Line](image)

Fatigue

Strain gages are used primarily to measure structural stresses (strains); however, they also can be used to assist in fatigue monitoring. Questions such as, "How long can the rig be offshore prior to an inspection?" can be partially answered with an instrumentation system. Strain measurements can be related to fatigue.

The output of the strain gages is transmitted to a Threshold Sensing Unit* which monitors the strain levels. The threshold sensing system counts the number of times a strain gage input signal exceeds a given level and/or measures cycle peak to peak.

* Patent pending, Mechanics Research, Inc.
amplitudes. A switch is provided for selecting Mode 1, the threshold sensing mode; Mode 2, the cycle amplitude measuring mode; or both Modes 1+2. A printed output summarizing the collected data corresponding to the selected operating mode is provided at predetermined time intervals.

There is a specified number of strain gage input channels and the detection range of each is divided into eight adjustable levels. For the level exceedance counting mode, Mode 1, a count cycle for each input channel consists of first sensing a level crossing and then counting the event when the signal recrosses a predetermined lower level in the case of positive signals or a higher level for negative signals. Utilizing a different level instead of the sensing level introduces hysteresis, and therefore, noise immunity. Each time a level sense and count cycle is completed, a counter corresponding to the level and channel will be incremented by one.

A similar detection scheme is used for measurement of cycle amplitudes, Mode 2. A cycle consists of two consecutive reversals in the input signal, and the amplitude is measured by the number of levels that the cycle spans, rounded off to the nearest integer. A set of eight amplitude levels is stored and utilized for each channel for classifying the cycle amplitudes.

Both the counting of level exceedances and the measuring of cycle amplitudes are accomplished with a fully contained micro-processing unit. The processor is permanently programmed, and controls the data input and output, and stores count data as well. The data output consists of a printout of the date and time, followed by a listing of the count data corresponding to the selected operating mode (level crossing, cycle amplitude, or both). A print sequence is automatically initiated nominally every twelve hours, (or any other predetermined interval), at which time the temporary storage counter is reset to zero for the next count cycle. A single visual display is also provided for observing the contents of the counter. Any of the pre-set detection levels, as well as the time of day, can be selected and displayed. In operation, the threshold sensing system is completely automatic. The system monitors the analog inputs and provides automatic printouts at predetermined intervals.

Other Measurements - Vessel Conditions

In order to properly analyze data (forcing function) and the structural response data, other parameters need to be measured. Desired functions are vessel heave, pitch and roll or surge and sway. Also, the vessel position relative to the hole is a desired value.

The measurement of vessel heave in the severe ocean environment is accomplished by use of a direct linear measurement device. The installation and the measurement technique of a heave transducer* is shown in Figure 17.

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* Patent pending, Mechanics Research, Inc.
The heave transducer is an electro-mechanical device which measures the stroke of the riser tensioner hydraulic cylinder piston or the actual tensioner wire rope travel. The transducer housing is mounted to the stationary part of the cylinder, and a stainless steel pull cable is attached to the piston assembly. The pull cable is tensioned by a counter weight system. The mechanical motion is converted to an electrical signal by a zero backlash ball screw and a linear transducer.

Vessel heave is also measured by the use of an accelerometer. An accelerometer is placed as close as possible to the center of gravity of the structure. The output signal from the accelerometer is double integrated to obtain displacement or heave of the vessel. This measurement also provides the structure natural frequency.

Vessel pitch, roll, surge and sway are measured by inertial reference systems (i.e., Humphrey). The transducers consist of small precision vertical gyroscopes with sensitive servo-type accelerometers mounted on their inner gimbal. These units provide outputs for pitch and roll angles in reference to vertical, and the vertical acceleration vector. They can also provide a vertical stabilized two-axis system with pitch and roll angle outputs and horizontally stabilized vertical and lateral acceleration outputs. Thus, surge and sway measurements can be obtained from this type of a unit. These stabilized systems are mounted on the deck of the vessel as close as possible to the center of gravity.

MEASUREMENT DATA ACQUISITION SYSTEMS

The measurements described in the previous sections are recorded on the rig. The recording systems fall into two categories: analogue and digital data acquisition systems. Prior to recording the data, all the measurements are transmitted to signal conditioning equipment. The transmission of the data is accomplished by multi-conductor instrumentation cable. Figure 18 shows the cable as it is attached to the riser. Fixed platform cable installation utilizes a conduit for cable protection.
Transducers installed on a riser
Instrumentation cable

Figure 18. Cable Installation on a Marine Riser

Data Conditioning Equipment

The data conditioning equipment is comprised of components necessary to supply power to the measurement transducer, provide balancing control, amplify the signals, filter the signals, and provide a signal calibration. The conditioning equipment is designed for specific measurements: the strain gages will have one type of system, and the wave height transducer will have a different system.

Analogue Data Acquisition System

The signal outputs from the data conditioning equipment are transmitted to the recording system. Analogue recorders utilized for collection of the data consist of magnetic tape recorders, oscillographs, and strip chart recorders. Magnetic tape recorders (i.e., Geotech) are utilized when long continuous data is desired. This recorder can be unattended for long periods of time. Other tape recorders (i.e., Ampex, Honeywell) and oscillographs are used when a system is attended and when on-site data is desired.

Digital Data Acquisition System

Digital acquisition systems consist of data loggers and computer based systems. A number of manufacturers provide digital data logging systems for field use. A majority of the data logging systems record the data either in printed form or on tape cartridges or cassettes. Since no industry standards exist with regard to recording formats for cartridge or cassette recorders, the use of such devices requires that a playback system be available from the manufacturer of the recorder. The majority of these units consist of a hardwired, combination multiplexer-analogue-to-digital converter, a controller-sequencer which responds to commands from switches and/or thumbwheels, and a write-only digital tape recorder.

The computer based digital data acquisition system provides the ability to both write and read magnetic tape data. This capability is most important in the field to insure
reliability of the desired measurements. The computer based system includes software to both acquire data flexibly and to retrieve it, either graphically or numerically, in the field. Because it is computer controlled, the software can be modified easily to add specific features which may be required for any given task. A typical system used is shown in Figure 19.

![Figure 19. Digital Data Acquisition System Installed on a Semi-Submersible in the North Sea](image)

This system consists of:

- A Varian 620/1 minicomputer.
- An analogue-to-digital conversion system.
- A digital-to-analogue converter to provide playback.
- An ASR 33 teletype for input/output.
- An IBM-compatible magnetic tape drive for recording the acquired data.
- A time code generator to provide time and day information to be recorded with the acquired data.

Data may be retrieved from magnetic tape whenever the system is not acquiring information. Time histories are provided either in analogue form via the digital-to-analogue converter or in numeric form on the teletype printer. In addition, engineering units and a statistical summary of all acquired channels covering a specific time period can be generated.

Data Processing

As previously stated, a limited amount of data processing is done offshore. The final data reduction is performed in a data processing laboratory. Digital tapes can be reduced readily by the use of a computer system identical to the one used for data acquisition. Software is available to reduce the data to obtain time histories, statistical properties, power spectral densities, transfer functions, coherence functions, and many other parameters. Example of processed data is shown in Figure 20: power spectral density of the wave motion.

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852) Geminder/Pomonik
Data Analysis
PSD of Wave Motion
(Units are \( \text{Ft}^2/\text{Hz} \))
RMS = 12.7 Ft
Bandwidth = .0008 Hz
Time Period 11-6-74/0300-0700

Figure 20. Offshore Data Analyzed in a Signal Processing Laboratory

USING THE DATA - SYSTEM DESIGN, AND OPERATIONAL SAFETY AND MAINTENANCE

The steps followed for assuring the structural and functional adequacy of the offshore system are described in this section. The contributions of instrumentation data to this design development are presented. Finally, a review is presented showing the application of monitoring data to system efficiency, safety and maintenance.

Figure 21 illustrates a typical approach for developing an offshore structural system.

Figure 21. System Flow Diagram for the Development of an Offshore Structural System
The steps followed in the development of the system are:

1. Define the function and life cycle of the structure (develop the system requirements).
2. Define the environments in which the system will operate.
3. Combine the functional requirements with the environmental levels, in order to generate a detailed design criteria.
4. Develop the preliminary design.
5. Develop analytical models in order to study responses of the system to operations and to the environments, to refine the design criteria, and to aid in the development of the final design. Analytical models can include:
   - Static structural model (for studying the effects of wind, current, pressure, accelerations, etc.)
   - Dynamic structural model (for studying the effects of waves, shock, vibration, acoustics, etc.)
   - Thermal model (for studying the effects of temperature, solar radiation and operational heating)
   - Functional models.
6. Apply the detailed environments to the analytical models in order to determine theoretical responses, critical conditions, and critical system components.
7. If necessary, modify the design based on the analytical results.
8. Use the design criteria and the results of the analysis to develop meaningful tests.
9. Conduct selected tests (under controlled conditions) on critical subsystems. Initial tests may be performed on prototype units and on system or subsystem models.
10. Develop a prototype or experimental system and study its behavior and functionality in the offshore environment. Extensive use of instrumentation and monitoring systems would be used during this phase of development.
11. When the operational system is deployed, monitor the significant environments and the critical areas of the system.
12. Use the results of the analysis, laboratory tests, ocean tests, and operational experience to:
   a. Verify design levels and upgrade future design,
   b. Establish limits for operating conditions,
   c. Establish and verify reliability and maintenance requirements,
   d. Troubleshoot operational problems.

Some of the applications of actual data to the design development have been discussed before and are as follows:
Environments

Measurements of the environments (wave height, wind velocity, current velocities, etc.) in the operational areas can provide valuable inputs to the development of design and operational criteria. Statistical profiles of the operational areas can be developed and improved using actual data. For example, wave data can be used to generate a profile of probable significant wave height as a function of time of year. The frequency content of the spectra can be developed from the data in order to better define its effect on structural response. In addition, weather data can be used to verify the probable combinations of environmental extremes.

Design Verification

Instrumentation is necessary in order to verify the design analysis and to assure the adequacy of the system. For example, instrumentation data is used to verify analytical transfer functions (ratios of responses to inputs), such as structural response (motions or loads) to wave inputs. Theoretical motions and loads induced by waves are sensitive to the analytical model used, and are particularly sensitive to the hydrodynamic coefficients used in the model. For complex structures it is important to verify and refine these coefficients (and the analytical results obtained from using them) by using data from actual operating conditions. Results are often dependent on unique characteristics of the structure and on the effects of combined environmental and operational inputs.

Design Improvement

The existence of a meaningful instrumentation package allows improvement of the system based on actual data, instead of on estimates and suppositions. Improvements can include such items as increased functional life, extension of operations to more severe environments, increased safety, and cost reductions for later designs.

Operational Efficiency, Safety, and Maintenance

During operations, a meaningful system for monitoring the environments and the structure is an important management tool. Safe and efficient operations are dependent on knowing what the input conditions are, what the associated loads and motions are, and how these compare with the acceptable limits for a given operating mode. During severe environmental inputs, it is very difficult to establish system conditions based on personal observations. Instrumentation provides the offshore manager with quantified data, which lets him decide to proceed, to stop, or to modify his operations. With an at-sea data analysis system (i.e., the MRI Offshore Digital Data Acquisition System), the decision-making process can be simplified and improved. Life cycle monitoring (such as fatigue load data, or wire rope cycling) can identify periods when preventive maintenance is necessary. Quantified data can also be used to check for a change in operating conditions which has actually reduced an adverse structural condition. For example, instrumentation data can be used to check whether or not a change in vessel position has reduced stresses in a marine riser.

CONCLUSIONS

As discussed in previous sections, instrumenting offshore structures is being done successfully. Data is being collected from all types of offshore rigs in all types of environments to assist in the design development of more reliable and safer structures. Instrumentation is used to verify analytical tools and assumptions in this design process. Additional data is needed from experiments or from operational offshore rigs to provide verification and confidence in the design process.
REFERENCES


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EXPERIENCES OF ICE FORCES AGAINST A STEEL LIGHTHOUSE MOUNTED ON THE SEABED, AND PROPOSED CONSTRUCTIONAL REFINEMENTS

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ABSTRACT

During the summer of 1973 a new tubular steel cylinder type lighthouse was constructed at the northernmost end of the Gulf of Bothnia. As soon as ice formation and drifting started with the winter 73/74 ice crushing against this new lighthouse induced severe vibrations which destroyed the lantern equipment and later also caused structural failures and finally collapse.

Vibration measurements and theoretical calculations showed that the frequency of the ice load tends towards the first or second natural frequency of the lighthouse structure. To find out the resonant condition a simple approximative formula was constructed to relate the stiffness of the structure to the ice properties and drifting speed. This verified that an ice drifting speed does exist that yields to resonance. The fatigue failures on overwater structures have been due to resonant vibrations, but the final collapse load on the foundation pile has been static in nature and evidently due to a pressure ridge.

Now refined design criteria have been established for future purposes to take into account severe pressure ridge loads for underwater structures. Instrumentation has been designed to measure the pressure distributions on a new concrete caisson type lighthouse. Steel lighthouses will have overwater structures isolated from the vibrating underwater structures by springs and shock absorbers and dimensioned against dynamic loadings. A 1:2 scale model has been constructed and tested. The results verified the expected behaviour of the vibration isolation system. This construction ensures the cheapness of the steel structure and eliminates inconvenient vibrations during ice crushing. Now design work is in progress for a fullscale vibration isolated test lighthouse.
INTRODUCTION

As soon as severe vibrations were discovered in December 73 at the Kemi I lighthouse, the University of Oulu was requested by the Finnish Board of Navigation to study the ice forces operating against this new lighthouse and to propose better design criteria.

The Kemi I lighthouse was situated at the northernmost end of the Gulf of Bothnia. The structure was a tubular steel pile hammered deep into the sand of the sea bottom. The underwater structure was conical, with the neck narrowing to the waterline to minimize ice forces. The above-water structure was also a tubular steel cylinder, widened to accommodate storerooms, lantern equipment, etc., app. 1a.

Compared with concrete caisson type lighthouses, the steel lighthouse was a light, flexible structure. The basic idea of this low-weight steel lighthouse was a cheap and simple structure with a minimal amount of construction work at sea. However, it appeared to be very sensitive to ice-induced vibrations. Even the crushing of a relatively thin ice sheet caused severe vibrations. The measurements and observations of vibrations are briefly described in this report and means of designing safer structures and constructing a vibration isolation system for upper structures and lantern equipment are proposed.

VIBRATIONAL BEHAVIOUR AND ICE FORCES

During measurements three types of vibrations were recorded: pure vibrations with the first and second natural mode of the structure, and a combination of these two. In the first type the frequency was 0.85 Hz, with amplitudes of 20 cm at the top of the lighthouse and 5 cm at the waterline; accelerations at the top were 0.58 g, ice thickness was 55 cm.

With the second mode of vibration the frequency was 3.65 Hz, at the waterline an acceleration level mean of 0.3 g with max. 1.2 g and displacement amplitudes of 1.3 cm. The acceleration values were mean 0.7 g with max. 1.8 g at a point 8.8 meters above sea level. This second type could continue unchanged for several minutes. Acceleration levels were high although ice thickness was only 10 cm. Maximum ice thicknesses of 100 cm would have occurred later in the winter but luckily they did not drift anywhere.

The third type of vibration was a combination of the first and second natural modes. It was more common during observation time than the pure first mode on thick ice. Usually when the ice thickness grew the resonant second mode disappeared and the structure started to vibrate with the first mode and increasing amplitudes. Very soon, however, the vibration changed to a combination of the first and second modes and this might continue as long as thick ice or a pressure ridge crushed against the lighthouse. Evidently the ice crushing frequency corresponding to the structure stiffness; ice strength and drifting speed did not match the natural frequencies of the structure.

At the beginning of the combined cycle, when the ice failed and crushed, the sudden release of the load caused the structure to spring out of its deflection with all its natural modes. Of these the first two are most
dominant, and as the second was four times faster, crushing would start with the second mode. During the return phase of the second mode the foundation pile came loose from the ice edge and later hit against it. At this stage all the energy of the second mode was dissipated and the crushing continued next only with the first mode for the remaining half cycle. A typical double-blow crushing noise was heard during this type of vibration. At the beginning of this process, with 10 cm ice thickness, acceleration levels of up to 3.6 g and displacement amplitudes of up to 3.0 cm were recorded at a point 8.8 meters above sea level.

Later in the spring, when the foundation pile was free of the dynamic effects of upper structures, a fourth type of vibration was observed. The displacement $\delta$ of the foundation pile grew almost linearly with the drifting speed of ice cover. At maximum deflection the ice strength was exceeded and crushing began and displacement sprang back to zero in a very short time. After crushing the foundation pile again stuck to the edge of the ice sheet, starting displacement growth and a new cycle without any additional vibrations, (Fig. 1).

![Figure 1. The displacement of the foundation pile vice time.](image)

In 93 cm thick ice the displacement average, peak to peak, varied around 10 cm while the maximum values reached up to 20 cm. The random character of ice strength was evident. A 20 cm deflection means a pulsating ice force of 3.5 MN or a mean ice-crushing strength of 350 N/cm² over the whole projectional area.

The vibrational behaviour of the ice sheet and the lighthouse structure is best described as "stick-slip" movement. The stick-period occurs when the pressure of ice against the structure is lower than the ice-crushing strength. Once this is exceeded the slip-period will begin. Observations revealed that particularly when the dynamic effects of the upper structure were absent vibrations were pure stick-slip vibrations. The measurements gave a relative damping coefficient of 0.10 in the first mode vibration. In the combinational vibration of the first and second modes aperiodic damping occurred for the second mode. Aperiodic damping was also observed in the fourth type of vibration. The energy dissipation during ice crushing plays a major role in the damping; the structural damping or hydrodynamic damping effects are of minor importance.
The final collapse load which brought down the foundation pile at the beginning of May was calculated as being at least 4.4 MN. This implies a mean ice crushing strength of 430 N/cm² in the case of a recorded maximum ice thickness of 93 cm earlier in the winter. The ice-crushing strength is considerably greater than reported elsewhere (Blenkarn, 1970; Afanas'yev, et al., 1972; Dinkla et al., 1970). The difference may be due to a small pile diameter/ice thickness ratio of 1.2, or more probably a collapse load caused by a pressure ridge.

**RESONANCE CONDITION**

The measurements and observations of vibrations revealed that the "stick-crush" type of relative movement between the ice sheet and the structure most probably occurred at the first or second natural frequency of the structure. From the structural point of view the resonant loading is the most critical one.

Considering the possibility of resonance for the ice force a simple approximative formula may be easily constructed to relate the following parameters: \(\sigma_c\) = ice crushing strength, \(h\) = ice thickness, \(v\) = ice drifting speed, \(d\) = pile diameter, \(k\) = structural stiffness coefficient for the ice load and \(f_c\) = crushing frequency.

The shape of the ice force function is similar to that of deflection shape, (Fig. 1). During low-frequency vibrations the dynamic inertia forces for the maximum deflection \(\delta_c\) of the foundation pile are negligible compared to the ice force, usually less than 10% as stated by computer runs. So the maximum ice crushing force \(F_c\) can be approximately given by the spring equation

\[
F_c = k \cdot \delta_c
\]

On the other hand

\[
F_c = \sigma_c h d
\]

The mean crushing length per cycle is almost the same as \(\delta_c\) - the negative deflection is small - and so an equation may be written for the ice drifting speed

\[
v = f_c \delta_c
\]

Eliminating the unknown \(\delta_c\) and \(F_c\) from equations 1-3 yields the resonant condition

\[
f_c = \frac{kv}{\sigma_c h d}
\]

Evidently it is always possible to have such an ice drifting speed that resonance is achieved. This is true regardless of the simplifications in the above equations. Also the measurements verify equation 4. The data \(v = 200-300\) m/h, \(h = 10\) or 55 cm, \(\sigma_c = 350\) N/cm (\(T = -16^\circ C\)), \(k = 256\) kN/cm, \(d = 110\) cm gave

\[
f_c = 0.67...1.01\ \text{Hz} \ (h = 55\ \text{cm})
\]

\[
f_c = 3.70...5.54\ \text{Hz} \ (h = 10\ \text{cm})
\]
These values coincide very well with the first two natural frequencies 0.85 and 3.85 of the Kemi I lighthouse.

During the combined vibration at the first and second mode the ice thickness did not match according to equation 4 with the natural frequencies of the structure. The properties of ice are statistical by nature and so the ice force and crushing frequency will also be random variables causing a random response for the structure. In any case the structure seems most likely to vibrate at its natural modes and frequencies.

Equation 4 may be refined to include the next higher natural frequency but more stringently the properties of structure and ice can be interrelated using the theory of self oscillations. The dynamic equations of equilibrium will have a loading term dependent on the ice crushing strength rate $\delta$, which has a negative slope when it is great enough. This gives rise to self oscillations. In any case the most important parameters are those in equation 4. In addition there will be the mass and damping effects of the structure and ice. The $\delta$ will also be dependent on ice drifting speed $v$ and pile diameter $d$ with the usual ice properties.

**COMPUTER ANALYSIS**

A beam FEM model was constructed to simulate the vibrational behaviour of the steel lighthouse under pulsating ice loads. The direct stiffness method was used, the structure was discretized into 12 elements with in all 26 degrees of freedom. The distributed masses were observed in a consistent manner. The actual unlinear damping was linearized using the concept of equivalent linear viscous damping. At first the damping matrix was constructed to be proportional to mass and stiffness matrices, but later direct principal mode relative damping coefficients were used. Loading functions were similar in shape to that of displacement function, (Fig. 1).

The dynamic equations of equilibrium were integrated using the principal mode presentation and a fourth-order Runge-Kutta algorithm.

The computer simulation verified the sensitiveness to vibrations of the low-weight steel lighthouse. Ice forces due to a relatively thin ice sheet would already cause too severe vibrations to the above-water structures to permit occupation or even conventional types of lantern equipment. The computer runs showed that it is not possible to construct a steel cylinder-type lighthouse that is both cheap and yet rigid enough to withstand ice-load induced vibrations without an efficient vibration isolation system for the above-water structures.

**NEW DESIGN CRITERIA**

With large pile diameter ratios the measured mean ice crushing strength values have varied from 85 to 170 N/cm² (Blenkarn, 1970; Afanas' ey et al., 1972; Dinkla et al., 1970). When calculating the ice force with equation 2

$$ F = \sigma_c h d $$

the ice crushing strength has to be adjusted according to to the pile diameter to thickness ratio. Several formulas have been presented by
researchers, a simple formula (Tryde, 1974) has been adopted

\[ \sigma_c = \sigma \cdot (1 + \frac{2h}{0.4 + d}) \]  

(5)

Here \( \sigma \) corresponds to an infinite pile diameter to ice thickness ratio.

Although the effective ice crushing strength according to equation 5 increases rapidly when the pile diameter to ice thickness ratio falls below two it is, however, advantageous to have a small diameter at the waterline in order to minimize ice forces.

The maximum ice load will be due to a maximum pressure ridge. It has been calculated (Dinkla et al., 1970, Peyton, 1968) that the resultant pressure ridge load will be 2 to 3.6 times the maximum fast ice sheet load, but no information about vertical load distribution is supplied. With such slender structures as a bottom founded tubular steel lighthouse it is also important to know the pressure distribution.

The pressure ridge model of reference (Dinkla et al., 1970) has been also measured in practice and predicted by computer simulations by Parmerter and Coon. According to vertical ice distribution in this model a vertical load distribution in a pressure ridge is adopted, app. 2. Maximum pressure is at the area of fast ice cover that is lifted about its thickness by underlying ice bricks. On the area of ice bricks nearest the fast ice sheet the pressure is assumed to be half of the maximum, reducing linearly to zero when going to the top of the sail or bottom of the ridge regardless of the slender cone shape. Total pressure ridge load will then be four times the fast ice sheet load.

A special instrumentation has been designed for installation in a new concrete caisson type lighthouse to measure pressure ridge load distributions. It consists of 30 pressure sensitive cells situated in five levels from -4m under-water to +1m above-water in the half circle of the lighthouse structure. It is planned to begin measurements during winter 75/76.

The signals from the pressure cells are measured by a portable battery operated data-logger. This is capable to scan up to 64 channels with a rate of 1000 channels per second. So the maximum time difference between two separate load cell measurements will be less than 17 ms. The digital results are recorded in serial form to a data cassette and later on analyzed by a computer.

The dimensioning of the underwater structures may be carried out using the Smith-diagram for a loading with both static and dynamic components. It is not likely that the whole pressure ridge load would behave dynamically according to (Fig 1). The probability of ice force reducing to zero at once is small since crushing would not happen simultaneously on the fast ice sheet and on separate ice bricks against the foundation pile.

The dimensioning of the above water structures will be undertaken with an eye on dynamic ice forces. At least half of the pressure ridge load has to be considered and the most critical loading, the one resonating

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with the lowest natural frequencies of the structure, has to be used.

VIBRATION ISOLATION SYSTEM

In order to obtain vibration levels low enough in above-water structures for lighthouse purposes it is more economical to arrange a vibration isolation system rather than to stiffen underwater structures. Dimensioning only against ice forces yields a too flexible structure with the great acceleration values at the top of the lighthouse.

The new steel lighthouse design, app. 1b, consists of a tubular underwater structure similar to the earlier Kemi I. The connection between the foundation pile and above-water structures is not, however, any more rigid. The widened above-water lighthouse structures are supported on the foundation pile by four elastic beams.

During the rapid deflection spring out of the foundation pile during ice crushing the light horizontal spring stiffness of the beams cannot transfer high loads and acceleration values to the upper structures. The four special beams, or three as a minimum, provide the action of a parallelogram regardless of the direction of horizontal deflections and so the upper structures will maintain their upright position although the foundation pile is vibrating in different phase and amplitudes in relation to it.

The supporting elastic beams are designed to give internal damping as well, but there is an additional double shock absorbing system with shock absorbers and stoppers. Relative movement between the upper and lower structures is restricted by a stopper ring which operates at the same time as an overload restricting device. The cut of relative movement is made watertight by an elastic toroidal membrane.

The design criteria for the vibration isolation system are the amount of permitted relative movement and the amount of energy dissipation capability in shock absorbers during dynamic resonant ice loading. Calculations and computer simulations have shown that acceleration values on over-water structures may be easily reduced to one tenth or to one thirteenth of that without a vibration isolation system.

A typical deflection history in a vibration isolated lighthouse during ice crushing is shown in appendix 3. Appendix 4 gives the results of computer simulation of a proposed steel lighthouse with vibration isolation. The deflection δ at the top and δ at the waterline and corresponding accelerations are plotted during one ice load cycle in resonant state.

The effectiveness of the vibration isolation system has been confirmed by scale model tests. A 1:2 scale model, app. 5, was constructed. The horizontal movement of the foundation pile top was created by the load cylinder of a programmable hydro-pulsator. The underpart of the vibration isolation system was on rolls, with rails inclined to simulate the foundation pile slope of deflection during deflection. The upper structures were simulated by an equivalent mass and second moment of inertia calculated with reference to the top of the vibration isolation part.
As the results of computer simulation and scale model tests were encoura-
ging for the vibration isolated lighthouse and as it would be economically a good choice, design work is in process for a full-scale vibration isolated lighthouse. The selected construction site will be in the southern part of the Gulf of Bothnia in a water depth of 11 meters and with maximum ice thickness of 70 cm. It is planned to complete the structure before winter 76/77.

REFERENCES
1 Blenkarn K, 1970: Measurements and analysis of ice forces on Cook inlet structures, Offshore technology conference, Dallas, Texas.
2 Afanas'yev V, Dologopolov I and Shvayshteyn, April 1972: Ice pressure on separate supporting structures in the sea. CRREL, Draft translation.

APPENDIX
1 The Kemi I lighthouse and a lighthouse with an vibration isolation system.
2 Pressure ridge load distribution.
3 The vibration isolation system operation during ice crushing.
4 Results of computer simulation of deflections and accelerations during resonant vibrations.
5 A scale-model of a vibration isolation system.
a) Kemi I lighthouse

b) A lighthouse with vibration isolation system
Pressure ridge load distribution

\[ M(x) = q_0 h^2 \left( \frac{4}{3} + \frac{x}{h} + \frac{x^2}{2h^2} + \frac{x^3}{12h^3} \right) \]

\[ 0 \leq \frac{x}{h} \leq 10 \]

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A lighthouse with vibration isolation system
A lighthouse with vibration isolation system
APPENDIX 5

SCALE MODEL 1:2
A lighthouse with vibration isolation system
ESTIMATING PILE ICING UNDER NORTHERN CLIMATES AND TIDAL CONDITIONS

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EXTENDED ABSTRACT

Ice accumulation may occur on piles and bracing members of marine structures where these structures are located in geographic regions which experience cold climates and large tidal variations. The ice accumulation increases the downward vertical load on the pile or member at low tide and results in buoyancy effects at high tide. It is considered that an estimate of the amount of icing which could result with a specific pile design in a given geographical location would be of value. This paper outlines a method for estimating the amount of icing which could take place.

A slender vertical pile placed in a location where a large tidal variation occurs and the ambient temperature is below the freezing temperature for the sea-water is alternately subjected to freezing air temperatures and immersion in the sea-water. At low tide, the pile is exposed to the cold air while at high tide it is entirely immersed in the sea-water. Furthermore, at different vertical positions on the pile the amount of time exposed to either the cold air or the sea-water varies.

While the pile is exposed to the cold air it loses heat to the air. The amount of heat it loses is proportional to the temperature difference between the pile and the air, the thermal properties of the pile, the thermal conductance between the surface of the pile and the air and the time it is exposed to the air. When it is alternately immersed in the water, its temperature is below the freezing point of the water and water freezes to ice on the surface of the pile. The amount of ice so frozen is in direct relationship to the amount of heat lost while it was exposed to the air. In addition, while it is immersed in the water the temperature of the pile tends toward that of the sea-water but the ice remains. This is so because the ice freezes from sea-water essentially as fresh water ice which has a freezing point higher than that of the sea-water. The above process repeats itself with each tidal cycle and ice is successively built up in layers which vary in thickness with vertical position on the pile. This variation results in the characteristic carrot shape of the ice accumulation.

The amount of heat lost by the pile to the air at each vertical location may be estimated through the use of a dimensionless heat flow which is a function of the Biot and Fourier module. The Biot modulus compares the relative magnitudes of surface heat conductance to the pile's internal heat conduction and the Fourier modulus compares an approximate temperature wave penetration depth for a given time with a characteristic dimension which is the radius for the case of the pile. Once the amount of heat lost by the pile is known per tidal cycle, then knowing the typical winter climate for the area under consideration

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the amount of icing which may be anticipated to take place may be estimated.

A typical case is presented for a concrete pile and tidal and climate conditions similar to those which exist at Anchorage, Alaska.
POSSIBLE USES AND CONSTRUCTION METHODS FOR ARCTIC COASTAL PLATFORMS FABRICATED FROM FROZEN SEA WATER

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EXTENDED ABSTRACT

Development of the potential gas and oil reserves in the immediate offshore area of the Alaskan Beaufort Sea will require working bases or platforms from which to conduct the necessary operations whether it be for exploration drilling or production. The working bases may be mobile such as barges which may be sunk at the desired location and then refloated for use elsewhere. They may also be man-made islands fabricated from locally-mined gravel or from frozen sea-water. In general those platforms required for exploration purposes should be of a type which may be readily removed or mobile in nature should the exploration effort not be successful. Essentially they are of short life span and in the extreme may be abandoned after only one well has been drilled. Production platforms, however, are more permanent in nature and have a life span which must equal that of the field being produced.

Regardless of the intended use of the platform, its fabrication, operation and removal or abandonment will result in a definite impact on the local environment. This impact will vary with the type of platform used and its ultimate life span. Thus with both exploratory and production platforms it is recognized that some interference with the environment is inevitable and that an acceptable compromise between minimum environmental interference and maximum derived resource benefit must be reached.

Several concepts such as gravel islands, ice islands and barges have been considered as platforms for offshore exploration and production. Each of these has been found to offer relative benefits depending upon water depth and site location. Of particular interest is a platform fabricated from frozen sea-water for use on exploratory drilling operations in water depths shallower than twenty-five feet. The platform or ice island is of sufficient height that it rests on the sea floor and has adequate surface area to accommodate the drilling operation and support activities. It is considered that the ice island concept offers distinct advantages for use in shallow water and the feasibility and construction methods for such an island are the prime topics of this paper.

The ice island is built up by thickening the natural ice sheet at its air-exposed face by successive lifts of frozen sea-water. The rate at which the build-up will take place is a function of the climate existing at the time and the method used for sea-water application. Several methods of sea-water application are considered and their relative advantages are discussed. Freezing rates for the various methods are presented. Inherent in the freezing of sea-water is the condition of brine entrainment and its subsequent drainage. The brine content of the formed ice has a direct bearing on its strength and thus on the integrity of the structure. Comment on brine content, its drainage, and methods of control are made in light of present knowledge.
Loading of the ice structure both due to ice forces and the drilling operation are analyzed and discussed. Particular attention is paid to the available shear which exists at the interface between the structure and the sea floor in relationship to its ability to withstand horizontal ice loading without structure movement.

As the ice island and the exploratory drilling must be carried out in a time span of one winter season, scheduling of the various operations involved in the fabrication of the island are critical. Fabrication may not commence until the natural ice sheet is of sufficient thickness to support the construction operation. The fabrication period must be short enough to allow for drilling and demobilization before ice break-up. These time constraints dictate that certain operations may only be carried out during definite periods of the winter and that the ice must be built up at the maximum rate which the prevailing climate will allow.

A typical ice island construction project is outlined for an offshore Prudhoe location and time schedules and approximate costs are given for a specific mode of construction.

ABSTRACT ONLY AVAILABLE
NORTH SEA OFFSHORE STRUCTURES
ENVIRONMENT AND ENVIRONMENTAL LOADS

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ABSTRACT

An introduction on the North Sea environment is given. Availability and quality of data are commented on, and a brief description of ways of analysis is given. Finally comments are given regarding load calculations where the irregular behaviour of the sea is taken into account.

NOTE: The author has requested this paper be read in conjunction with the related papers, "North Sea Offshore Structures", by P. Bruun (p. 719) and "Safety Verification of North Sea Structures Practices and Trends", by J. Eri et al. (p. 791).
INTRODUCTION

The environmental conditions in the North Sea are of the most severe encountered in the offshore search for oil and gas. This goes particularly for the northern part where the so-called "100 year wave" is predicted to be in the order of 100 feet. Additional particular features are the severe conditions which may occur at any time of the year, although of course, with somewhat smaller probability of occurrence during the summer season.

The extreme wind and wave conditions represented a challenge to the designers of offshore structures who had to design for forces much larger than were usual. Evidently these challenges also intrigued a number of people not previously engaged in offshore design. This is seen from the abundance of new design proposals, new both with respect to material and technical solution.

The high frequency of storms also represents a problem with respect to operations offshore. Careful planning and good weather forecasting are essential factors in this connection.

THE NORTH SEA ENVIRONMENT

Data

In order to be able to adequately predict the environmental conditions, data from a long period of time is needed. Most of the existing data covering an observation period of 10 years and more are in the form of visual observations. Observations have been taken from weatherships, lightvessels, voluntary observing ships etc. These data are, of course, of varying quality. However, the great number of observations to a large extent evens out the inaccuracy of each individual observation.

Another point is that transfer formulae giving the relation between visual and instrumental data have been established, through analysis of simultaneously observed and instrumentally recorded data. Such a transfer function is expected to take care of most of the systematic error that may have influenced the quality of visual observations.

Nordenström (1973) derived a formula of the form

\[ H_s = a H_v^b \]  

(1)

where \( H_s \) is the significant wave height found from a recording, \( H_v \) is the visually observed wave height characterizing the sea state and \( a \) and \( b \) are constants.

The relation between periods is found to be of the same type.

During recent years more instrumental data have been recorded both by the oil companies, the authorities, and private companies. These data are of course for the larger part, of better quality than visual observations. However, it must be emphasized that such data must be recorded regularly for a long time before they can be used for predictions with sufficient accuracy.

It should be mentioned that Norwegian authorities have now issued regulations requiring both mobile and fixed offshore units to record environmental data when working on the Norwegian continental shelf.

The data situation is consequently rapidly improving.
Waves and Wind

A short term sea state may be characterized by a wave spectrum giving the distribution of wave energy with wave frequency. A commonly used spectrum is the Pierson-Moskowitz spectrum (Pierson and Moskowitz, 1964) which in modified form may be written

$$E(f) = \frac{\beta_1}{\pi f_m} \left(\frac{f}{f_m}\right)^{-5} \exp\left(-\frac{1}{\pi} \left(\frac{f}{f_m}\right)^4\right)$$  \hspace{1cm} (2)

$$\beta_1 = 0.178$$

$$\beta_2 = 0.71$$

$f$ is the wave frequency and $f_m$ the frequency of the peak of the spectrum.

In recent years rather strong indications have been given that the Jonswap spectrum may be more suitable for a fetch-limited area like the North Sea (Hasselmann et al., 1973). This spectrum is written

$$E(f) = a g^{2(2\pi)} f^{-5} \exp\left(-\frac{5}{4} \left(\frac{f}{f_m}\right)^{-4}\right) \cdot \exp\left(-\frac{(f-f_m)^2}{2 \sigma^2 f_m^2}\right)$$  \hspace{1cm} (3)

$$\alpha = 0.008$$

$$\sigma = 0.09$$

$$\nu = \text{peakedness parameter}$$

The comparison shown in Figure 1 reveals the most important difference between the two spectra, that the P-M spectrum distributes the wave energy over a wider range of frequencies than the Jonswap spectrum.

It is felt that the Jonswap spectrum is not yet sufficiently documented, particularly for severe sea states, but it obviously is of some importance for load predictions which spectrum is chosen.

It is generally recognized that the statistical distribution of individual wave heights is well described by the Rayleigh distribution

$$P(H) = 1 - \exp(-2(H/H_s)^2)$$  \hspace{1cm} (4)

and that the Weibull distribution is suitable for statistical description of the significant wave height $H_s$

$$P(H_s) = 1 - \exp\left(-\left(\frac{H}{H_c}\right)^\gamma\right)$$  \hspace{1cm} (5)

$P$ is the probability that the (significant) wave height is smaller than or equal to $H_s$. $H_c$ and $\gamma$ are parameters determined from the fitting of actual data to Equation 5.

Figure 2 gives an example of a Weibull distribution fitted to data from the rescue vessel Panita, N57°30', E3°.

As described by Pedersen (1971) the long term distribution of individual wave heights may be determined by

$$P(H) = \int_0^\infty (1 - \exp(-2(H/H_s)^2))dP(H_s)$$  \hspace{1cm} (6)
Figure 1. Comparison between the Jonswap and the Pierson-Moskowitz wave spectrum.

Figure 2. Weibull distribution of wave heights (Famita).
i.e., all short term distributions are summed with a weighting factor given by their probability of occurrence. The probability of occurrence is given by the probability density of the only parameter of the Rayleigh distribution, i.e., by \( dP(R_s) \).

The result of such a long term prediction based on the Famita data is shown in Figure 3.

Also wind velocities may be statistically described by a Weibull distribution (Fig. 4). Olsen (1974) has used data from the Famita and a similar procedure as given above to predict extreme values of the 10 minute average velocity (Fig. 5). Included in the figure is a scale giving return period, \( R_p \), found from the relation

\[
Q(V) = \frac{T}{R_p}
\]

(7)

\( T \) is the sampling time, 10 minutes in this case, and \( Q(V) \) is the probability of exceedance of the wind velocity \( V \). \( Q(V) \) is found from the fitting of the Weibull distribution to the actual data

\[
Q(V) = 1 - P(V) = \exp \left( -\frac{V}{V_c} \right) \gamma
\]

(8)

The fitting determines the parameters \( V_c \) and \( \gamma \).

Attempts have been made to determine statistical distributions for wave periods similar to those existing for wave height. So far, no good solution has been found.

As will be evident from the next chapter it is essential to be able to determine the wave period that should be associated with the maximum waves. Figure 6 gives an example of the simultaneous wave heights and periods in a given recording. The spreading is rather large. The results seem to indicate, however, a trend towards relatively smaller wave periods being associated with the higher waves, i.e., the wave steepness seems to be increasing with increasing wave height. The same trend is found from analysis of other recordings.

**ENVIRONMENTAL LOADS**

In a given sea state the wave energy is distributed over a range of frequencies as shown in Figure 1. Any offshore structure will react differently to waves of different frequencies both with respect to motions and loads. Figure 7, showing the heave transfer function for a semisubmersible, illustrates this fact. In order to account for the real wave conditions as accurately as possible one must take the energy distribution into consideration.

Referring to Figure 7, it is self-evident that the heave motion in a sea state with the spectrum peak at a wavelength seven times the length of the vessel, will be considerably larger than that in a sea state with the spectrum peak at \( \lambda/L = 3 \).

By applying the linear superposition principle, as illustrated in Figure 8, the contribution from all significant parts of the wave spectrum will be taken into account. The response spectrum will then represent the response of the structure in a given sea state. The statistical distribution of the response amplitudes is, as is the case of waves, represented by a Rayleigh distribution

\[
P(X) = 1 - \exp \left( -(X/E)^2 \right)
\]

(9)

The only parameter, \( E \), is related to the area, \( m_o \) of the response spectrum through

\[
E = 2m_o
\]

(10)

A link between the irregular behaviour of the sea and the corresponding irregular (time-wise) response of a structure is thus established.
Figure 3. Predicted long-term distribution of individual wave heights (Famita).

Figure 4. Weibull distribution of 10 minute average wind velocity (Famita).
Figure 5. Predicted long-term distribution of 10 minute average wind velocity.

Figure 6. Correlation between wave height and wave period for individual waves.
Figure 7. Heave transfer function for semi-submersible.

Figure 8. Principle of linear superposition.

Figure 9. Principle for calculating statistical long-term distributions.
If, in a similar fashion as was described for waves, one takes into account the probability of occurrence of the different sea states, one may obtain the long-term statistical distribution of the response amplitudes.

In this procedure both the variation of wave conditions in a given storm as well as the long-term variation of storm intensity if thus taken into account.

The different steps in such a procedure is illustrated in Figure 9. Further descriptions may be found in Nordenström (1973) and Pedersen et al. (1973).

Before being able to use such a procedure the following requirements must be fulfilled:

1. The wave spectrum must be known.
2. The long term variation of wave conditions must be known (data).
3. We must be able to determine the transfer function for the response in question.
4. The linear superposition principle must be applicable.

Items 1 and 2 have been commented on in the preceding paragraph.

Transfer functions may be determined by model tests or theoretical calculations.

Theoretical methods for finding transfer functions for motions and loads of ships have been available for some time (Salvesen et al., 1971). Similar methods have now also been developed for large volume offshore structures. It is necessary to take account of the disturbance from the structure on the flow field, and a diffraction theory is used (Faltinsen and Michelsen, 1975). This method seems very promising, as should be evident from the comparisons between theoretical results and model test results shown in Figures 10 through 12.

The linear superposition principle will be valid for a large volume structure while it is inapplicable on a jacket-type structure where drag forces dominate.

The procedure illustrated in Figure 9 should, in principle, be the most sound approach and the way in which actual conditions are most realistically described. How results from this statistical approach may be applied in a strength analysis is described in Røren et al. (1975).

Real life may, however, be different. The most commonly used method for determining the wave loads on an offshore structures today is the so-called design wave method. The loads on a structure are then determined for a single wave passing the structure. The "100 year wave" is commonly used for such calculations. The procedure described in the first paragraph is used to determine the wave height. The corresponding wave period is not that easily determined. Usually a wave period between 13 and 20 seconds is chosen depending on how the loads vary with wave period, and with due regard taken to the possibility of existence of the chosen height/period combination. Different combinations may also be chosen for different loads and different parts of the structure.

For jacket-type structures the design wave load is determined using the well-known Morison equation, while the loads on large volume bodies are determined experimentally and/or theoretically (Faltinsen and Michelsen, 1975; Røren et al., 1975).

The described method for determining design loads may seem somewhat arbitrary. One must, however, be aware of the fact that although much research is needed in a number of areas, in order to obtain good explanations and a rational approach, there is a large amount of experience upon which to base your choice: and the experience is rapidly increasing.
BOX, $L \times B \times d = 90m \times 90m \times 40m$

Figure 10. Calculated added mass, heave, compared with model test results.

DRIFT FORCES BOX, $L \times B \times d = 90m \times 90m \times 40m$

Figure 11. Calculated heave motion of floating box compared with model test results.

MOTIONS OF FLOATING BOX, $L \times B \times d = 90m \times 90m \times 40m$ AMPLITUDES

Figure 12. Calculated drift force on floating box compared with model test results.
CONCLUDING REMARKS

Further research and accumulation of experience are key words in our work for further improving the methods for determining environmental loads.

First of all, more and better environmental data are needed. The necessity of regular recordings over many years is emphasized. Hindcasting is a powerful tool that may be used before actual recordings become available. A method for determining the joint probability distribution of heights and periods should be developed.

The theoretical work well in progress for calculating wave loads must be continued. The need for full scale measurements to support the theoretical work is fully realized and a number of projects are well under way.

One must not, however, isolate the individual problem areas. In order to put things in the right perspective a total safety approach should be adopted, whereby, through considering causes and effects, the right weight is put on each part of the design work creating the complete structure.

REFERENCES


ICE FORCE RESPONSE SPECTRUM MODAL ANALYSIS OF OFFSHORE TOWERS

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ABSTRACT

The similarity between ice-force and earthquake records suggests the application of the well known Response Spectrum method in earthquake response analysis to ice-force response analysis. As far as the author's knowledge goes this is the first time response spectra have been developed for ice forces. This study describes the use of ice-force response spectra for dynamic response analysis of framed offshore towers subjected to ice forces. The structure analysed is a fixed three-dimensional tower modelled as a planar frame. The added water mass is assumed equal to the mass of water displaced and the masses per unit length of the members in the plane of the frame are computed by summing up the structural mass, the mass of water contained in the tubular members and the mass of water displaced. The masses of the members perpendicular to the plane of the frame are assumed to be lumped at the horizontal cross-brace level joints. The records are the continuous field ice force records obtained by Blenkarn at Cook Inlet, Alaska. The ice force records are digitised using the Nyquist Criterion and the stationarity of the chosen ice force records is verified by the Kolmogorov-Smirnov test. The mean values of the ice force records are subtracted to obtain the randomly fluctuating ice-force records. The displacement response-spectra for these ice-force records are obtained by a computer programme developed by Nigam and Jennings. The deflection time-history for the mean force was obtained using the STRUDL-II computer package. The probable maximum displacement is determined by adding the response spectra displacement values and the maximum of the time-history values.
INTRODUCTION

The similarity between ice-force and earthquake records suggests the application of the well known Response Spectrum method in earthquake response analysis to ice-force response analysis. The displacements at the horizontal cross-brace levels of a fixed offshore tower are determined from modal responses based on response spectra for Cook Inlet, Alaska, ice-force records. As far as the authors' knowledge goes this is the first time response spectra have been developed for ice forces. The work, aimed to help in the formulation of code requirements for ice-structure interaction, is part of the extensive programme in Cold Regions oriented Ocean Engineering at the Memorial University of Newfoundland.

REVIEW OF LITERATURE

Earlier analytical, experimental and field studies on the determination of static ice pressures on structures, primarily due to temperature fluctuations within the ice sheets, were summarized in considerable detail in the publications of Zubov (1945), Proskuryakov (1959), Korzhavin (1962), Drouin (1966) and Drouin and Michel (1974). They studied the influence of the thermal regime, the degree of restraint imposed by the shores and the discrete piers, the crystal structure of ice and the presence of cracks on the static ice pressure and discussed in detail numerous case studies. The comprehensive studies made by Korzhavin (1962, 1965, 1968), from 1933-1962, on the determination of ice pressures have been summarized by Michel (1970). The work included the effects of shape and inclination of the indentor, velocity, areal extent and nature of ice-floes, elastic deformations of the supports and water drag forces, on dynamic ice pressures on structures. Peyton (1966, 1967, 1968) described his extensive studies on the mechanical and structural properties of sea-ice (1958 - 1965), which included influences of temperature, salinity, rate of loading, crystal size and orientation, and depth of ice sheet, on strength, by testing nearly 3800 specimens in tension and compression. Different mathematical models were formulated for predicting ice strengths and verified with the experimental results. Peyton also described the continuous measurements of ice-forces on a test-pile installation at Cook Inlet, Alaska. The anticipated ice-conditions and the maximum ice forces in the Grand Banks, off the coasts of Newfoundland, were described by Blenkarn and Knapp (1968). Butyagin (1966), Weeks and Assur (1967), Gold (1970) and Lavrov (1971) studied the strength, deformation and failure properties of ice and ice covers. The effects of ice floes on coastal structures were discussed by Gerritsen (1971) and Bruun and Johannesson (1971). Edwards (1973) reviewed the characteristics and the problems of Cold-Regions-Oriented-Marine Technology and the state-of-the-art of dimensional analysis, mathematical and physical modelling, full scale testing of ice and the available ice model basins. A baseline study carried out in CRREL, Hanover, New Hampshire, U.S.A., and reported by Carey et al (1973), detailed the recommendations for future research programmes on ice-effects on hydraulic, riverine and offshore structure.

Shadrin and Panfilov (1962) conducted penetration tests on ice in conventional testing machines. Peyton (1966, 1967, 1968) formulated mathematical models from his extensive experimental work and correlated them by doing linear multiple regression analysis; the models showed correlation coefficients between 0.70 and 0.98. Nuttall and Gold (1966) carried out compression tests with semi-circular indentors, on columnar-grained laboratory-prepared ice blocks 3" high and 24" diameter, to determine the load that ice could carry without failure and observed a carrying capacity of 400 psi for two days without failure. The use of model basins to simulate full-scale ice-structure interaction was described by Voelker and Levine (1972) and Coon (1972). Afanas'ev et al moved piles, mounted on an overhead carriage through model sea-ice up to 1.4 inches thick. Kopaigorodski et al (1972) carried out model studies on ice sheets to...
determine the mean compressive strength and the dispersion of the values around the mean; they reported that sheets with small h/d (thickness to indentor width) ratios fail by instability while sheets with large h/d ratios fail as a result of the shear strains developed inside the ice sheets. Edwards and Wheaton (1972) carried out model tests with the U.S. Coast Guard Icebreaker Mackinaw and indicated good correlation between dimensionless impact load and the product of the Froude number and dimensionless flexural strength. Based on observations of noted crushing, splitting, shear, buckling, and bending failures in model tests by moving a pile, Nevel et al (1972) indicated that the failure mode has a pronounced influence on the peak nominal stress. Ryndel and Strahle (1973) did model tests on artificial ice (paraffin) by pushing a pile into a hand-packed (artificial) ice-sheet to determine the modes of failure and the continuous time-history of the force exerted on a lighthouse. Based on extensive model tests on laboratory-grown ice by pushing a pile mounted on a variable speed gear-driven carriage through ice, Hirayama et al (1973, 1974) and Schwarz et al (1974) suggested an empirical formula for the compressive strength of ice which takes into consideration the indentor shape and size, the relative velocity between ice and the structure. From extensive tests on ice, Roggensack (1974) concluded that ice strength is a function of confining pressure. Zabilansky et al (1975) carried out extensive and precise model tests by pushing vertical and sloped pile sections into ice and studied the effect of indentor shape, inclination and size, and the velocity of motion on the compressive strength of ice sheets; they developed an expression for the bending failures of ice sheets. Per Tryde (1975) conducted model tests on artificial ice sheets, made from a mixture of plaster of Paris, plastic granulates and several additives, and verified his empirical formula. Metge et al (1975) presented experimental techniques for testing the failure strength of ice sheets by moving a floating ice sheet against a stationary structure. Haugsoen (1975) indicated the need for extensive model testing to determine the forces ice will exert against various marine structures.

Gamayunov (1947) of Russia was the first to make attempts to measure the ice-forces of a bridge-pier due to moving ice-floes; since his load-measuring device was heavy, it did not respond properly to the force fluctuations and, therefore, did not measure the maximum ice forces correctly. Hogg (1952) and Wilmot (1952) of the Hydro-Electric Power Commission of Ontario measured static ice pressures on dams. Peyton (1966) reported ice-force measurements at Cook Inlet, Alaska, on a hinged test pile, 20' long and 36"Ø cylinder, supported on a temporary drilling platform; the forces were measured by a 300 kips load cell. The use of photogrammetry for measuring the movements of ice covers was reported by Korozhavin and Morgunov (1967), Van Wijk (1966) and Gold (1966), and these results were later correlated with independent ice-force measurements. Drouin et al (1972) described a technique to measure ice-floe velocities from bridges. Sanderson (1965) measured ice-forces on bridge-piers. Neill (1970, 1972, 1974), Neill et al (1972) and Sanden and Neill (1968) described measurements of ice-forces on several bridge piers at Alberta (1967 – 74), including the synchronisation of force recording and movie photography to compare force fluctuations with the nature of ice-failure. Blenkarn (1970) described the measurement and analysis of ice-forces at Cook Inlet, Alaska from 1964-1969. The ice-force records were those determined from two instrumented structural devices: i) a strain-gauged test pile driven into the ocean bottom adjacent to an existing temporary offshore drilling platform. ii) a field test beam hinged at both ends to the platform leg with a load cell measuring the reaction at the upper hinge. Croasdale (1970, 1974) developed a nutcracker ice strength tester for measuring ice-forces in the Beaufort Sea and observed that the ice-crushing strength varied from 600 to 900 psi; it was noticed that the strength was not sensitive to the loading rate. Schwarz (1970) correlated laboratory compressive strength tests on ice cubes with ice force measurements on a test-pile instrumented with an array of pressure cells, at Eider Estuary, West Germany. Kennedy (1966),

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Lazier and MacLachlan (1969) and Lazier and Metge (1972) investigated the deformations and movements in ice sheets due to temperature variation. The thermal cracks occurring in floating ice sheets were investigated by Evans and Untersteiner (1971). Ice pressure measurements on navigation light piers at St. Lawrence river in Canada were reported by Danys (1965, 1970) and Danys and Bercha (1975). Bradford (1971) carried out field observations in the Gulf of St. Lawrence to determine the relationship between wind and ice pressure distributions on navigation vessels. Atkinson et al. (1971) and Danys (1975) described the installation of a specially instrumented light pier to measure ice forces at Lac St. Pierre. Continuous in-situ ice-pressure measurements at the Volzhkaya and Mamanskaya dams in Russia and at the shore-fast ice off Barrow, Alaska on the Chuckchi Sea were carried out by Sinyavskaya (1972), Nelson et al. (1972) and Rogers et al. (1974). Macroscale strain and deformation measurements in ice shelves and pack ice have been reported by Zumberge et al. (1960) and Hibbler et al. (1973). Measurements of long term deformations and stresses in the Arctic Pim Island, between Ellesmere Island and Greenland, have been reported by Ito and Muller (1975). Goodman et al. (1975) and Ishida et al. (1972) described experimental methods for determining the elastic deformations in ice sheets and pack ice. Tabata (1971) used remote-sensing techniques to measure the deformations and movements of ice-fields. Measurements of water drag forces on an ice-floe, by pulling it over water with a boat, were reported by Susuki and Fujino (1971). Banke (1971) determined wind stresses over the ice and water of the Beaufort Sea using a sonic anemometer-thermometer. The ice thrust exerted by a moving unconsolidated ice cover against a horizontal boom was measured by Perham (1974) and Perham and Racicot (1975). The static ice thrust exerted on three dam gates in U.S.S.R. was measured by Sinjavskaya and Dick (1975) and correlated with the temperature and ice thickness. The measurement of stresses in ice sheets over long periods of time by embedding wide, thin and soft stress sensors was described by Metge et al. (1975). Instrumentation needed for the determination of ice forces and other dynamic forces on offshore structures was described by Geminiger and Pomonik (1975).

Assur (1959) presented a mathematical expression for the determination of the maximum compressive stress developed in ice covers by thermal expansion. Berdennikov (1964) analysed the problem of stresses in ice covers and discussed the relaxation of sustained stresses. Beccat and Michel (1959) and Michel (1966) investigated the forces developed on horizontal boom structures by an unconsolidated ice cover. The pressure exerted on a horizontal boom by broken ice was analysed by Berdennikov (1965). The failure of ice-floes, by shear and bending, on the sloped piers of the Northumberland Straits Crossing - between Prince Edward Island and the coasts of New Brunswick and Nova Scotia - was investigated by Lavoie (1966). The determination of the static ice thrust due to thermal variation was discussed by Drouin (1970). The analysis of the failure of the slow moving and fast moving ice-floes by impact on marine structures was carried out by Kivisild (1968). The work also discussed the buckling and vibration of ice-sheets. Kozhavin's formula (1962) was modified by Allen (1969) to make it applicable for very large diameter structures. Afanasev (1971, 1973) modified Prandtl's indentation formula to determine ice forces for h/d ratios varying from 0.1 to 20. Failure of ice-sheets on conical structures by shear and bending has been studied by Danys (1971). Dinkla and Sluymer (1970) analysed ice-pressures on inclined faces of offshore structures subjected to accidental impacts of pressure ridges. Assur (1971, 1972) presented a useful method to determine effective ice loading on vertical structures taking into account the effect of internal friction on the compressive strength of ice. The indentation solutions of Noble and Hussain (1969) were modified by Frederking and Gold (1971) for determining the ice forces exerted by a moving ice cover on an isolated pile. The strain rate, the temperature profile and the elastic properties of ice were incorporated into the formula for determining the ice pressures. Synotin et al. (1972) included the effects of the daily temperature cycles,
salinity, area and velocity of ice-floes, shape and size of the indentor in obtaining the static and dynamic ice-pressures on vertical and sloping faces. Per Tryde (1972, 1973, 1975) included the effects of the friction of the inclined surface, the eccentricity of the reactive force on the ice sheet, the wind forces, the velocity of ice-floes and the break-off distance in determining the ice-pressures on inclined surfaces. Reeh (1970, 1971) determined the break-off distance, the horizontal and vertical ice pressures, the deflection mode, the natural frequencies and the bending moments in the ice-sheets that fail by impact on sloped surfaces. Bercha (1975) and Bercha and Danya (1975) described the mathematical simulation of ice-structure interaction with particular reference to the adjustment of analytical models (obtained from classical theories) using three-dimensional finite-element simulators. Ross et al (1970, 1971) considered the elasto-plastic interaction between ice-sheets and a rigid off-shore pile in which the dynamic plane stress problem was solved by a computer code to obtain the complete stress fields in the ice sheet and the ice forces experienced by the pile. The effect of hummocked ice on the piers of marine hydraulic structures was studied by Dolgopolov et al (1975) and an empirical relationship was developed to estimate the ice pressure. From a dimensional analysis carried out to determine the failure mode of ice sheets by impact, crushing and bending, Gerard (1975) indicated that a minimum of three dimensional parameters is required to determine the failure mode of ice sheets.

The formulation of macroscale analytical models for pressured ice has been the subject of recent analytical work carried out by AIDJEX and many other investigators. These models will be needed to determine the ice forces on structures located in Arctic pack ice and shore fast-ice (Croasdale, 1975). Campbell (1966), Reed and Campbell (1960, 1962), Karlsson (1969), Glen (1970) and Campbell and Rasmussen (1971) developed three different types of rheological models for modelling the macroscale deformation of ice taking into account the influence of air stress, water stress, Coriolis force and the internal ice stress. Karlsson (1971) extended his viscoelastic model to a visco-elastic-plastic one. A thermo-viscoelastic model was proposed by Maykut and Untersteiner (1971) and a elasto-plastic ice model by Coon et al (1974) and Pritchard (1974, 1975). The model was used by Maser (1975), Colony (1975), Pritchard and Schweager (1975), Evans and Rothrock (1975) to study the large scale deformation of arctic pack ice. Evans (1975) analysed the usefulness and limitations of the present AIDJEX model. The recently proposed AIDJEX model for ice (Coon, 1974 and Pritchard, 1975) provides for the modelling of atmosphere, ocean and ice. Hibbler et al (1974, 1975) developed accurate predictive models for the deformation of arctic pack ice by using Glen's viscoelastic constitutive law. The predicted results were in good agreement with observed measurements.

Blumberg and Strader II (1969) considered the effect of ice-forces in determining the maximum response of the monopod offshore tower at Cook Inlet, Alaska, by replacing the horizontal dynamic ice-forces by a constant static ice thrust at the highest tide level on the structure. Assuming the primary structural response to be in its fundamental mode of vibration, Matlock et al (1971) analysed a cantilever pier, idealised as a single-degree-of-freedom (S.D.O.F.) system and subjected to a postulated saw-tooth type of deterministic ice loading. Since the loading was expressed as a function of space and time, the differential equation was solved numerically and the results checked with Peyton's (1966) field measurements. Sundararajan and Reddy (1973) studied the random responses of a S.D.O.F. model, to ice-floe loading of an actual field record of Blenkarn and assumed to be stationary and ergodic. The work was extended to a multi-degree-of-freedom model by Reddy and Cheema (1974). The analysis was further extended to a three-dimensional tower by Reddy et al (1975). The sampling of the field records of Blenkarn was done using the Nyquist criterion and the Kolmogorov-Smyrnov test. Määttänen (1975) carried out the dynamic analysis of bottom-founded offshore steel lighthouses subjected to ice forces and observed that the

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structure resonates depending on the ice properties, stiffness of the structure and the velocity of drifting ice floes.

Sommerville and Burns (1966) analysed the damages caused to two cells of a reservoir in Winnipeg by the vertical and horizontal components of the ice thrust on the upper row of slabs. Danys (1972, 1975) outlined design and analysis procedures adopted in the construction of offshore light-houses and lightpiers built in Lake St. Pierre, Lake St. Louis, St. Lawrence river, etc. The design, construction and operation of an Arctic Ice Model basin in ARCTEC was described by Edwards et al (1971). The problems encountered due to the execution of the Zuiderzee project, barrage constructions in the Rhine, Rotterdam Waterway deepening for irrigation, and the Delta project in Holland were described by Gerritsen (1971). The failure of two offshore lighthouses in the Gulf of Bothnia off Sweden was studied and analysed by Reinius et al (1971) and Bergdahl (1972). The structural design and analysis of a year round oil terminal in the ice-infested waters of the St. Lawrence river was described by Kivisild (1971). Jazrawi and Davies (1975) presented an offshore monopod drilling system for the shallow waters of the Canadian Beaufort Sea for year-round operations.

The values of ice-pressures recommended in the past, by the various codes of practice, have been conflicting owing to the lack of proper experimental and analytical verification. The American Petroleum Code (1971) has recommended pressures of 200-500 psi for crushing strengths of ice. The Russian Code of Practice (1966) has given detailed and explicit formulae for determining the static and dynamic pressures on walls, sloping surfaces, piers and piles. The ice forces, used in the design of offshore structures by the Norwegian designers, have been detailed in a publication of Det Norske Veritas (1974). The ice pressure values specified in some other codes, like the Canadian, Swedish, Finnish and American codes, have been excellently summarized by Danys (1975).

The use of the response spectrum (defined as a plot of the maximum values of a response parameter of a family of linearly elastic single-degree-of-freedom systems with different frequency characteristics and with a given ratio of system damping to critical damping when subjected to a ground motion time-history versus the frequency characteristics such as natural periods or frequencies of the systems) developed by Benioff (1934), Biot (1941) and Housner et al (1953) is now a standard technique in Earthquake Engineering. The 'probable' value of the maximum response of a multi-degree-of-freedom system is approximately the square root of the sum of the squares (SRSS) of the modal maxima (Rosenblueth, 1956) in the decoupled modes which are treated as independent one-degree systems. The individual modal values are determined from the use of the 'participation factor' (a measure of the extent to which the concerned mode participates in synthesizing the total load on the structure - Hurty and Rubinstein (1964).

THE PROBLEM

Procedure

The structure analysed is a fixed offshore tower analysed by Corotis and Martin (1975) for response to random wave forces (Fig. 1). The members are assumed to be rigidly connected and the added water mass assumed equal to the mass of the water displaced. This assumption for the added water mass has been found to be reasonable for the first few modes although there has been considerable discussion regarding possible frequency dependence and modified values for flexible members. The structural modelling is based on a two-dimensional representation of the tower assuming a constant dimension equal to the base length perpendicular to the plane. The masses per unit length of the
FIGURE 1: OFFSHORE TOWER—TWO DIMENSIONAL FRAME AND LUMPED MASS MODELS.

NOTE: FIGURE FROM COROTIS AND MARTIN WITH MODIFIED MASSES AND APPROXIMATED ELEVATIONS (*)
members in plane of the frame are computed by summing up the structural mass, the mass of the water contained in the tube and the mass of water displaced. The masses of the members perpendicular to the plane are assumed to be lumped at the horizontal cross-brace level joints. The force records are those of Blenkarn (1970) obtained at Cook Inlet, Alaska (Figure 2). The interval of digitization is based on the Nyquist frequency. The Kolmogorov-Smirnov test is used to verify that the ice-force records are drawn from identical continuous distributions. The mean value of the force distribution \( P_{\text{mean}} \) is deducted from each segment of the ice-force record \( P(t) \) to obtain \( Q(t) \). The displacement response spectra for \( Q(t) \) are obtained from a computer programme for generating response spectra for strong-motion earthquake records developed by Nigam and Jennings (1968). A unit mass is used to transform the force record to an acceleration record. Using the response spectra and modal participation factors, the displacements for \( Q(t) \) are computed for the first three modes and superposed by the SRSS method. These values are added to the maximum displacements obtained from the time-history response for \( P_{\text{mean}} \), to determine the maximum displacements at that level.

**THEORY**

The equations of motion for a \( n \)-degree-of-freedom system are

\[
[m] \{\ddot{y}\} + [c] \{\dot{y}\} + [k] \{y\} = \{Q\}
\]

where \( \{y\}, \{Q\} = \) random displacement and force vectors respectively,

\[ [m], [c], \text{and} [k] = \text{mass (sum of the structural and added water masses), damping (viscous equivalent of the structural and hydrodynamic damping) and stiffness matrices respectively.} \]

and \([c] = \alpha[m]\) in which \( \alpha \) is a constant.

The \( r \)th decoupled equation of Eqs. (1) obtained by the Normal Mode method Biggs (1964) is

\[
\ddot{\eta}_r + 2\zeta_r \omega_r \dot{\eta}_r + \omega_r^2 \eta_r = \frac{\{\phi_r\}^T \{q(x)\} f(t)}{M_r}
\]

where \( \eta_r = \text{normal coordinate (r=1,} \ldots, n) \),

\[
\zeta_r = \frac{\{\phi_r\}^T [c] \{\phi_r\}}{2\omega_r M_r}, \text{a fraction of critical damping,}
\]

in which \( \{\phi_r\} \) is the modal vector,

\[
\omega_r = \text{circular frequency of the} \ r\text{th mode,}
\]

\[
M_r = \{\phi_r\}^T[m] \{\phi_r\} = \text{generalised mass,}
\]

\( q(x) = \text{force distribution function,} \)

and \( f(t) = \text{time dependence} \)

The modal participation factors are defined as

\[
\Gamma_r = \{\phi_r\}^T \{q(x)\}
\]
FIGURE 2: ICE FORCE RECORD (BLENKARN)
(APPROX. ICE VELOCITY = 3 FT/SEC)
The responses of the one-degree systems (corresponding to the individual modes) in terms of the displacements \( y_{r, \text{max}} \) are read from the response spectra for different damping ratios in Fig. 3 for the corresponding periods. The modal amplitudes \( n_{r, \text{max}} \) are obtained by multiplying \( y_{r, \text{max}} \) by the corresponding participation factors and the 'modal mass ratio factor' \( \frac{1}{M_r} \). This accounts for the unit mass used to transform the ice force record to an acceleration record for the computer programme input. The displacements along the tower are computed by multiplying the modal amplitudes by the characteristic modal shape factors as given below

\[
y_{ir, \text{max}} = n_{y, \text{max}} \psi_{ir} / M_r
\]

where the subscript 'i' refers to the location of the tower.

**SOLUTION**

**STIFFNESS MATRIX**

<table>
<thead>
<tr>
<th></th>
<th>( 14211.3 )</th>
<th>(-8165.0 )</th>
<th>( 421.0 )</th>
<th>( 732.6 )</th>
<th>(-588.9 )</th>
<th>( 610.8 )</th>
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<tbody>
<tr>
<td>( 17569.9 )</td>
<td>(-10428.5 )</td>
<td>( 1336.0 )</td>
<td>(-944.2 )</td>
<td>( 880.2 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( 21368.0 )</td>
<td>(-11600.7 )</td>
<td>( 358.6 )</td>
<td>( 1238.1 )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( 11237.7 )</td>
<td>( 26330.9 )</td>
<td>(-24700.9 )</td>
<td>( 23934.0 )</td>
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<td></td>
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**DAMPING**

<table>
<thead>
<tr>
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<th>6% of the critical value</th>
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<tr>
<td>( {f} = 1.0279 )</td>
<td>1.8133 ( \phi )</td>
</tr>
<tr>
<td>( 3.5128 )</td>
<td>6.2578</td>
</tr>
<tr>
<td>( 8.5001 )</td>
<td>17.7868</td>
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**GENERALIZED MASS MATRIX**

<table>
<thead>
<tr>
<th></th>
<th>13.508</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 53.42 )</td>
<td>( 1039.65 )</td>
</tr>
<tr>
<td>( 2764.2 )</td>
<td>( 215.83 )</td>
</tr>
<tr>
<td>( 23.57 )</td>
<td>( 896 )</td>
</tr>
</tbody>
</table>

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FIGURE 3 RESPONSE SPECTRA FOR DAMPING RATIOS 0.005, 0.02, 0.04, AND 0.06.
Consider the first three modes which provide the major contribution to the modal response and also satisfy the 'thumb rule' of less than 10% errors for the first n nodes of a (2n + 1) lumped mass system.

**PARTICIPATION FACTORS**

\[
\begin{align*}
\Gamma_1 &= 0.3871, \\
\Gamma_2 &= -0.8321, \\
\Gamma_3 &= -5.6714
\end{align*}
\]

**MODAL RESPONSES**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period</th>
<th>Displacement ((y_{r,\text{max}})) (From Response Spectrum)</th>
<th>Modal Amplitude ((\eta_{r,\text{max}})) (= \Gamma_r y_{r,\text{max}} \frac{1}{M_r}) ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.012 sec.</td>
<td>0.3702 ft.</td>
<td>0.01061 ft.</td>
</tr>
<tr>
<td>2</td>
<td>0.544</td>
<td>0.1979 ft.</td>
<td>-0.003083 ft.</td>
</tr>
<tr>
<td>3</td>
<td>0.299</td>
<td>0.0483</td>
<td>-0.0002635 ft.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>(\phi_{4r})</th>
<th>(y_{4r,\text{max}}) (= \eta_{r,\text{max}} \phi_{4r})</th>
<th>Probable Maximum Displacement ((\text{at the load point - E1.300'})) ((91.5\text{ m}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.3871</td>
<td>0.004107</td>
<td>0.008166 ft.</td>
</tr>
<tr>
<td>2</td>
<td>-0.8321</td>
<td>0.002565</td>
<td>0.008166 ft.</td>
</tr>
<tr>
<td>3</td>
<td>-5.6714</td>
<td>0.001494</td>
<td>0.002489 m</td>
</tr>
</tbody>
</table>

**Constant Part of the Load \((P_{\text{mean}})\)**

Maximum displacement from time-history analysis = 0.02821 ft. (0.008598 m)

Total maximum displacement = 0.03638 ft. (0.01109 m)
DISCUSSION

The use of ice-force response spectra for the determination of the maximum numerical values of the responses permits the use of smoothed spectra which can be obtained as envelopes of an ensemble of records. The necessity for obtaining tedious and costly time-history responses and the dependence on particular records are avoided. The study contributes to the possibility of specifying code requirements for design response spectra on a regional basis (See USAEC Regulatory Guide 1.60, 1973).

The procedure described in this paper can be extended to non-linear structures by constructing inelastic response spectra curves following an approximate method formulated by Newmark (1971) for seismic analysis. If an increased number of ice-force measurements are available, a statistical processing of response spectra can be made for more reliable estimates of 'design spectra' as indicated by Newmark et al (1973).

While the number of field records available for force measurements is itself small, the authors have been unable to get any data on deflection responses. This indicates the urgent necessity for more field tests with adequate equipment to record dynamic displacements. Based on actual ice pressures measured in some Alberta rivers, Neill et al (1972) have indicated possible design pressures of 150-200 psi as compared to prescribed code values of 400 psi. Sanden and Neill (1968) cite an interesting example of the piers in Battle River, Alberta in which design ice pressures, of magnitudes one quarter of the code values, reduced the construction time by one-half and the cost by 40%. They also remark "the saving that would ensue from a reduction in specified ice forces justifies considerable effort by many organizations". In this connection it must be pointed out that considerable emphasis should be laid on instrumentation to directly measure the ice forces instead of determining them through structural responses (Peyton, 1966 and Blenkarn, 1970).

ACKNOWLEDGEMENTS

The support of the investigation by Imperial Oil Co. Grant No. 04-2026 and D.R.B. Grant No. 9767-08 is gratefully acknowledged. The authors wish to express their gratitude to Dr. R.T. Dempster, Dean, and Dr. A.A. Bruneau, Vice-President of Professional Schools and Community Services, Faculty of Engineering and Applied Science, Memorial University of Newfoundland for their keen interest and encouragement. Thanks are due to Mr. D.C. Heale, Instructor, College of Trades and Technology, St. John's for initial discussion that lead to the formulation of the project proposal. Valuable discussions on sampling of data with Professor D. Dunsiger and the computational assistance of Mr. A.K. Haldar, both from the Faculty of Engineering and Applied Science, Memorial University of Newfoundland, helped significantly in the work.


American Petroleum Institute, New York. 1971. API recommended practice for 'planning, designing, and construction - fixed offshore platforms'.


Colony, R. 1975. The simulation of arctic sea ice dynamics. III Int. Conf. on POAC, University of Alaska, Fairbanks.


Korzhavin, K.N., and V.K. Morgunov. 1967. Experiments for determining the ice pressure on engineering structures during the spring ice drift on Siberian rivers. IAHR XII Congress, Fort Collins, Colorado.


Määtänen, M. 1975. Experience of ice forces against a steel lighthouse mounted on the seabed, and proposed constructional refinements. III Int. Conf. on POAC, Univ. of Alaska, Fairbanks, 18 pp.


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Maser, K.R. 1975. A mechanical model for the deformation of arctic pack ice. III Int. Conf. on POAC, Univ. of Alaska, Fairbanks.


Per Tryde. 1975. Ice forces on slender structures. III Int. Conf. on POAC, Univ. of Alaska, Fairbanks, 2 pp.

Per Tryde. 1975. Ice forces acting on inclined wedges. III Int. Conf. on POAC, Univ. of Alaska, Fairbanks, 3 pp.


Pritchard, R.S., and R.T. Schwaegler. 1975. Applications of the AIDJEX ice model. III Int. Conf. on POAC, Univ. of Alaska, Fairbanks.


Rosenblueth, E. 1956. Some applications of probability theory in aseismic design. Proc. World Conf. on Earthquake Engng., Earthquake Engineering Research Institute and the Univ. of California, Berkeley.


Zubov, N.N. 1945. _Arctic Ice_. Izdatel'stvo Glavsevmorputi, Moscow (1945), Translated by the U.S. Navy Oceanographic Office and the American Meteorological Society (1963), 489 pp.

TECHNIQUES FOR THE STUDY OF ICE/STRUCTURE INTERACTION

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Production Research Laboratory
Calgary, Alberta
Canada

ABSTRACT

Design conditions for two classes of Arctic offshore drilling platforms have been investigated with unique experimental techniques. A facility was designed and built to permit the evaluation of the interaction between a conical bottom founded drilling platform and moving ice sheets. A small prototype structure is positioned in a large ice test basin with a built-in towing system which moves naturally grown ice sheets past the prototype structure. The design and operation of the basin is described and a typical qualitative result of an ice/cone interaction is presented.

An apparatus to study the interaction between cylindrical bottom founded drilling platforms and moving ice sheets was designed, built, and operated. Testing of floating lake ice sheets up to four feet thick is possible with an apparatus which permits the efficient handling of two-900 tonne capacity hydraulic rams and their load faces. A test consists of lowering the rams and load faces into a hole in the ice and forcing the load faces apart into the ice sheet. A mobile gantry supports the rams and load faces during moves across the ice sheet and two vertical legs support the entire apparatus on the shallow lake bottom during tests. The system operation is presented along with a qualitative result.
INTRODUCTION

In recent years, exploration for oil and gas has moved into frontier areas of Canada. Most recently, activity has progressed onto the continental shelf regions in the Arctic. As is well known, this part of Canada presents unique environmental conditions, which include an ice cover and very low temperatures for most of the year.

One section in which Imperial Oil Limited has been very active is the MacKenzie Delta Region of the Northwest Territories identified in Figure 1. Land based exploration has seen expansion into the Southern Beaufort Sea by means of artificial islands which serve as drilling platforms. The development of tools with which to explore this area of the Arctic for petroleum reserves has been of the utmost importance in the last few years.

Various concepts have been proposed for drilling platforms in the continental shelf regions of the Arctic and the bottom founded structure is one of the leading contenders. In current Beaufort Sea practice, nine artificial islands, Hayley and Sangster (1974), have been built by Imperial Oil Limited (seven), de Jong et al. (1975), and Sun Oil Company (two), Brown and Barrie (1975), and two ice strengthened drillships may be introduced into the area. However, the bottom founded concept is not limited to islands and includes structures of steel and concrete which are mobile at least in the sense of being refloatable and movable to new locations.

In any case, the drilling platform, whether bottom founded or not, will be required to interact with the indigenous ice cover to maintain its drilling station. The platform designers will require a detailed knowledge of the ice conditions in the intended area of operations. An example of the expected interaction of an ice sheet with a conical bottom founded structure is presented in Figure 2. The Small Prototype Cone structure shown will be discussed below.

Figure 1. MacKenzie Bay Area of the Southern Beaufort Sea

Figure 2. Small Prototype Cone structure

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Typical ice features in the Southern Beaufort Sea which must be considered are sheets, floes, pressure ridges (Fig. 3) and ice islands, Kovacs and Mellor (1974). Of importance in quantifying these features are their size and mobility. Analyses can be made to establish probabilities of occurrence of extremes in the variables which serve to quantify these ice features. For completeness, it should be stated that a knowledge of ice properties quantifying strength and deflection characteristics of all ice forms is essential.

A particular bottom founded structure will interact with the indigenous ice uniquely. Combining an understanding of ice conditions and properties with observed or probable interaction mechanisms leads to an assessment of loads which a structure must be capable of withstanding.

As well as the artificial island concept, two other bottom founded structural schemes have received considerable attention. These are the cylindrical, Jazrawi and Davies (1975), and cone shaped structures.
DESIGN ANALYSIS OF BOTTOM FOUNDED STRUCTURES

Experimental facilities were designed and built by Imperial Oil Limited to investigate the forces and mechanisms involved in the interaction of conical and cylindrical structures with ice features.

Conical Structures

It is well known that an ice sheet impinging on a conical structure will fail in bending, Croasdale (1974). This interaction can be studied in several ways. First a full size "test structure" can be built. This alternative is heavily dependent on nature producing conditions which may be characterized as the "design conditions" for the concept. The already extreme cost of such an enterprise is amplified by the implication of some over-design. Second on the list is the construction and monitoring of a Small Prototype structure which could be deployed in the actual environment which will be encountered by the full size structure. Cost again is high but can be made acceptable if the ice conditions can be controlled somewhat. A drawback is that the loads and failure mechanisms must be related by some sort of analytical technique to a larger structure. A third scheme which requires a high degree of technical sophistication is scale modelling in one of several available modelling mediums. This technique is applied to the design of ice breakers in a fairly routine fashion. However, for application to offshore structures, the lack of full size data with which to compare results is a drawback. Finally, a fourth approach is Mathematical Analysis. Here some knowledge of the interaction mechanisms and, therefore, results from the other techniques are required before realistic analyses can take place.
Imperial Oil has chosen a combination of the Small Prototype Structure interacting with natural ice and scale modelling; outlined in Table 1, in an effort to develop a safe efficient drilling platform. In theory, full scale natural ice data generated by a Small Prototype can be applied to the calibration of a modelling technique which can then be used to optimize a design. The following section constitutes a description of the Small Prototype Cone experiment.

<table>
<thead>
<tr>
<th>SMALL PROTOTYPE TESTING</th>
<th>COMPARE RESULTS</th>
<th>CALIBRATE MODELLING TECHNIQUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>MODELLING OF SMALL PROTOTYPE RESULTS</td>
<td></td>
<td>MODELLING VALID</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MODELLING INVALID</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OPTIMIZE STRUCTURE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MORE PROTOTYPE TESTS</td>
</tr>
</tbody>
</table>

**TABLE 1. Small Prototype and Model Testing Program**

Small prototype testing

In order to bring the Small Prototype technique into reality, the outdoor Ice Test Basin, Figure 4, was constructed to house a Small Prototype Cone and subject it to moving ice features under semicontrolled conditions. The facility is located at the Imperial Oil Limited Production Research Laboratory in Calgary, Alberta.

Figure 4. Ice Test Basin
Since long periods of sub-freezing temperatures are available, this location offers great advantages over a field location. Initially, the entire program is much less costly and, of course, operationally convenient. Although naturally grown ice is used, some control is possible. Ice movement rates, ice thickness, ice features, ice properties and with patience, temperature (because of frequent temperature swings caused by Southern Alberta’s "Chinooks"), are variable. Ultimately, complete control could be exercised by the addition of a refrigerated enclosure.

As shown schematically in Figure 5, the Ice Test Basin is a reinforced concrete pool measuring 31 by 55 metres. In its present application, a Small Prototype Cone is mounted in the deep end. The depth varies from 1.4 metres over one half of its length down to 3.1 metres around the structure by a sloped transition section. A 300KW towing system can move the ice sheet, once it is cut away from the wall, past the structure on over the end wall into an ice collection pit at rates ranging from 0.91 to 16.7 metres per minute. A maximum horizontal force of 81 tonnes can be transmitted into an ice sheet.

Figure 5. Schematic Plan of Ice Test Basin

In the design of any experimental facility, the avoidance of unrealistic constraints or other effects is of the first priority. The 10 to 1 ratio of ice sheet width to structure diameter achieved in this system has in practice shown no influence of the ice sheet/cone interaction at the sides of the basin. Even with two feet of ice, the facility presents an essentially infinite sheet to the Small Prototype Cone.

The purpose of the basin is to provide relative movement between a structure and an ice sheet. This implies that either the structure or ice can move if velocity effects are ignored. In previous smaller scale designs in terms of horizontal loadings, the structure moves through a stationary ice sheet. This is justified because the small size results in manageable forces. Considering structure movement at the present scale, one is faced with an overhead gantry spanning 31 metres or an underwater "railway". Both concepts are very expensive and the "railway" is relatively inaccessible for maintenance. Also, both alternatives have the drawback that the movement machinery is physically joined to the load measuring system in which case velocity effects can be artificially introduced into the data. It is, therefore, both practical and realistic to move the ice sheet past a stationary structure.
The force required to move the ice sheet past a structure is transferred to the ice sheet through a floating boom resembling a spinal column, Figure 6. It consists of three-one inch diameter wire ropes with 1 metre long Hemlock blocks clamped around the cables. At the beginning of each winter, the boom is laid out in a semi-circular shape with its ends parallel to the longitudinal walls and left to be frozen into the ice sheet. The wooden blocks have steel crampons attached to grip the ice during spring conditions when the adfreeze bond between the wood and ice is broken by rising temperatures. Once the ice becomes more than 15 cm. thick, the boom has worked satisfactorily even with ice sheets approaching 60 cm. in thickness.

Each end of the boom is connected to a wheeled car called a trolley on each wall. Each trolley, pictured in Figure 7, connects to the boom through a trailing arm whose end is rigidly held at ice level. Each trolley has wheels which run in channel section tracks on each side of both walls. A trolley reacts against the wall the moment generated by the misalignment between the ice sheet resisting force and track level pulling force.
Figure 7 shows the styrofoam lining around the pool designed to absorb ice sheet stresses and limit local thickening of the ice sheet near the concrete wall.

Each trolley is connected to a continuous chain/wire rope drive loop running in the horizontal plane. A double roller chain is used on the high tension side of the loop. Its stiffness is high to limit "spring back" during load fluctuations. The low tension part of the drive loop consists of two one-inch diameter wire ropes which with a turnbuckle provide a pretensioning system. A double sheave is used on the cable part of the loop and double sprockets on the chain section. The former serves as an idler and the latter is the driven member.

The sprocket is driven directly by a roller chain from a larger sprocket which is attached to the output shaft of a worm gear reducer. This component is driven by a hydraulic motor/pump combination. Primary power is derived from a 150 KW induction motor. Symmetrical systems are built into each wall.

The only contact between the systems is a dual closed loop servo-control system which governs the speed and position of one trolley with respect to the preset speed of the other. In practice, the system has functioned well.

As a moving ice mass nears the end of the basin past the structure, it is constrained to ride-up a 1:5 sloped water level weir; see Figure 4; which serves to retain the water but let the ice pass over into an ice collection area.

Although full strength fresh ice is acceptable for use in the basin up to 15%, salt content water is used in practice to decrease the incidence of thermal cracking.
The Small Prototype Cone shown in Figures 2 and 4 has a diameter of 3.1 metres at the waterline from which it rises at 45° to a 60 cm. diameter cylindrical neck. It is supported on the bottom of the basin by a 3.7 metre diameter solid reinforced concrete cylinder.

Load transfer from the cone to the foundation cylinder is affected through a force sensing system shown schematically in Figure 8. At the waterline, the cone is anchored to the foundation by three sets of load cell pairs. Each pair is composed of a horizontal and a vertical bolt whose longitudinal stiffness; 4.5 diameter; greatly exceeds its lateral stiffness; 45 cm. effective length. In this way, the horizontal and vertical forces are largely decoupled at each support point.

Figure 8. Schematic of Small Prototype Cone Force Sensing System.

The deflection of each bolt is measured by three equally spaced longitudinally oriented LVDT elements. Electronic averaging of output signals minimizes bending induced errors.
Calibration is affected by applying hydraulic loads at selected points within the cone in both horizontal and vertical directions.

On test day, activities include the following:

- **Flexural strength of the ice sheet** is measured by floating cantilever and submerged simply supported beams under third point loading as in Figure 9. Elastic modulus is measured by central loading of the ice sheet and beam deflections during flexure testing. Beam tests are conducted on the ice near the basin sides, and to the back and sides of the cone to eliminate any effects the testing may have on the interaction between the ice sheet and structure.

- Each ice sheet is marked with gridlines to assist in matching motion picture records with oscillograph recordings of force level.

- Shortly before moving the ice sheet, it is cut away from the basin walls. Chain saws, a 1.2 metre diameter circular saw and steam lances are effective cutting tools.

- With preparations complete, the ice sheet is accelerated to a preset velocity which can be varied during a test. A single test can consume up to 32 metres of ice sheet.

- Once the ice sheet is consumed and cone ice free, a calibration of the load sensing system takes place.

- Typical ice action on the Small Prototype Conical Structure appears in Figure 2. Interesting ride-up action can be seen in the weir behind the cone.

- Not all of the ice is pushed over the weir. Excess sheet and broken pieces are removed with a two rope crane using an expanded metal basket.

Test programs have been conducted in this facility for the past two winters to produce data on the interaction of natural ice sheets and natural ice beams, simulating multi-year ice pressure ridges, with a Small Prototype Structure.
Cylindrical Structures

Cylindrical bottom founded structures exhibit a somewhat different interaction with ice fields. An ice sheet initially stationary and then set into motion by wind stress will tend to crush against a structure with vertical sides. This failure condition produces very high loads which a designer must know.

An investigation of this phenomena has led investigators to cover the spectrum from small scale laboratory experiments, Schwartz et al. (1974), with a few inches of ice to full scale test structures positioned in a natural ice environment.

Large Scale Crushing Tests

Imperial Oil has chosen to perform large scale laboratory type crushing tests in natural lake ice. Toward this end, a portable testing apparatus capable of duplicating the action of moving ice sheets encountering vertical cylindrical structures was developed to extend the work of Croasdale (1974). The equipment can be moved over an ice sheet from one test site to the next. Figure 10 shows the equipment in place on Eagle Lake near Calgary, Alberta. Clearly visible are the insulated hoods and refrigeration units used to control ice temperature.

Figure 10. Crushing Test Apparatus in Position on Eagle Lake, Alberta.

As schematically shown in Figure 11, the apparatus consists of a wheeled gantry which supports the 907 tonne capacity hydraulic rams and load face (indenter) assembly. One indenter, flat or circular in shape, resembles a structure and the other a reaction face. To avoid unrealistic load combinations, the unit in test configuration is supported off the shallow lake bottom by legs much like an offshore jack-up drilling rig.
Preparations for a test series begins months ahead of the first tests. In early winter, the natural lake ice sheet is removed from rectangular areas and these test ponds are seeded with blown snow to produce the horizontal C-axis crystals typical of sea ice.

Reasonably, the size of test ponds should be sufficient to avoid edge effects from the thicker natural ice sheet. Also, it is dependent on indentor width. In practice, a 5 metre wide test pond is used for the 1.2 metre wide indentor shown in Figure 12.

The preparation for a test begins by cutting an appropriately shaped hole in the test pond and the adjacent natural sheet, then positioning the gantry and finally lowering the rams and indentors into the hole. The "structure like" indentor bears against the thinner test pond ice and the reaction indentor on the thicker natural ice. Typically, the ram and ice sheet centre lines coincide. To complete preparations, the apparatus is left in position under the cooling hoods overnight to "freeze-in" the indentor. This corresponds to the condition of an ice sheet forming undisturbed around a structure.
Figure 12. A Semi-Circular Indentor Penetrating an Ice Sheet.

The aftermath of an actual test is pictured in Figure 12. In this case, a 1.2 metre diameter semi-circular indentor activated by a single ram was used. For these tests, a jig mounted chain saw was used to cut a semi-circular hole in the test pond. The ram, structure-like indentor, jack-up legs and gantry wheels are clearly visible. The wheels run in tracks which steer the apparatus and spread its load on the ice sheet.

During a test imbedded thermisters are used to measure ice temperature profiles near and away from the structure-like indentor within the test pond ice.

As seen in Figure 14, only some snow ice spalling is evident after a failure.

Instrumentation includes ram pressure transducers and displacement measuring rotary potentiometers coupled to a tensioned reel which is activated by the relative movement between load faces.

After indentor movement has resulted in ice sheet failure, the rams are depressured and the ice sheet is examined to determine the failure mechanisms involved and ice samples taken for unconfined compression and plane strain tests as well as crystallographic analysis to characterize the ice.

This apparatus has been proven in three winters of testing. Up to 1.2 metre thick ice sheets have been failed. Structure-like indentors used range from 30 cm. diameter semi-circular to 4.2 metre wide flat. A test rate of one per day has been achieved showing the apparatus to be an economical producer of very large scale data.
Recent Applications

Presently, the Ice Test Basin is being used to investigate the phenomenon of ice ride-up and pile-up on scale models of artificial islands. In these test series, it is important to scale the ice properties to the geometric scale of the island. An upward breaking flexural strength of 103 kPa measured in third point loading on a beam ten times as long as it is thick has been achieved.

The crushing test apparatus shown in Figure 10 has been modified to investigate the ice failure mechanisms involved in an ice sheet continuously moving against a cylindrical structure. A capability of 3.6 metres of movement through a 0.67 metre thick natural ice sheet has been achieved. All continuous penetration data can be related to initial failure results from the tests described above.

ACKNOWLEDGEMENTS

I would like to thank Imperial Oil Limited, for permission to present this paper.

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Finally, many thanks to colleagues and co-workers who have contributed to the topics discussed herein.

REFERENCES


VERIFICATION OF NUMERICAL WAVE COMPUTATION
RECONSTRUCTION OF EXTREME STORMS

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ABSTRACT

Based on accurate wave measurements off the Norwegian coast we have tried to verify the numerical wave model in use at the Norwegian Meteorological Institute. We conclude that the model is adequate, with the exception of a few special situations, and note that any such model will have an inaccuracy of about 15 percent.

Our present work is concerned with mapping the wave climate at the entire Norwegian continental shelf, based on visual observations at 15 lighthouses along the coast in the 25-year period, 1949-1973.
VERIFICATION OF NUMERICAL WAVE COMPUTATION

We have obtained some accurate wave data from the Division of Port and Ocean Engineering at the Technical University of Norway; namely, data from the position 70.6°N, 21.8°E in the period 9 March 1971 to 7 March 1972 and from the position 64.2°N, 9.1°E in the period 10 October 1972 to 4 May 1973. The wave measurements were made by use of a Dutch Datawell Waverider (WR). The WR, which is described in detail by Houmb (1974), measures primary vertical acceleration.

In our work with a rather extensive hindcast project which is intended to cover the whole Norwegian continental shelf, we have tried to compare these wave measurements with computed values, using the wave model at the Norwegian Meteorological Institute. This model worked out by Haug (1968) has been slightly modified, the last time being early in 1974.

The treatment of one weather condition requires weather maps for a period of four days in order to get a satisfactory swell computation. We may, as an alternative, use only two days if we are primarily interested in the wind-driven sea. From the weather maps, in a grid system with grid distance 300 km, we read off pressure values with a precision of 0.5 mb with as much as 12 times 12 points. The computation machine in turn interpolates between these points. Taking into account the effects of friction and stability wind values are computed. Originally the effect of curvature of the isobars, as well as isallobaric effects, were taken into account, but these corrections in most conditions have probably been overvalued and are accordingly neglected. The output from the machine is wind direction and force, significant wave height, average period, maximum and minimum period in every grid point. We also get computed sea and swell separately at selected points.

In 10 diagrams we compared measured values every three hours with corresponding computed values every six hours. The actual weather conditions determined which of the grid points to be most representative for the actual WR.

GENERAL INACCURACY

The analysis of the weather maps, however good it may be, will only give an approximately correct wind field. Having well-defined wind fields and moderate wind forces we must admit an "error" in the order of ± 2 kt. An example with fully developed sea (Kinsman, 1965) will probably clarify this object but also urge the need for accurate analysis: Having wind forces like 40 kt with an inaccuracy of ± 2 kt will result in an inaccuracy of ± 2 m with regard to computed wave height.

The (historical) observed pressure values will most likely have their inherent errors, which, however, are mainly uncontrollable. However, the inaccuracy in time of observation may give us an estimate. For example, with a pressure fall of 6 mb in three hours (not unusual) a variation in the time of observation of ± 0.5 hour will give a pressure value with an inaccuracy of ± 1 mb. If there is then a rapidly increasing wave height a time displacement of 0.5 hours may very well result in a wave height displacement amounting to 0.5 m.

Besides the uncertainty in the input values, the numerical computation is nothing but an approximation. Taking into account these mentioned uncertainties we consider our wave model in most cases, as sufficiently accurate, but the quality of the model will vary from one situation to another.

CONCLUSION

As would be expected, our results show the model to give a more continuous, and to a certain extent, a smoother variation of wave height than what is found in the direct measurements. The differences between measured and computed values can partly be explained by
the existence of local vorticity centers which our method of analysis and our pressure observations are not able to show. Nor can we exclude the influence of existing currents, cross-sea and refraction.

No uniform difference appears. Sometimes the measured wave heights are greater than the computed ones, sometimes they are smaller. *A sound estimate of the mentioned inaccuracies in the input values results in an inaccuracy of the computed values of about 15 percent, independent of the quality of the model.*

We have reason to believe that the wave model used may very well be in use in our subsequent statistic analysis intended to take place in our hindcast, although the results in extreme weather conditions ought to be appraised and perhaps corrected according to our present experience. Although our model is somewhat simple, using the Neumann spectrum, it is practical and fast-working. Our 10 conditions were treated in about 35 minutes.

**COMPARISON BETWEEN OBSERVED AND COMPUTED WAVE HEIGHT**

Direct Comparison

The visual observations of wave height are reported in classes according to following table.

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<th>Class numbers</th>
<th>Wave height intervals in meters</th>
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<td>4</td>
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<td>9</td>
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</table>

First we make a direct comparison.

As calibrating material we have chosen the period from 1 November 1972 to 31 December 1973, a time period of 14 months, with observed S from 14 lighthouses along the Norwegian coast from Ferder to Vardø.

Our complete material gives observed values of S, air pressure, wind direction and wind force every 6 hours in a 25-year period from 15 lighthouses. One of these stations did not observe in our chosen period.

Our wave model gives computed wave height and swell once a day in corresponding grid points along the coast. Among the four daily observations we have chosen only one, namely the one made at mid-day which is assumed to be the most reliable, at least in the winter darkness of the northern coastal regions.

Our testing period is thus limited to only one observation each day for 14 months. This is done for merely practical reasons.

The computed values are converted according to the table above, thus getting S values from 0 to 9 instead of values in dm. The values for wind-driven sea and swell are compared, and the greater one is the one being converted, then compared with the observed value.
For each station we have drawn histograms for observed and computed frequencies of S. The observations are showing too little dispersion and mostly too little mean value. Figure 1, representing the station Ona, is showing this as an extreme case. As to the other stations this disproportion is more or less present, with a mean standard deviation (σ) for the 13 stations close to 0.9, while the computations give a mean σ close to 1.5.

These results which are somewhat disappointing, are nevertheless very important in our further work, despite any discrepancy in the observations to be based on that material.

Observations Without Offshore Winds

According to the foregoing the observations must necessarily be corrected.

There are many reasons why the observer at the lighthouse is not able to give the true wave conditions on the high seas. We may obviate one of these reasons by omitting all observations having offshore wind.

The calibrating material then becomes somewhat reduced. The sector giving offshore wind varies from 30° to as much as 220°, the mean for all stations being 150°. The accordance between computed and observed values become better, and on the whole the observed mean values and dispersion increased. Compare Figure 1 and Figure 2.

Nevertheless, it is quite obvious that the observations still deviate from the computed values, and the deviation is not inconsiderable, as might be expected. The observations are not made on the high seas, but are under the influence of refraction, sea bed conditions, local current conditions and cross-sea. The result is too small observed frequencies for the highest S values. Besides, we often get an accumulation in one distinct interval of observation, caused by the natural inertia of the observer, his tendency to observe the same as he did on previous occasions.

The Establishing of a Transfer Function

It is necessary to further correct our observations. The most obvious corrective is, of course, our computed values (the veracity of which we have determined). Therefore, we try to establish a transfer function between computed and observed values.

In Figure 2a we have drawn a histogram for computed frequencies corresponding with the observations having S=4. This has been done for all the stations, also for S=3. For those stations having enough observations we also have drawn histograms for S=2 and S=5.

These histograms are satisfying: except for very few they show an almost exact normal distribution with approximately zero skewness, justifying our use of the following normalized transfer function, TF:

$$\text{TF} = \frac{1}{(\sqrt{2\pi})^n} \exp \left( -\frac{(x + d)^2}{2\sigma^2} \right) \quad \text{where} \quad x = 0, 1, 2, \ldots, 9$$

We thus apply an error function although the computed and observed values are not from the exact same sample.

The variation of the values of d and σ along the coast is mostly very small. We mean for certain that much of this variation is of random nature. An extended material would result in a smoothing out of the variation. It is then unnecessary, and even wrong, to apply one special TF for each station.

When fixing the general d and σ values we have to have consideration for the following:

a) How good, or reliable is our calibrating material?
Observed frequencies

\[ \bar{S} = 3.19 \]

\[ \sigma = 0.59 \]

Computed frequencies

\[ \bar{S} = 4.12 \]

\[ \sigma = 1.55 \]

Figure 1. ONA 426 observations
Figure 2. ONA without offshore wind
292 observations
Observed frequencies
\( S = 3.27 \)
\( \sigma = 0.66 \)

Figure 2a. Computed frequencies related to the 96 observations having \( S = 4 \)
\( S = 5.40 \)
\( \sigma = 1.02 \)
\( d = 5.40 - 4.0 = 1.40 \)
b) Two neighboring stations must give approximately the same frequency distribution after being transformed.

c) We have comprehensive wave observations from the weather ship M, Polarfront, in the Norwegian Sea and from the rescue vessel Famita in the North Sea. Our transformed observations from the lighthouses must give a frequency distribution not essentially different from the conditions on Polarfront and Famita.

d) A transformation of the calibrating material must give a distribution as equal to the computed distribution as possible.

A thorough estimate of the above considerations led to the general TF given in Table 1.

Two stations, namely Torsvåg and Frøholmen, stand out among the others, as having considerably smaller d values. They are given a distinct TF, presented in Table 2.

The Transformation Procedure

We now are able to transform the whole lighthouse material without offshore wind by means of our transfer functions.

Example: If any station, e.g., Ona, has observed S=6 1000 times altogether, the transformed frequencies, according to Table 1, will be 80 in the group S=5, 420 in the group S=6, 420 in the group S=7, and 80 in the group S=8.

We thus mean to have established observations representing the conditions close to the coast, still under the influence of, e.g., refraction and the sea bed, but systematical failings in the observations should be corrected.

Out to Sea Together With Offshore Wind

We now wish to translate the observations away from the coast in certain steps in order to make them more representative.

Our wave model can also work with optional grid distance. We have in this connection chosen the distance to be 12.5 km. In this grid system we apply an offshore wind field with 20.30 and 40 kt in succession with direction step of 30°. We get computed wave height every 12.5 km offshore established in 6, 12, 18, and 24 hours.

Each of the lighthouses has its own prepared wind statistics, e.g., percentage number of days with offshore wind in a given direction and force. We put out to sea along a central direction in the offshore wind sector and count the computed frequencies with given wave height, 5, applying the different wind forces and wind directions. At present we work at a distance of 20 NM from the coast. The farther we get offshore, the more we may misuse the wind statistics, being exactly valid only at the lighthouse on shore.

Based on observations every hour from the weather ship Polarfront we have found the mean duration of wind forces of 30 and 40 kt to be very close to 6 hours. Thus we, on the average, use the computed values established in 6 hours. However, we cannot neglect the extreme situations, that actually exist, having much longer durations.

Wind forces of 20 kt give fully developed sea in about 10 hours and give, virtually, always S=4, i.e., wave height between 1.25 m and 2.5 m.

Having offshore wind forces below 15 kt we transform the observations as before according to Table 1 and Table 2.
### TABLE 1. General TF.

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### TABLE 2. TF for Torsvåg and Fruholmen.

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932

Småland 8
CONCLUSION

Our final result then should represent the imaginary observations 20 NM offshore. If our reasoning is correct and our sources of error are not too deep, we possess material that in a unique way gives us wave measurements on the high seas along the coast in a 25-year period; data which for a long time has been missing, and not only in Norway.

We will ultimately calculate statistics from this material and work out both Weibull and Gumbel distributions. This work will be presented in a forthcoming report.

REFERENCES


REINFORCED ICE: ITS PROPERTIES AND USE IN CONSTRUCTING TEMPORARY ENCLOSURES

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The University of Calgary
Calgary, Alberta, Canada

ABSTRACT

This paper describes part of an experimental programme aimed at investigating some properties of reinforced ice and its possible use in the construction of domes for temporary shelters in arctic conditions.

Glass fibre yarn was used to produce reinforced ice samples. These were tested to determine the effect of such reinforcing on the tensile and shear strength and to establish the bond behaviour of such reinforcing yarn in ice.

Reinforced ice domes were constructed by spraying water onto an inflatable shell made of reinforced vinyl cloth supporting a network of glass fibre yarn. The domes were built up to a designed thickness of 10mm and were tested under axisymmetric distributed dead loading. Their behaviour up to collapse was observed and is described.
INTRODUCTION

Due to the quest for further energy supplies from the north of the continent, construction techniques in cold regions are becoming more and more important. Convenient temporary shelters are one important aspect of such construction activity. It is suggested that reinforced ice domes may be an efficient and economic solution to this problem.

The material properties of plain ice have been the subject of extensive study (Gold, 1967; Gold, 1970; Gold, 1972; Inland Waters Branch Report, 1971; Kraus, 1963). Data from these studies, particularly those relating to the temperature-time dependent behaviour of ice, are important to the study described herein. As temperature increases and approaches the freezing point, strength properties of ice, including tensile strength, yield or ultimate strength and Young's modulus, decrease significantly while rate of creep increases. This establishes important boundary conditions for the use of reinforced ice domes. Observed significant creep behaviour of ice, even under relatively low stress levels, suggests the use of some type of reinforcing to control excessive deformations in the structure. Since such reinforcing would obviously enhance the behaviour of ice as a structural material, and plain ice has received a great deal of attention, it may be appropriate to increase our research activity in the field of reinforced ice. Emphasis should be placed on finding suitable and optimal reinforcing materials and defining the interaction between ice and such reinforcing (Coble et al., 1962).

Snow and plain or reinforced ice have been used as structural materials in the past. Eskimos build igloos with compacted snow; wood fibre or straw reinforced ice has been suggested as a suitable construction material; the Russians have built large scale warehouse facilities using plain ice (Voitkovskii et al., 1959); ice bridges reinforced with trees (Michel et al., 1974) or ferry cables (Gold, 1971) and aircraft parking aprons constructed with fiberglass mats (Kingery et al., 1962) represent a few more examples.

Inflatable structures have been proposed as temporary enclosures for the northern environment, however their instability in high winds makes their use rather limited. In searching for a practical means of stiffening such structures, the concept of reinforced ice domes emerged. An inflatable, stabilized by a network of cables which ultimately serve as reinforcing, is sprayed with water, at temperatures below freezing, to produce a reinforced ice dome. After successive applications of water have built up the ice to design thickness, the inflatable may be removed from the inside and reused, as often as desired, in the erection of further domes. Such an erection technique presupposes suitable temperature and wind conditions and the availability of water or, alternatively, sufficient energy and means to thaw snow.

This paper details experimental work performed at The University of Calgary, the purpose of which was to establish the feasibility of the above described erection technique. In particular, four small-scale, spherical-cap ice domes with a door opening were constructed and tested under dead load conditions. In addition, tests were performed on small samples to evaluate the suitability and effectiveness of glass fibre yarn as a reinforcing material.

DETAILS OF EXPERIMENTAL WORK

(a) Ice Domes

In the fabrication of the inflatable which served as formwork for the erection of the 2.35 m (92.5 in) base diameter and .57 m (22.5 in) high domes (Fig. 1), a black, 18 oz "Shelter-rite"coated nylon fabric was used. To approximate the dome surface, spherical triangular segments were cut, and joined by means of a Shelter-rite adhesive.
In order to obtain a true spherical cap, it was found necessary to perform the joining on a similar sized model made of plywood and perspex. A strip of fabric was attached near the base of the dome to simulate a large door opening providing access to the prototype.

In lieu of a temperature controlled laboratory facility, the experiments were conducted on the grounds of The University of Calgary, next to a shed immediately west of the Civil Engineering building. The site was open to weather except for the protection the shed provided on the west side. To prevent vandalism, a wire-mesh cage was constructed around the workspace. This also was used as support for burlap sacking which was installed to block out the sun's rays after melting, due to absorption of heat by the black fabric, had been observed at temperatures as low as -14°C (7°F).

The first inflatable, which was built as a sealed enclosure using a fabric bottom and which, as a result of the joining technique, had some small deviations from a true spherical surface, was erected on smooth frozen ground. Because the next inflatable was made without the sealed bottom, and in order to provide a good seal around the base perimeter, a plywood base was used for Dome No. 2, whereas Domes No. 3 and 4 were erected on a sheet of polythene. The domes were inflated by means of a portable air compressor to a pressure slightly above the complete unfolding stage in order to remove all irregularities.

The first dome, which served only as a pilot test, was sprayed without any reinforcing, using a compressed air garden spray of 5.5 litre (1.25 gallon) capacity. Rate of application was primarily controlled by the setting of the adjustable nozzle. When freezing occurred in the brass tube connecting the nozzle to the trigger, it was found necessary to insulate this pipe with styrofoam.

Dome No. 2 was reinforced with a complete network of spun fibreglass yarn, having 24
equally spaced meridianal cables and longitudinal cables at 100 mm (4 in) intervals. The yarn, obtained from Fibreglass Canada, was ECG 150 4/8 with a nominal diameter of .9 mm (.035 in) and an average breaking strength of 570N (128 lbf). Perspex seats elevated the cable network away from the inner ice surface. No reinforcing was used in Dome No. 3 to obtain a comparison between the behaviour of plain and reinforced ice domes. As a result of data obtained from this dome, Dome No. 4 was constructed with reinforcing only around the door opening. Construction details of the last three domes are shown in Table 1.

Axisymmetric distributed dead load was applied to these three domes placing 58.5N (13.2 lbf) sand bags on a circular area concentric with the apex. The total load on Dome No. 2 was applied in two stages (Table II). For each dome, vertical displacements of selected points relative to a chosen benchmark were observed using a Wild automatic level. To obtain reproducibility in the readings it was found necessary to attach thumbtacks to the ice surface with points facing outwards on which the readings were taken.

(b) Reinforced Ice Samples

Tests on reinforced ice samples, cast in precooled metal molds and frozen in a 12 cft deep freeze at -15°C, were carried out to determine the tensile, shear and bond strengths of glass fibre yarn reinforced ice. The tension specimens (Fig. 2) measured 310 mm long by 26 mm thick with 100 mm of 26 mm x 11 mm reduced section. Steel end pieces frozen into the end blocks were used for load application. The extension of the specimens, observed on the relative movement of two steel pins frozen into the end blocks at a gauge length of 165 mm, was monitored by two, high sensitivity transducers (L.V.D.T.'s). Shear tests were performed on 25 mm x 25 mm cross section specimens in a shear box apparatus, such as is used for soil testing. To determine the bond between the yarn and the ice, pull out tests were conducted on 25mm x 25mm cross section specimens of varying lengths.

All tests were short load duration experiments (Gold, 1967), with a period of load application from start to failure varying from 3 to 5 seconds for the tension and shear tests and from 4 to 15 seconds for the bond tests. Load application in the cases of the tension and bond tests was affected by means of a Hougfield Tensometer. Since all tests were performed outside the deep freeze in air at 21°C (70°F), the tension and bond specimens were placed inside a perspex box precooled to -15°C in order to minimize temperature rise of the ice.

DISCUSSION OF RESULTS

(a) Ice Domes

Dome No. 1 helped identify a number of construction and erection problems. When water was applied to cold ice (e.g. -15°C), large temperature cracks propagated through the unreinforced dome. After the ice warmed up due to the presence of freezing water, this cracking no longer occurred and existing cracks tended to seal. However, upon cooling, the same cracks reopened and extended when water was applied.

As a result of the reinforcing network in Dome No. 2, the effects of temperature cracking were observed to be less severe than in Dome No. 1. The loading applied to the dome (Table II) produced displacements at various points on the dome as indicated in Figures 3 and 4. Temperature data during the tests, as recorded by The University of Calgary Weather Station, are also given on these figures. Due to the warm chinook wind conditions during the last 49 hours, when a maximum of 0°C was recorded, the rate of creep and hence the rate of deformation sharply increased. Also, immediately before collapse, the deformations, although not measured, were observed to be large and
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<td>3/2/75</td>
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<td>2</td>
<td>full network</td>
<td>plywood</td>
<td>1000</td>
<td>1700</td>
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<td>54 l (12 gals)</td>
<td>-24.5, -23.5, -23.0, -21.5</td>
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<td></td>
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<td>1100</td>
<td>1600</td>
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<td>-19.5, -20.0</td>
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<td>1600</td>
<td>1800</td>
<td>1</td>
<td>18 l (4 gals)</td>
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<td>7/3/75</td>
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<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>0300</td>
<td>0600</td>
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<td>59 l (13 gals)</td>
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<td>4</td>
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<td>polythene</td>
<td>1600</td>
<td>1700</td>
<td>0.5</td>
<td>9 l (2 gals)</td>
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<td>0600</td>
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<td>10/3/75</td>
<td>10/3/75</td>
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<td>Total - 49 l</td>
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TABLE II. DOME TESTING DETAILS

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<tr>
<th>Dome No.</th>
<th>Load</th>
<th>Time of:—</th>
<th>Total Time</th>
<th>Air Temp. at Failure</th>
<th>Average Ice Thickness</th>
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<td></td>
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<td>Placement</td>
<td>Failure</td>
<td></td>
<td></td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.4 KN (320 lbf)</td>
<td>1600</td>
<td>72</td>
<td>-</td>
<td>-</td>
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<tr>
<td></td>
<td>2.4 KN (540 lbf)</td>
<td>1400</td>
<td>190</td>
<td>Sudden</td>
<td>-5.0</td>
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<tr>
<td>3</td>
<td>1.7 KN (380 lbf)</td>
<td>1300</td>
<td>0.2</td>
<td>Sudden during loading</td>
<td>-12.0</td>
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<tr>
<td>4</td>
<td>2.4 KN (540 lbf)</td>
<td>1500</td>
<td>29</td>
<td>Cracked but supported load for 1/2 hour more</td>
<td>+6.0</td>
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</tbody>
</table>
Figure 2

TEST DETAILS OF DOME NO. 2

TEST DETAILS OF DOME NO. 4
POINT DISPLACEMENTS FOR DOME NO. 2

Figure 3
POINT DISPLACEMENTS FOR DOME NO. 2

Figure 4a
POINT DISPLACEMENTS FOR DOME NO. 2

Figure 4b
indicated the presence of large bending moments particularly in the vicinity of the door. Sudden and complete failure of the dome occurred when, as a result of fracture emanating from the highly stressed door region, the loaded central portion collapsed vertically. From an examination of Figures 3 and 4, it appears that significant deformations in the dome occurred only above air and hence ice temperatures of -5° C.

Subsequent domes were erected on a sheet of polythene since walking on the plywood base of this structure caused minor distress and audible cracking in the dome.

Dome No. 3, plain ice, collapsed suddenly and unexpectedly, during loading, at a load of 1.7 KN (380 lbf). It demonstrated, however, that an unreinforced dome can be built up by spraying water, provided that the formation of major temperature cracks is avoided by proper sequencing of erection. It also indicated that the weak part of the structure, as expected, is the area where the door is located and that there is a certain inherent material instability in such a structure in as much as a crack, once initiated and with nothing to prevent its further propagation, will extend and cause collapse.

The overall load-deformation behaviour of Dome No. 4, as can be seen from Figures 5 and 6, was very similar to that of Dome No. 2. A contour plot of the deflected shape immediately prior to the first sign of failure is given in Figure 7. Collapse was initiated by two meridional tension cracks forming and opening up in the vicinity of the door in places where temperature cracks had formed during spraying. Due to reinforcing present in this region, this crack (Fig. 8) was semi-stable and the structure held the load for a further half hour before collapsing (Fig. 9).

The average thicknesses of the last three domes are shown in Table 2. The ice of Domes No. 2 and 3 appeared white and granular with a distinct layered structure visible. However, the ice in the last dome was clear with very little layering.

(b) Reinforced Ice Samples

The glass fibre yarn, composed of individual continuous fibres spun together, exhibits a surprisingly high degree of flexibility (Fig. 10). In spite of this fact, tests conducted on reinforced ice samples indicate that this yarn is a suitable reinforcing material for ice. The tension tests suggest that the pre-cracking tensile strength is enhanced by approximately 25% irrespective of the amount of reinforcing. Results from these tests are summarized in Figure 11.

Results of the direct shear tests showed that the observed value of approximately 2.5 MPa (360 psi) ultimate shear strength was unaffected by the presence of the reinforcing. Clearly, the yarn adds no dowel action in shear.

The pull-out tests gave the results shown in Figure 12, where the average and the range of data are indicated by means of small circles and the vertical lines. As the length of the specimen is increased, more water is required and freezing times correspondingly increase, allowing better penetration of the water into the yarn. This phenomenon also appeared in tests performed on specimens which were poured in a warm mold, where far greater forces were required to pull similar lengths of yarn from the ice. It may be a reason for the non proportional increase in bond strength with increased embedment length, as indicated by the concavity of the average curve. The minimum slope of this curve, indicating anticipated minimum bond strength, is shown by the straight line $P = 3L$. 

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Stanley and Glockner 11
Figure 5

POINT DISPLACEMENTS FOR DOME NO. 4

Air temp. (°C)

Time (Date)
POINT DISPLACEMENTS FOR DOME NO. 4

Figure 6a
POINT DISPLACEMENTS FOR DOME NO. 4

Figure 6b
POINT DISPLACEMENTS FOR DOME NO. 4

Figure 6c
POINT DISPLACEMENTS FOR DOME NO. 4

Figure 6d
EXAGGERATED PLOT OF 100mm CONTOUR DISPLACEMENTS

Figure 7
Major Crack in Dome No. 4

Figure 8

Dome No. 4 Collapsed

Figure 9
Figure 10

TYPICAL LOAD – STRAIN CURVE FOR YARN
Figure 11

Nominal failure stresses of tension specimens in short load-duration tests
Vertical lines indicate spread of results

AVERAGE PULLOUT FORCES OBTAINED IN SHORT LOAD-DURATION TESTS ON SAMPLES OF VARYING EMBEDMENT LENGTHS

Figure 12
CONCLUSIONS

The construction of the four test domes suggests that spraying water onto an inflatable is a feasible and practical method for erecting temporary enclosures in the north, provided there is a supply of water available. The testing of the domes indicates that such shell structures, as is known, have high strength and may be suitable for providing shelters in regions where air temperatures remain below \(-5^\circ C\) for extended periods of time.

On the basis of these tests and data obtained from reinforced ice samples, one concludes that fibreglass yarn appears to be a suitable reinforcing for ice and that reinforced ice may be an appropriate construction material for the north.

REFERENCES

ICE FORCES ACTING ON INCLINED WEDGES

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Institute of Hydrodynamics and Hydraulic Engineering
Technical University of Denmark
Building 115 • DK-2800 Lyngby • Denmark

ABSTRACT

The force acting on an inclined wedge can be expressed by the formula: \( F = C_F \ F_{\text{max}} \), where \( C_F \) is a reduction factor being a function of the system data, \( F_{\text{max}} = r_o \), \( r_o \) is the maximum force acting on a structure with vertical faces, and \( r_o \) is the crushing strength. A simple formula of \( C_F \) has been derived, showing the functional relationship with: \( E \) Young's modulus, \( \rho \) density of the ice, \( u_c \) velocity of ice sheet, \( \mu \) friction coefficient between ice and structure, \( \varepsilon = r_y/r_o \) ratio, and finally the shape of the wedge expressed by the included angle at the point of the wedge in horizontal plane, and by the inclination of the wedge to horizontal.
A theory, by which the ice force acting on inclined wedges can be determined, has been developed. The maximum force on a given ice section in front of the structure with vertical faces is

\[ F_{\text{max}} = r_c e d \]

producing a crushing or shear failure. \( r_c \) is the compression strength (in kN/m²), \( e \) is the thickness of the ice (in m), and \( d \) is the width of the structure (in m).

The actual force on the inclined wedge is expressed as

\[ F = C_F F_{\text{max}} \]

producing a bending failure as parallelogramic pieces break off. \( C_F \) is a function of the system data (dimensionless)

\[ C_F = \phi \left( \frac{E}{\rho u_c^2}, \frac{e}{d}, \frac{v}{e}, \frac{r_b}{r_c}, \alpha, \beta, \mu \right) \]

where \( E \) is Young's modulus (in kN/m²), \( \rho \) is the density of ice (in t/m³), \( u_c \) is the velocity of the floe (in m/sec), \( v \) is the characteristic breaking off distance (in meters), \( r_b \) is the flexural strength (in kN/m²), \( 2 \alpha \) is the included angle at the point of wedge in horizontal plane, \( \beta \) is the inclination of wedge to horizontal, and \( \mu \) is the coefficient of friction.

**PROPOSED FORMULA FOR REDUCTION FACTOR \( C_F \)**

The reduction factor \( C_F \) can be expressed by the formula (developed for engineering application)

\[ C_F = \frac{2.1 \sqrt{\varepsilon} \sqrt{C}} {\sqrt{C}} \]

where \( \varepsilon = \frac{r_b}{r_c} \) and \( C = 0.16 \sqrt{\frac{E}{\rho u_c^2 \sin^2 \alpha}} \frac{C_1}{C_2} (C_3)^2 \)

The parameters \( C_1, C_2, \) and \( C_3 \) being
The velocity $u_c > 0$.

For practical application the system constants may vary within the intervals

$$0.1 \leq u_c \leq 4.0 \text{ (m/sec)} \quad 0.1 \leq \frac{C_1}{C_2} \leq 0.9 \quad 1.0 \leq C_3 \leq 4.0 \quad 0.2 \leq \varepsilon \leq 0.5$$

$\beta$ should not exceed $70^\circ$.

The higher the velocity, the greater $C_F$ becomes. The greater the value of $E$, the smaller $C_F$ becomes, but since it is the sixth root, $C_F$ is not substantially influenced by variation of $E$. It appears, $C_F$ becomes larger for increased values of the friction coefficient, for larger values of $\beta$, and for decreasing values of $\alpha$.

For $e/d = 0$, i.e. $d = \infty$ for a very wide wedge, $C_F$ will be larger than for values of $e/d = 0.3$, which is the limiting value of $e/d$ for the theory applied.

**NUMERICAL EXAMPLE**

Example 1: $E = 2.1 \cdot 10^5 \text{ kN/m}^2 \quad \frac{T_b}{V_c} = 0.25 \quad \rho = 0.9 \text{ t/m}^3 \quad \mu = 0.2$

$$\beta = 60^\circ \quad \alpha = 50^\circ \quad \frac{e}{d} = 0.1 \quad u_c = 1.3 \text{ m/sec}$$

$$C_1 = 1 - 0.45 = 0.55 \quad C_2 = 0.2 + 2.26 = 2.46$$

$$C_3 = 6 \cdot 0.1 \cdot 0.64 + 6 \cdot 0.22 = 1.71$$

$$C = 0.16 \sqrt{\frac{2.1 \cdot 10^5}{0.9 \cdot (1.3^2 \cdot 0.766^2)}} \cdot 0.22 \cdot 1.71^\frac{2}{3} = 49.9$$

$$C_F = \frac{2.1 \cdot 0.63}{\sqrt{49.2}} = 0.36$$

**INTERMITTENT NATURE OF FORCE**

The forces acting are of an intermittent nature since they increase abruptly each time a parallelogramic piece is about to break off, after which the forces decrease until the next encounter takes place. The period of time between each peak force can be expressed by

$$T_c = \frac{\frac{\gamma}{u_c \sin \alpha}}{\frac{\gamma}{u_c \sin \alpha}} = \frac{\gamma}{u_c \sin \alpha} \frac{e}{u_c \sin \alpha} = 1.5 \sqrt{\frac{\gamma}{u_c \sin \alpha}}$$

corresponding to a frequency $n_c = 1/T_c$. Each peak of force taking place within $1/50$ sec.

This should be considered in the design of the structures as resonance response may prove fatal.

Example 2: (see Example 1)

$C = 49.9 \quad e = 0.3 \text{ m} \quad u_c = 1.3 \text{ m/sec} \quad \alpha = 50^\circ$
giving

\[ T_c = 1.5 \sqrt{\frac{9.9}{1.3 \sin 50^\circ}} = 1.7 \text{ sec} \]

\[ n_c \approx 0.6 \]

CLOSING REMARKS

The formulae given above are based on a theoretical investigation verified by model tests with artificial and natural ice.

Since the investigation is not completed, revisions may be introduced in the formulae given in this paper.

REFERENCES


ICE FORCES ACTING ON SLENDER STRUCTURES

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Building 115 · DK-2800 Lyngby · Denmark

ABSTRACT

The effect on the indentation strength of the ratio $h/d$, where $h$ is the thickness of the ice sheet and $d$ is the width of the slender structure, has been evaluated by the author, and a formula has been derived. A comparison with various theories has been made. The indentation force can be expressed: $F = k \sigma_\infty h \frac{d}{h}$, where $k = 1 + 2.1(0.4 + d/h)^{-1}$, and $\sigma_\infty$ is the indentation strength for $d/h \to \infty$. 
INTRODUCTION

It has long been known that the nominal strength increases with the slenderness of the structure (pile). Thus for a pile with a cross-dimension $d$ equal the ice thickness $h$, the nominal strength $\sigma$ is approx. $2 - 2.5$ times the strength $\sigma_\infty$ to be found for $d = \infty$.

This has particularly been studied by Assur (1972), Schwarz (1974), Allen (1970), and Afanas'ev (1973).

PROPOSED FORMULAE

The author proposes to use the formulae

$$ k = \frac{\sigma}{\sigma_\infty} = 1 + 1.5 \frac{h}{d} $$

$$ F = k \sigma_\infty h d $$

where $h$ is the thickness of the ice, $d$ the width of the structure, and $F$ the total force. A more refined formula giving a boundary condition of $5.20$ for $d/h = 0.1$, i.e. slightly higher than proposed by Assur (1972), is

$$ k = 1 + 2.1 \left(0.4 + \frac{d}{h}\right)^{-1} $$

It is shown that these simple formulae are in good agreement with existing theories.

COMPARISON WITH OTHER THEORIES

In Figure 1 the results of the different investigations have been presented. Assur (1972) has pointed out the effect of internal friction and the problem of buckling. Also the phenomenon of horizontal splitting appears to indicate a limiting boundary of $\sigma/\sigma_\infty$, which can be explained as follows:

Before splitting we have

$$ \frac{\sigma}{\sigma_\infty} = 1 + 1.5 \frac{h}{d} $$

which for $d/h = 1.0$ gives $\sigma/\sigma_\infty = 2.5$.

After splitting we have (assuming symmetrical)

$$ \frac{\sigma}{\sigma_\infty} = 1 + 1.5 \frac{h/2}{d} = 1.75 $$

i.e. a lower value.

This may prove to explain the lowering of curves for $0 \leq d/h \leq 1$.

Schwarz (1974) has shown that the strength is also a function of the ice thickness. This effect is not included in this investigation.

REFERENCES


Figure 1. Indentation strength as a function of d/h
SECTION 10
SEABED AND SUBBOTTOM

Bafus, G. R., G. L. Guyman and R. F. Carlson
Some Results of a Thermal Analysis of Offshore Artificial Islands
(Puget Sound Naval Shipyard, Washington, University of California, and
University of Alaska, United States)

Brewer, M. C.
The Seaward Extension of Permafrost Off the Northern Alaska Coast
(Juneau, Alaska, United States)

Bryant, W. R.
Determination of the Physical Characteristics of Marine Sediments
Related to Offshore Construction VIA Seismic Methods
(Texas A & M University, United States)

Carstens, T.
Seabed scour by currents near platforms
(The Norwegian Institute of Technology, Trondheim, Norway)

Gerritsen, F.
Coastal Erosion Processes with Special Reference to the Function of Groins
and Headlands in Coastal Protection
(University of Hawaii, United States)

Hakkila, J. O. and Balch, J. C.
Permafrost: From the Bottom Up
(Earth Freezing-Warming Systems Incorporated, Alaska, United States)

Harrison, W. D. and T. E. Osterkamp
Theoretical Models for Sub-Sea Permafrost
(University of Alaska, United States)

Hunter, J. A. and A. S. Judge
Geophysical Investigations of Sub-Sea Permafrost in the Canadian
Beaufort Sea
(Department of Energy, Mines and Resources, Ontario, Canada)

Lai, N. W. and R. F. Dominguez
Determination of Wave-Induced Pressures in the Soil Media Contiguous
to a Buried Pipeline
(Texas A & M University, United States)

Rogers, J. C., L. H. Shapiro, L. D. Gedney and D. Van Wormer
Near Shore Permafrost in the Vicinity of Pt. Barrow, Alaska
(University of Alaska, United States)
 SOME RESULTS OF A THERMAL ANALYSIS OF OFFSHORE ARTIFICIAL ISLANDS

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ABSTRACT

A numerical model is formulated for analyzing artificial offshore islands by obtaining an equivalent variational solution to the transient heat conduction equation with isothermal phase change. Trial simulations indicate a successful comparison of numerical solutions with exact solutions to freezing front propagation and in situ measurements of soil temperatures at Fairbanks, Alaska. A final simulation is applied to a hypothetical artificial island subjected to typical arctic coastal meteorological conditions. The study situation includes a 25 foot (7.6 m) high, 24 foot (7.4 m) radius cylinder immersed in 20 feet (6.1 m) of seawater. Simulation results indicate with natural freezing conditions and negligible snow cover, an equilibrium frost penetration depth of 27 to 48 feet (8.2 to 14.6 m) will occur and summer thaw depth will vary from 3 to 4 feet (1 to 1.2 m) depending on the nature of embankment material used. It is concluded that artificial islands are thermally feasible in nearshore arctic regions.
INTRODUCTION

Recent studies indicate that offshore regions in the Beaufort and Chukchi Seas may contain extensive oil and gas reserves. Exploratory offshore drilling in the MacKenzie Delta has already commenced, and it can be expected that the petroleum needs of the United States will stimulate exploratory offshore drilling in Alaskan waters. Assuming an adequate knowledge of critical design loading due to sea ice, conventional offshore drilling structures could probably be constructed but at a prohibitive cost. A feasible alternative to existing technology is the use of man-made islands as exploratory platforms (Riley, 1974). This paper describes the results obtained from a theoretical thermal analysis of man-made artificial islands as offshore drill sites. A mathematical model is formulated for computer solution utilizing the finite element method. Numerical solutions are compared with analytical solutions prior to application to a hypothetical offshore island.

FINITE ELEMENT MODEL

A finite element solution to the transient heat conduction equation is obtained by searching for an equivalent variational functional which upon minimization will converge to an exact solution to the heat conduction equation.

Given the partial differential equation of two-dimensional transient heat conduction in Cartesian coordinates:

$$\frac{\partial}{\partial x} \left( K_x \frac{\partial \phi}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial \phi}{\partial y} \right) + G = \rho c_v \frac{\partial \phi}{\partial t}$$  \hspace{1cm} (1)$$

where $\phi$ = temperature

$K_x$ and $K_y$ are the thermal conductivities in the x and y directions, respectively

$G = G(x,y,t)$, a variable heat source or sink

$\rho c_v$ = volumetric heat capacity

$t$ = time

it can be shown that the solutions $\phi$ which minimize the functional $P$, i.e.,

$$P = \frac{1}{2} \iint \left( K_x \left( \frac{\partial \phi}{\partial x} \right)^2 + K_y \left( \frac{\partial \phi}{\partial y} \right)^2 + 2 \left( \rho c_v \frac{\partial \phi}{\partial t} - G \right) \phi \right) dx dy$$  \hspace{1cm} (2)$$

are also equivalent solutions to the transient heat conduction equation (Zienkiewicz, 1971).

Equation 2 is formally solved by discretizing the solution domain into numerous subregions or "elements" by assuming that the temperature distribution within each element can be approximated by an nth order polynomial. A linear polynomial is selected to approximate the state variable in a two-dimensional finite element mesh composed of arbitrarily shaped triangular elements as shown in Figure 1.

Thermal properties are assumed to be constant within each element, although abrupt changes in properties between elements are permitted.

Substitution of a linear temperature distribution shape function into Equation 2 and minimization of the integrated contributions of all elements produces a system of simultaneous linear first order differential equations of the form
Figure 1. Typical finite element mesh
where

\[
[K] \{\phi\} + [C] \{\frac{\partial \phi}{\partial t}\} = \{G\}. \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (3)
\]

\[\begin{align*}
[K] &= \text{system conduction matrix} \\
[C] &= \text{system capacitance matrix} \\
\{G\} &= \text{system load column matrix} \\
\{\phi\} &= \text{nodal temperature column matrix}
\end{align*}\]

The system conduction and capacitance matrices are banded and symmetric, which significantly reduce computer memory requirements for storing arrays. Solutions in the time domain are obtained by employing the Crank-Nicolson finite difference scheme which utilizes the arithmetic mean of derivative of the state variable at the beginning and end of each time step to move the solution ahead in time.

Simulation of heat conduction and phase transformation in two dimensions involves a minor modification to the variational formulation of transient heat conduction by employing a simple heat balance procedure. Phase change is not permitted to occur at a given node until a requisite amount of latent heat has been exhausted or added. The prevailing quantity of latent heat for each node is computed from one-third the volume of material subject to phase transformation contained in elements common to a specific node such as node "A" in Figure 1.

Once phase change has occurred at a given node, the thermal properties of elements common to that node are updated based on the volumetric proportion of soil-water and soil-ice present in the respective elements.

Comparison of Finite Element Solutions of Phase Change With Analytical Solutions

As the method formulated to solve the latent heat problem is a numerical approximation, the comparison of finite element solutions with exact solutions to simple isothermal phase change problems is discussed. Two problems are considered: one in which thermal parameters remain unaffected by phase change and secondly, one where phase transformation produces a significant variation between frozen and unfrozen thermal properties. As a first example, an exact solution of the solidification of a one-dimensional homogeneous slab of liquid of finite thickness is used for comparison (Allen and Severn, 1952). This same example was used by Zienkiewicz et al (1973); however, the close agreement shown in Figure 2 is somewhat better than that obtained by Zienkiewicz, et al.

Comparison of a one-dimensional finite element simulation with a Neumann solution for freezing front propagation is shown in Figure 3. The Neumann solution was extracted from an example cited by Jumikis (1966). Typical soil thermal properties were used. The freezing front locus is plotted versus the square root of time to demonstrate the stability of the finite element solution, substantiate the isothermal phase change assumption and enable computation of a numerical constant of proportionality. The theoretical constant of proportionality is \(0.575 \text{ ft} \cdot \text{day}^{-1/2}\) \((17.53 \text{ cm} \cdot \text{day}^{-1/2})\) and the finite element solution constant of proportionality is \(0.561 \text{ ft} \cdot \text{day}^{-1/2}\) \((17.10 \text{ cm} \cdot \text{day}^{-1/2})\). As can be seen from Figure 3, the constant of proportionality produces the appropriate solution (finite element or analytical) for freezing front propagation when multiplied by the square root of elapsed time.
Figure 2. Solidification of a slab of liquid: Progress of freezing through slab. Crank-Nicolson Method used for the Time Domain Solution.
Figure 3. Comparison of a variational solution and an analytical solution to the Neumann problem.
SIMULATING AIR-GROUND INTERFACE ENERGY EXCHANGE

The thermal processes occurring at the interface between a soil system and the atmosphere are often too complex for an exact description; yet, their quantification is crucial to conducting an analysis of any thermal regime. From a macroscopic viewpoint, an energy balance at the air-ground interface can be approximated by expression

\[ Q_{\text{soil}} + Q_{\text{swr}} + Q_{\text{nlwr}} + Q_{\text{conv}} - Q_{\text{evap}} = 0 \] ............... (4)

where

- \( Q_{\text{soil}} \) = heat flux into the soil column
- \( Q_{\text{swr}} \) = absorbed shortwave radiation
- \( Q_{\text{nlwr}} \) = net long wave radiation exchange
- \( Q_{\text{conv}} \) = convective heat transfer
- \( Q_{\text{evap}} \) = evaporative heat loss

Once each of the individual heat flux components has been determined from previously computed boundary surface temperatures and existing meteorological parameters, net energy exchange across a two-dimensional surface is simulated by coupling one-dimensional elements (having a thermal conductivity equal to a turbulent heat transfer coefficient and a volumetric heat capacity of zero) to the two-dimensional elements representing the boundary surface. An equivalent temperature is calculated from the relationship

\[ \phi_{eq} = \phi_{\text{amb}} + \frac{Q_{\text{swr}} + Q_{\text{nlwr}} - Q_{\text{evap}}}{K} \] ............... (5)

where

- \( \phi_{eq} \) = equivalent temperature
- \( \phi_{\text{amb}} \) = ambient air temperature
- \( K \) = turbulent heat transfer coefficient

and other terms remain as previously defined. The computed temperature from Equation 5 is then imposed at the end of each one-dimensional element as shown in Figure 4.

The theoretical and experimental contributions of Scott (1964, 1969) are used to compute a turbulent heat transfer coefficient that is allowed to vary at the end of each time increment or when meteorological parameters change.

Simulation of Surface Energy Exchange Under Fairbanks Field Conditions

Numerical simulations of surface energy exchange were conducted for Fairbanks, Alaska summer and winter climatic conditions to permit evaluation of several solution techniques. Numerical instabilities encountered in modeling winter snow cover effects led to the development of the equivalent temperature approach for handling thermal processes at the air-surface interface.
\[ \phi_{eq} = \text{Equivalent Temperature} \]

\[ \rho_c K = \text{One-Dimensional Surface Energy Exchange Element} \]

\[ K = \text{Turbulent Heat Transfer Coefficient} \]

\[ \rho_c = \text{Volumetric Heat Capacity} = 0 \]

Figure 4. Modeling surface energy exchanges
Using local meteorological data for Fairbanks, summer surface temperatures of exposed silt were simulated over a ten day period. Silt thermal properties were computed from the experimental data of Kersten (1949). A clear day outgoing evaporative heat flux was determined from Fairbanks evapotranspiration loss data (Patrick and Black, 1968) and further modified by a factor equal to the ratio of actual solar energy received to theoretical available clear day solar energy. Input data and results are shown in Table 1. Computed ground surface temperatures were approximately 8°F (4.4°C) warmer than the ambient air temperature.

A ten day winter simulation of heat conduction in a soil column with an 18 inch (45.7 cm) snow cover was conducted to allow comparison of measured and computed temperature profiles at the end of the 10 day period. Of secondary interest were the computed snow surface temperatures. Average Fairbanks January snow density and thermal properties were determined from values presented by Wheeler (1973). In this simulation, solar and long wave radiation fluxes were placed directly across the snow surface without employing an equivalent temperature and turbulent heat transfer coefficient. Solution instabilities caused by day to day fluctuations in meteorological parameters were encountered when all surface energy balance terms were simulated in such a manner. Available options to solve the stability problem were those of utilizing very small simulation time increments or applying the equivalent temperature concept. The latter was chosen.

Results of the winter simulation are shown in Table 2. Ten elements were used to simulate the snow cover and six elements to model a 45 inch (114 cm) soil column. A lower soil boundary temperature of 32°F (0°C) was imposed since field observations indicated a mean January position of the groundwater table at a depth of 45 inches (114 cm) below the ground surface. Calculated and measured results were in good agreement.

TWO-DIMENSIONAL ARTIFICIAL ISLAND THERMAL ANALYSIS

A theoretical thermal analysis was conducted to determine the thermal feasibility of man-made permafrost islands. Two different problems were considered:

1. The thermal degradation of an island drill site built on location during the winter months and completely frozen prior to spring breakup.

2. The propagation of a winter freeze front through an island constructed during the summer from dredged seafloor sediments.

Simulated island geometry was a right cylinder with a vertical height of 25 feet (7.6 meters) radius of 24 feet (7.3 meters) immersed in 20 feet (6.1 meters) of water. Two-dimensional finite element discretization of a vertical cross section of the island is shown in Figure 5. Symmetry in the transient heat conduction solution was assumed thus only the left half of the island is shown.

The upper 4 feet (1.2 meters) of the island consisted of saturated gravel with a porosity of 45 percent and the remaining 21 feet (6.4 meters) were saturated silt at a porosity of 50 percent. Soil properties were determined from Kersten (1949).
### Table I
**Summer Simulation (1-10 July 1972)**

#### I. Meteorological Data

<table>
<thead>
<tr>
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#### II. Simulation Results

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*All Energy Terms are Expressed in BTU/FT²-Day*
# Table 2
## Winter Simulation (20-29 January 1973)

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### II. Simulation Results

<table>
<thead>
<tr>
<th>Day</th>
<th>Simulated Snow Surface Temp. (°F)</th>
<th>Simulated Net Long Wave Radiation Exchange</th>
<th>Simulated Ground Surface Temp. (°F)</th>
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<td>5.</td>
<td>104.3</td>
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<tr>
<td>10</td>
<td>11.</td>
<td>73.3</td>
<td>20.6</td>
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### III. Computed and Measured Soil Column Temperatures at the End of the 10 Day Winter Simulation

<table>
<thead>
<tr>
<th>Soil Column Depth (Inches)</th>
<th>Simulated Soil Temp. (°F)</th>
<th>Measured Soil Temp. (°F)</th>
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</tr>
<tr>
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<tr>
<td>45.</td>
<td>32.0</td>
<td>32.0</td>
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*All Energy Terms are Expressed in BTU/FT²-Day*
Figure 5. Two-dimensional finite element mesh
96 elements
61 nodes
Historical daily meteorological data from Barter Island, Alaska, was utilized to simulate surface boundary temperatures. Barter Island is 2 miles (3 Km) from the north shore of the Alaskan mainland. The climate is determined by the surrounding ocean environment which prevents extremely low winter temperatures and limits the warming effects of continuous daylight during the summer months.

First Thermal Analysis: Prefrozen Drill Site

The first thermal analysis assumed a prefrozen island built during the winter at an initial uniform temperature equal to an average ambient air temperature of 15°F (-9.4°C). Thermal simulations for the period 1 May 1970 through 31 August 1971 were conducted. Snow cover was assumed to be negligible. A mean annual seawater temperature of 28.5°F (-1.9°C) was applied along the completely submerged vertical sides of the island. Simulation results are summarized in Figures 6 and 7. These figures show computed isotherms on May and August of 1971. Phase change was assumed to occur at 32°F (0°C). Approximately the upper 3 feet (1 meter) of the island thawed at the end of each summer.

Second Thermal Analysis: Dredged, Unfrozen Drill Site

A second two-dimensional thermal analysis was undertaken to determine winter freezing front propagation in a dredged artificial island with the same geometry and soil structure as the previously discussed case. Salinity of the seawater in the saturated silt and gravel was assumed to be 15 parts per thousand which depressed the freezing point to 30.4°F (-0.9°C). It was recognized that the imposition of a mean annual sea temperature of 28.5°F (-1.9°C) would not be completely representative of winter boundary conditions imposed along the vertical sides of the island. For this reason, the growth of sea ice was simulated and the estimated temperature distribution in the ice was imposed along the upper submerged portion of the island considered to be in contact with the ice.

Simulation results are shown in Figure 8. Approximately the first upper 13 feet (4 meters) of the island froze during the first winter. To assist in predicting the elapsed time for the entire island to freeze, a one dimensional simulation incorporating geothermal heat flux was conducted. It was estimated that the island would be completely frozen after 5.5 years. Extrapolation of results indicated an equilibrium frost penetration depth of 27 feet (8.2 meters) in a saturated silt soil structure with a 4 foot (1.2 meter) saturated gravel overburden (Figure 9). A second simulation using saturated gravel predicted that a gravel island of the same dimensions would be completely frozen in 3.5 years. Computed equilibrium freeze front penetration in gravel subjected to Barter Island climatological conditions was 48 feet (14.6 meters) after an elapsed time of 18 years (Figure 10).

CONCLUSIONS

Based on the foregoing thermal simulation results, the following conclusions are made subject to Barter Island, Alaska, climatological conditions:

1. Under natural freezing conditions with negligible snow cover, equilibrium frost penetration depth for saturated silt with a 4 foot (1.2 meter) gravel overburden is approximately 27 feet (8.2 meters) and for saturated gravel, 48 feet (14.6 meters).

2. Summer thaw of saturated frozen gravel at a porosity of 45 percent can be expected to be 3 feet (1 meter).
Figure 6. Computed isotherms (°F) on 1 May 1971
Simulation Time Span: 1 May 1970 - 31 August 1971
Winter snow cover assumed to be negligible
Figure 7. Computed isotherms (°F) on 31 August 1971
Simulation time span: 1 May 1970 - 31 August 1971
Winter snow cover assumed to be negligible
Figure 8. Two-dimensional simulation of the freezing of a saturated gravel and silt island
Computed isotherms (°F) on 15 May 1971
Simulation time span: 1 November 1970 - 15 May 1971
Figure 9. Simulated elapsed time to reach an equilibrium freeze front in a saturated gravel/silt soil structure.
Figure 10. Simulated freeze front propagation in saturated gravel
3. Dredged island sites are thermally feasible in shallow water depths.

REFERENCES


The thickness of permafrost decreases rapidly, with corresponding increases in temperature, as the edge of the ocean is approached in northern Alaska. Most of the change occurs within two or three hundred meters of the coast, in the case of a stable shoreline and nearshore ocean depths exceeding two meters. This thinning and warming of the permafrost continues rapidly for the first few hundred meters offshore.

A thin layer of permafrost, by definition but probably existing in an unfrozen state if the sediments contain brine, extends seaward until the water depths exceed 150 to 300 meters. The thickness of the offshore permafrost is not expected to exceed 30 to 50 meters. The average annual bottom water temperatures are limiting on the minimum temperatures to be found in this subsea permafrost. In no event will these permafrost temperatures be lower than -1.8°C; seldom will they be below -1.1°C. Where the shoreline is retreating, the offshore thickness of permafrost will be greater and the temperatures of the permafrost colder than in the case of a stable shoreline.

The temperature, at a depth of 20 meters, beneath vegetated and stable islands off the northern Alaskan coast, should approximate -9.0 to -9.5°C; beneath unvegetated but stable islands and exposed offshore bars of considerable width, it should approximate -7.0 to -7.5°C. Beneath recently formed bars, the temperatures to be expected at a similar depth may vary from approximately -0.7 to -7.0°C. At the edge of a stable coastline, in northern Alaska, the temperatures to be expected at a depth of 20 meters are between -4.3 and -4.8°C.

At 20 meters permafrost temperatures beneath shallow lagoons, on the order of one meter in depth, are colder than the temperatures found at a similar depth beneath unvegetated barrier islands and mud flats. They more closely approximate the temperatures found beneath tundra covered areas. However, the phase lag in the annual temperature cycle, at the various depths, is delayed to a greater degree than in tundra covered areas because of the extended zero curtain in shallow lagoons.

Ice wedge polygons, although not as strongly developed as on land, are expected beneath these lagoons. In the case of a retreating shoreline, the ice wedge polygons should remain little changed from the way they developed prior to the encroachment of the sea. In either case, artificial islands of gravel, established for the placement of engineering facilities, could be expected to become bonded to the subbottom permafrost in these shallow lagoons.
The discharge from rivers, if sufficiently large, could be expected to locally result in slightly warmer subsea permafrost temperatures, depending on the annual heat input of this runoff water to the bottom waters. However, it is suggested that the major geothermal impact from the river discharges is in the earlier melting of the sea ice locally and the tendency to keep the immediate area free of floating ice during the summer months.

Whereas offshore pingo and eroded relicts of pingos have been reported on the eastern portion of the Beaufort Sea shelf, none are anticipated to exist in the western portion of the Beaufort Sea, or on that portion of the Chukchi Sea shelf off the northern Alaska coast.

ABSTRACT ONLY AVAILABLE
DETERMINATION OF THE PHYSICAL CHARACTERISTICS OF MARINE SEDIMENTS RELATED TO OFFSHORE CONSTRUCTION VIA SEISMIC METHODS

William R. Bryant
Texas A&M University
College Station, Texas
United States

EXTENDED ABSTRACT

The use of offshore high resolution seismic surveys has increased by an order of magnitude during the last five years. In addition, the number of offshore boreholes drilled for soil and foundation investigations also increased markedly. The data generated by these two methods of investigation in addition to other techniques such as sound velocity density and reflectivity determinations of marine sediments allows us to determine within a certain degree of accuracy the structure, lithology, semi-quantitative gas content and physical properties of marine areas at a relative low cost.

Correlation of high resolution seismic profiles with borehole data has made it possible to identify the following features in routine multi-sensor engineering surveys:

1. The extent of shallow gas pockets within sedimentary layers.
2. The delineation of deep gas charged sediments.
4. Influence of gas pressures on the disruption of bedding plains.
5. Delimiting the vertical migration of deeply buried gas to the surface via gas plumes and vents.
6. Association of migrating gas with fault planes and diapiric structures.
7. Determine the degree of migration of gas fronts within marine sediment layers.
8. Identification of deep and surficial regional sand layers, desiccated clay horizons and shell pavements.
9. Identification of large changes in physical properties within certain sedimentary sections.

Velocity, density and reflectivity measurements can within certain restrictions provide means of determining grain size, wet density, porosity and dynamic compressibility of marine sediments. The design of adequate foundations for offshore structures and underwater installations requires the determination of the bearing capacity of the sea floor.
Empirical relationships between porosity, coefficients of compressibility and seismic velocities have been established.

These relationships may be used to estimate the anticipated ultimate settlement of sea-floor structures within an accuracy of approximately 50%.
SEABED SCOUR BY CURRENTS NEAR PLATFORMS

Torkild Carstens

The River and Harbour Laboratory
The Norwegian Institute of Technology
Trondheim
Norway

ABSTRACT

The gravity platforms in the North Sea, sitting directly on granular seabeds, may be under­
out by scour and need scour protection. Conventional methods of protection are developed
for structures 1 or 2 orders of magnitude smaller and seem unnecessarily conservative when
scaled to these larger sizes. The reason is that some of the flow details that exert extra
stresses on the bed, are not preserved during scale-up.

Some new ideas for scour protection are discussed. These are all built into the structure
so as to reduce expensive offshore operations as much as possible. A breakthrough in pro­
tection technique is anticipated before long as a result of the present effort in several
countries to come up with satisfactory designs.
INTRODUCTION

The research reported here was initiated by the appearance of a new type of structures in the North Sea: The gravity platform made of concrete. This structure had its forerunners in the concrete lighthouses developed in Sweden during the last 35 years (Reinius et al., 1971). Several lines of reasoning converge on the gravity platform as the solution for a certain range of depths and wave conditions. Low construction cost is probably the most powerful argument, but it is also claimed that maintenance costs will be low because concrete corrodes very slowly compared with steel.

An underlying assumption is that the scour problem is relatively unimportant. That is, a reasonable safety must be obtained by a reasonable investment in scour protection. If we were to apply the recommendations of the literature on scour, this is by no means a foregone conclusion.

Consider, for instances, the recent predictions for unidirectional flow (Fig. 1) by Bonasoundas (1974) and by Hjorth (1975) that an area some 35, respectively 14 times the cylinder cross section be protected, and apply that to a platform of 100 m diameter. The cost for that kind of protection is no longer a minor item in the total cost of the platform. Before recommending such a massive protection work, we felt the need for an investigation of the similarity conditions when scaling up 1000 times from typically 100 mm laboratory cylinders to 100 m full scale cylinders.

Our suspicion was triggered by the prediction of scour depth, which by published formulas would fall in a range between 1.4 and 2.4 times the cylinder diameter. We found it hard to believe that a hole 100 m deep or more would open up in the seabed, no matter how wide the obstacle. If this were to happen, there ought to exist deeper natural scour holes in the vicinity of rock outcrops and other large obstacles. An interesting example of the general lack of dramatic scour holes is the echogram taken across the wreck of Lusitania and reproduced in Fig. 2.

Figure 1. Recommended scour protection for piers.
Another clue to limiting factors in scour as the size of the obstacle increases is afforded by the erosion and deposition pattern of snow near buildings and other large obstacles. The action of the flow feature referred to in the literature on scour as the horse-shoe vortex is readily seen: It is the characteristic trench near exposed walls of a house surrounded by snow drifts. The width of this trench is at most a weak function of the length of the house, while it is a stronger function of the height of the house.

THE FLOW FIELD

The Potential Mean Flow

The wellknown two-dimensional potential flow around a cylinder

\[ U_r = U_\infty \left(1 - \frac{a^2}{r^2}\right) \cos \theta \]
\[ U_t = U_\infty \left(1 + \frac{a^2}{r^2}\right) \sin \theta \]

where

- \( U_\infty \) = undisturbed flow velocity
- \( a \) = radius of cylinder
- \( r, \theta \) = polar coordinates

is shown in Figure 3. The velocity is slowed down by the buildup of pressure in an upstream sector which is 30° (i.e., half the central angle) wide at the wall, increasing asymptotically to 45° with increasing radius. For \( \theta = \pm 90° \) the velocity at the wall has a maximum of twice the free stream velocity \( U_\infty \). A doubling of the velocity means a quadrupling of the bed shear stress \( \tau_o \) which is proportional to the velocity squared, \( \tau_o \propto u^2 \). Substantial velocity increases prevail on the sides one radius out from the wall.

The primary flow pattern \( U/U_\infty \) versus \( r/a \) (Fig. 3) is nondimensional and can be scaled to any velocity and any diameter.

It is well to remember that outside of the bottom boundary layer the real flow pattern cannot be much different from potential flow. A wall boundary layer will necessarily envelope the cylinder, but its displacement thickness will be small (of order \( 10^{-2}r \)).
This means that under no circumstance will it be possible to avoid stresses on the bed near the cylinder that are several times the free stream bed shear stress. Unless this latter stress is down to 25% or so of the limiting shear stress for movement of the particular bed material in question, scour protection of some kind will be required.

Figure 3. The two-dimensional potential velocity field.

The Secondary Flow

The no-slip condition at the bed causes a vertical velocity gradient which is felt within a boundary layer of, say, 10 m thickness. The actual thickness depends on the eddy viscosity which in turn depends on the roughness of the seabed. In any case a stagnation pressure is set up at the upstream face of an obstacle. The initial pressure distribution might look something like $P_{st} = 1/2p u^2$ in Figure 4, but such a pressure gradient has never been observed. The observed pressure is almost uniform, but has a slight dip in the middle part of the boundary layer ($P_{obs}$ in Figure 4).
The primary stagnation pressure gradient $P_{st}$ causes a vertical flow which is referred to as secondary flow in the literature. This secondary flow (Fig. 5) in turn causes a secondary stagnation pressure when deflected by the bottom. Since the strength of the secondary flow about equals that of the primary flow, $P_{obs}$ at the bottom approximately equals $P_{st}$ above the boundary layer.

Unlike the two-dimensional potential flow which is scalable with the simple parameter $r/a$, the three-dimensional secondary flow depends not only on $r/a$, but also on $a/\delta$, where $\delta$ is the thickness of the bottom boundary layer, and on $h/\delta$, where $h$ is the water depth.

**BED SHEAR**

Because of the secondary flow bed shear stresses will be magnified many times in the vicinity of the upstream wall of a cylinder. Hjorth (1975) has made an extensive investigation of scour near cylinders, and Figures 6 and 7 are taken from this report. Figure 6 shows the flow pattern obtained by studying details on a plaster of Paris model. Figure 7 shows the relative bed shear stress obtained by hot film technique.
An area of intense shear stress with a maximum $\tau = 12 \tau_0$ was found around $\theta = 45^\circ$ a short distance from the wall.

Several model studies of scour have corroborated this result (Carstens and Sharma, 1975). The scour hole contours are similar to the shear stress isolines of Figure 7.

The area of the bed covered by the secondary flow shows up readily in tests where the bed
is covered by a thin layer of sand (Fig. 8). The bed is wiped clean by the flow away from the cylinder upstream. Downstream the sand is picked up primarily by vertical eddies rolling up along the separation surfaces and acting somewhat like vacuum-cleaners.

![Figure 8. Scour induced by cylinder. After Hjorth (1975).](image)

It is unlikely that the secondary flow observed on these small scale models is preserved for large upscaling. A mathematical description of the flow seen in Figure 6 is not readily forthcoming, however, and we shall have to learn by watching large scale experiments.

**SCOUR PATTERN**

Our experience with snow drifts again has influenced our thoughts on the scaling of scour processes. A characteristic feature of the drift pattern around obstacles of many sizes from tree trunks to buildings is that the horse-shoe vortex has downstream extensions. Between the trailing eddies on either side there is an area of deposition, which does not show up near small diameter cylinders in movable sand models.

We hypothesized that by increasing the diameter of the test cylinder, keeping everything else the same, the scour pattern ought to change. The inverted frustum reported by so many researchers would eventually have to yield to a horse-shoe pattern, with scour around less than entire periphery.

Figure 9 shows the result of such a simple test series. The upper figure shows a nearly symmetric scour hole obtained with a 0.12 m cylinder. The pattern in the middle, with a 45° arch downstream where deposition rather than scour occurred was obtained for a 0.50 m cylinder. The lower figure shows accretion along 110° of the periphery, for a 0.75 m cylinder. As the scour near the cylinder decreases, downstream trails appear. In this case the transport happens to generate dunes, which explains the succession of scour holes instead of clean trenches downwards.

All three tests were run with approximately the same water depths, the same undisturbed mean flow velocity $V/V_c = 0.8$ and the same bed material, polyethylene shavings with a limiting velocity for incipient motion of 0.10 m/s.
Figure 9. Change in scour pattern with diameter of cylinder.
SCOUR DEPTH

Our simple test series also yielded interesting results on scour depth. Looked at separately, the observed scour depth could be considered independent of the diameter, with but little scatter. In fact, Torsethaugen (1975) proposed

\[ \frac{S}{D} = 1.8 \left( \frac{V}{V_{cr}} - 0.54 \right) \frac{h}{D}. \]

When the range of diameters was extended downwards with results from other sources, a strong dependence of scour depth on diameter appeared. The curve in Figure 10 has the expected shape, but we had not anticipated that \( \frac{S}{D} \) should fall off quite so quickly with \( D \).

![Figure 10. Change in scour depth with diameter of cylinder.](image-url)

While we are willing to accept a trend such as revealed in Figure 10, we do not recommend that particular curve for prediction purposes. There is probably a lower limit to the relative scour depth \( \frac{S}{D} \) which is approached asymptotically. This asymptote depends on a number of factors, among which both vertical (depth of flow, boundary layer thickness) and horizontal dimensions (diameter) are important.

SCOUR PROTECTION

Once we had gained an understanding of the fluid mechanics of the scour process, several ideas for scour protection suggested themselves.

First of all, it should be clear from the discussion above that one cannot get away from a substantial increase of the bed shear stress \( \tau_0 \sim \rho u \) near the 90° points. In fact, the best one can hope for is purely horizontal flow, which would give essentially a potential flow pattern around the structure. As we have seen, the maximum velocity is then \( 2 U_\infty \) for a cylinder and the maximum shear stress is \( 4 \tau_\infty \).

To obtain the reduction of \( \tau_{max} \) from \( 12 \tau_\infty \) to \( 4 \tau_\infty \) it is necessary to modify the structure so as to reduce, eliminate or reverse the secondary flow.
Deflector

In hydraulic design a common trick to prevent scour at outlet works, spillways and other threatened areas is to deflect the high-velocity flow away from the bottom. The deflected jet will entrain water from underneath. This process reverses the flow along the bottom, feeding in sufficient material to maintain the bed level and frequently even building it up with deposits.

In the present case a similar mechanism is conceivable where the secondary current has a substantial component away from the wall at the bed. The deflector might be a suitably designed bottom plate, with or without a sill as suggested in Figure 11.

![Figure 11. Deflector.](image)

The deflected secondary outflow will cause a tertiary inflow which is more or less efficient in transporting bottom material.

We hypothesized that there ought to be an optimum height of the deflector. If it is too low, the eddy underneath will be too small, and the jet will reattach too close to the structure and cause scour (Fig. 12a). On the other hand, if the deflector is too high, it will induce a new secondary flow down its own face, below the entrained backflow (Fig. 12c).

![Figure 12. Effect of height of deflector.](image)

A set of tests confirmed this reasoning. Whether the bottom plate had a sill or not seemed to make little difference provided the combined height of plate plus sill was used. Figure 13 shows the experimental scour depth versus deflector height.
A deflector should minimize the area requiring protection, and the results obtained are encouraging.

Shape of Caisson

A commonly used base for a vertical cylinder is to flare it out in a cone. Structurally this gives a strong and stiff tower, while from a construction point of view it is somewhat complicated. The conical base is best suited for structures subjected to heavy loadings such as platforms in deep water or in ice.

For less severe loadings a simpler solution is a cantilevered horizontal foundation plate. Potential flow gives a velocity $u(x,y,o)$ at the base of a cone which is higher than it would be if the cone were not there. This implies a higher shear stress for such an imaginary flow with the cone than without.

However, secondary currents, driven by gradients in stagnation pressure as described above, will subject the bed to additional stresses that may outstrip the primary flow differences.

The amplification of bed shear due to secondary currents near a cone should be less than for a cylinder. The potential flow itself has a vertical component away from the bed on the upstream half of the cone. This will counteract the vertical secondary current which is towards the bed. For a certain angle between the cone surface and the horizontal the opposing vertical flow components will cancel, resulting in a horizontal flow sideways and around the cone.

However, for a given slope of the cone this reasoning is only valid in the plane of symmetry. We cannot eliminate the vertical flow simultaneously around the entire cone. In principle this is feasible if the slope is varied around the periphery, perhaps with a cone tilting downstream. That solution is ruled out for marine structures, because the current is rotary and oscillatory. On the other hand, in unidirectional flow in rivers and canals it is common practice to give bridge piers and similar structures asymmetric shapes.
Realizing the difficulties involved in completely eliminating secondary currents by manipulating the shape of the base, we tried out various ways of living with a limited scour. One such idea was to arrange a cone of hinged plates that were free to fall into the scour hole at their lower ends (Fig. 14).

Figure 14. "Tent" of hinged plates.

Together the plates formed a leaky tent, as it were, around the cylinder, enclosing an area protected against direct flow.

The test setup in a 1 m wide flume left much to be desired, and a number of scale effects must have influenced the results. Nevertheless, the basic flow mechanisms described above were dominating, and the results were reasonable.

For comparison the cantilevered plate was also tested. The results indicate that there was little to be gained from a tent compared with a slab that covered the same area. The scour depth was not reduced by the hinged plates, although they did act as intended, falling into the scour hole and thus shielding the inner slope.

EXISTING SCOUR PROTECTION

The protection used so far is interesting in that it illustrates the two basic strategies available. These strategies are i) to accept the flow pattern and provide protection against the prevailing forces and ii) to modify the flow so as to ease the stresses on the bed near the structure.

Ekofisk

The perforated wall structures of Doris/Jarlan design are of the second category. Both The Ekofisk and the Frigg structure (Fig. 15) have a perforated wall outside the actual platform. Model studies, Marion (1974) demonstrated that this wall had the desired effect of reversing the flow along the bottom as suggested in Figure 15.
The Ekofisk structure included a permeable nylon cloth that was attached to the structure and unrolled on the seabed. This screen was then weighted down with a stone blanket of 8-10 m$^2$ per m of tank periphery with rocks of maximum diameter 10 cm.

In view of the inflow along the bed this latter protection may seem redundant. However, on the sides where the wall is parallel to the flow, such inflow is not assured while we know the potential flow to have its maximum here. Since the flow must be assumed to take on any direction, it is necessary to provide protection against the primary flow. Thus we have to fall back on a basic scour protection against prevailing forces. In this case, however, the amplification of forces due to secondary currents is eliminated or reduced.

So far no scour has been observed. Current measurements and tracer studies with labeled sand indicate only weak currents without any preferred direction (Bratteland, 1975).

**Condeep**

The second gravity platform to be placed on the seabed was the Condeep structure on the Beryl and Frigg fields in the British sector of the North Sea. The base of this structure consists of 19 cylinders, and the periphery is a set of 12 arches of radius 10 m. The circumscribed radius is 50 m (Fig. 16).

Observations near a model to scale 1:200 revealed that the amplification factor for the flow velocity at $\theta_1 = 90^\circ$ was not 2 as for a single cylinder of radius $r_1$, but higher. The explanation appears to be that a superposition takes place at $A$ in Figure 16 of the flow caused by the bundle of cylinders, with the local flow field around the individual cylinder to produce a strong amplification of the flow velocity in a thin layer near the wall.

Protection is in this case by a stone blanket, which has proved difficult to dump in place due to rough weather.
Other proposed protections:

The Norwegian Subtank, which is yet to be built, is proposed with an interesting design. The basic idea is that the scour protection should be operational from the moment the platform is sunk in position. The soundness of this philosophy has been proved lately by the difficulties experienced in placing rocks at the first Condeep.

Figure 17 illustrates the idea, which features a set of hinged plates around the lower circumference. In the space between the cylinder wall and the plates in their "up" position, stones can be stored during towing. As soon as the platform is placed on the seabed at its permanent location, the plates are lowered and the stones spread out.
The British company ICI has proposed an artificial seaweed type solution in which numerous bundles of polyester filaments are suspended under a frame cantilevered from the wall of the platform (Fig. 18). The field of porous and resilient material has accumulated sand in full scale tests and is a very promising idea. The fluid mechanics of this type of protection has not been studied, however.

![Image](image.png)

**Figure 18.** Polyester filaments to be suspended near platform base (ICI).

**CONCLUSIONS**

We have shown that the existing prediction formulas for scour depth at bridge piers are likely to be misleading when applied to large gravity platforms. With increasing diameter the scour at the cylinder wall becomes less extensive both in relative area and relative depth.

Scour will nevertheless occur, and ideas for built-in scour protection are discussed. These ideas have been tested to some extent in small scale hydraulic models, with results ranging from full protection to reduced scour. Full scale performance data are still missing or at best uncertain. Several promising methods are at present in the development phase and need large scale testing before they can be finally evaluated. There is every reason to be optimistic in the sense that one or the other of these methods will provide a technically and economically sound solution to the scour problem.

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REFERENCES

For periodicals -


COASTAL EROSION PROCESSES WITH SPECIAL REFERENCE TO THE FUNCTION
OF GROINS AND HEADLANDS IN COASTAL PROTECTION

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EXTENDED ABSTRACT

The first part of the paper examines the various causes of erosion of sandy beaches. Natural erosion, man-made measures will be given.

Special attention will be given to the use of groins and the function of headlands in a littoral drift environment.

Groins have been used in coastal protection for many years and were often considered the exclusive way in which coastal sections could be stabilized.

In the low countries around the North Sea groins have been used extensively; in the Netherlands, for example, they have been built since the 18th century.

In the examination of the functioning of groins under a variety of conditions, attention will be given to three different regimes:

1. wave dominant regime (uninterrupted coastline),
2. wave and current regime (vicinity of tidal entrance),
3. current dominant regime (wave protected tidal channels).

For the effective functioning of groins the following criteria have to be justified:

1. they will have to stabilize the beach in the areas where they are built,
2. they will have to allow bypassing of sand to nourish the downdrift coast to avoid (or reduce) leeside erosion.

These two requirements induce ambivalent characteristics of the system to be designed. On one hand, it implies that sand transport is halted or reduced. On the other hand, the need for sand bypassing requires that the system is designed accordingly.

The interaction between the groins and the littoral transport depends not only on the characteristics of the groin, but also on the nature of the littoral drift.

In particular, it is important whether the littoral drift is basically in the form of beach drift, such as occurs under predominantly swell conditions or whether a large portion of the sediment is transported by the wave induced littoral current. The possibility of a strong tidal current along the shoreline, such as occurs along the Dutch coastline, adds another complication.
In recent years much progress has been made with the application of hydrodynamic theories to shoreline processes. This has led to much better understanding of the nature of the wave induced littoral current and the generation of rip currents as well as of the associated processes of sediment transport along the shoreline. Numerical techniques are available to make computations and predictions for nearshore conditions.

The insertion of groins into the littoral drift regime, however, is still basically an unsolved problem. Design procedures are still primarily based on semi-empirical rules. The progress that has been made in the dynamics of nearshore processes fails when groins are introduced into the system.

It is true that advanced mathematical models have been developed regarding shoreline behavior induced by groins and this information is certainly helpful in the design procedure. These approaches, however, have concerned themselves with rather hypothetical boundary conditions and need further refinement.

In view of the above, the value of groins as a measure of coastal protection has been dubious and has led to controversial viewpoints. When considering the functioning of groins, the following aspects will be considered:

1. the effect of groins on the littoral regime considering both differences in groin characteristics as well as differences in littoral drift behavior,
2. the behavior characteristics of beaches in the vicinity of a tidal entrance and the functioning of groins in a current dominant regime.

Observations about beach behavior in tropical and subtropical regions have shown that rocky headlands often serve as a natural way of coastal protection. The behavior of the beaches in the vicinity of headlands and offshore barriers is very instructive as an example of nature's way to handle a beach stabilization problem. The situation in the vicinity of headlands can usually be characterized in two ways: whether there is or there is not bypassing of sediment around the headland, and the behavior of the beach is different under each of these conditions.

The question is posed if man may be able to learn from nature in protecting and stabilizing the coastline. It is suggested that nature indeed provides valuable instructive examples in this regard and that in the design of adequate groin systems the lessons from nature may have to be learned better.

It is furthermore suggested that in the light of the above comparisons in many instances low headland type groins, T-groins and offshore breakwaters may have better performance characteristics than the traditional spur-type groin systems.
As the Alyeska pipeline is completed and Alaska becomes a major oil producing province of the world, new techniques will be needed to contend with the unique conditions found in these northern latitudes. One of the techniques used to support the above-ground portions of the Alyeska pipeline is a thermo-tube which freezes the permafrost to temperatures cold enough in the wintertime so that it will not thaw in the summer, thereby stabilizing the soil permanently. This device is similar to one experimented with by J. C. Balch before obtaining a patent on his thermo-tube.

A more recent innovation useful for permafrost applications is the thermo-loop, designed by J. C. Balch, and further experiments having been conducted by the University of Alaska.

It is proposed that such devices may be used in all permafrost stabilizing applications, including the stabilization of coastal areas where new ports will be constructed and needs exist for the building of drilling platforms.

The thermo-loop has the distinct advantage over the thermo-tube of being installed in any configuration.

It is proposed that drilling platforms be erected on offshore ice islands to be anchored to the ocean floor by freezing from the ocean floor upwards.
ABSTRACT

In order to understand the sub-sea permafrost regime off much of Alaska's arctic coast, it is necessary to consider the response of land-formed permafrost to changing temperature and salinity conditions associated with ocean transgression on the land. Sea bed temperatures seem to be negative in much of the Beaufort Sea, but inundated permafrost will still thaw downward from the sea bed if the sea water is above its freezing temperature. The process cannot be understood within the framework of conventional heat transfer models because of the key role played by salt. This is illustrated by a simple coupled heat and salt transfer model, solved in closed form, in which heat and mass are transferred by diffusion. The solution is a generalization of the Stefan solution for growth of an ice cover. It illustrates how the thawing rate depends almost entirely on salt transport properties at a sea bed temperature of -1°C, on thermal properties at +1°C, and on both at intermediate temperatures. The calculated thawing rates are so slow in this diffusion model that the significance of pore liquid motion is suggested. The theory can be easily reformulated to include this or other effects.
INTRODUCTION

There is little doubt that permafrost is common under the northern seas of Alaska, Canada, and the Soviet Union (see MacKay, 1972, and references therein). Although our knowledge about it is still quite limited, several experimental programs have been completed or are underway. Examples in this volume are Brewer (1975), Hunter and Judge (1975), and Rogers and others (1975a). The work presented here is theoretical. Its use at this stage is not so much for detailed numerical prediction as for the insight it provides into processes which determine the sub-sea regime, and which should be studied as part of the experimental work. It will be seen how salt plays a key role in these processes.

At the scientific core of the sub-sea permafrost problem is the question of how old permafrost formed on land responds to changing temperature and salinity boundary conditions after an ocean transgression. Transgression seems to be the rule in the Beaufort Sea, not only from the melting of the Wisconsin ice 5,000 to 10,000 years ago, but also from present day active erosion of the coastline at a rate amounting to several meters per year in some places (Lewellen, 1970). As the shore line retreats we can imagine at least five stages in the evolution of the "surface" (no distinction being made between emergent land surface and shallow sea bed) temperature and salinity boundary conditions (Osterkamp, 1975). Stage 1 is the initial condition on land, stage 2 is beach, stage 3 is water less than 2 m in depth where the sea ice freezes to the bottom in winter, stage 4 is slightly deeper water where the ice does not freeze to the bottom but poor circulation causes the winter water to be highly saline, and stage 5 is deeper water in which circulation is sufficient to maintain normal sea water salinity all year. Other complications, such as ocean transgression into a fresh water lake, or reworking and deposition of shallow sediments, can be imagined.

In an equilibrium situation the thickness of the permafrost, usually defined as the depth to the 0°C temperature contour (Muller, 1947), is determined by the thermal conductivity, the geothermal heat flux, and the surface temperature (assuming there is no liquid motion). With ocean transgression the surface temperature becomes warmer, and the permafrost slowly thins until equilibrium is re-established; tens of thousands of years would be required in thick ice-rich permafrost at a location such as Prudhoe Bay. In the process any ice in the permafrost is melted from the bottom by geothermal heat. If the sea bed temperature is positive, the new equilibrium thickness is zero, and in the process of achieving it the permafrost will thaw from the top as well as the bottom (see MacKay, 1972). The question of what happens at the top if the sea bed temperature is negative is the problem addressed here. It is a highly relevant problem, because the state of the permafrost near the sea bed is of considerable engineering and environmental importance, and the mean annual bottom temperature in much of the Beaufort Sea is probably negative (MacKay, 1972; Lewellen, 1974; Osterkamp and Harrison, in preparation).

If the transgressing ocean maintains its temperature above that at which it begins to freeze, say -1.8°C, it will melt any ice in contact with it. A sub-bottom thawed layer will develop in any previously ice-bearing permafrost, with temperature and pore water salinity gradients in it. The rate of thawing may be primarily controlled by the rate that salt, rather than heat, penetrates the layer. Because of the role of salt, the process cannot be understood within the framework of a conventional thermal model. For the same reason, the conventional definition of permafrost, ground material below 0°C for several years (Muller, 1947), is less useful than on land.

Although the importance of salt seems obvious enough, the mechanism of its transport in the ground, or its possible presence before ocean transgression, are important questions ultimately to be answered experimentally, and the answers will depend on the location. But in the meantime the basic ideas can be illustrated fairly well by a simple theoretical model which can be solved in closed form, the predictions of which can be taken as a starting point for comparison with experimental data. In this model the initial condition
is a thick layer of frozen, ice-saturated, salt-free permafrost. At time zero this ma­
terial is suddenly covered with sea water at a temperature and concentration which remain constant thereafter. Only one spatial dimension is considered. The key physical assump­
tions are: (1) The pore liquid generated in the sediment as the ice melts does not move. Heat is then transported only by molecular conduction and salt by molecular diffusion; the heat transported by the moving salt can be neglected. (2) There is no salt sink in the thawed layer; in other words, no salt is adsorbed on soil particle surfaces. Some other assumptions and approximations are: The specific volume change of the ice upon melt­
ing is neglected; the influence of the soil particle surfaces on melting temperature is neglected; a sharp boundary exists between the thawed and frozen material. There is some field evidence for the last assumption (Osterkamp and Harrison, in preparation).

A preliminary version of this work is given in Rogers and others (1975b).

MODEL WITH THERMAL PROCESSES NEGLECTED

Because the diffusivity of salt in the thawed sub-sea permafrost is roughly three orders of magnitude less than that of heat, it is reasonable to assume, as a first approxima­
tion, that the thawing rate after ocean transgression is controlled by the rate that salt rather than heat diffuses, at least when the sea bed temperature is sufficiently negative. Therefore we first consider a simple model in which thermal processes are neglected en­
tirely. This is completely opposite from the situation on land, where permafrost condi­
tions can usually be understood within the framework of Fourier heat conduction. This model is illustrated by the following sketch.

If all the salt ions are assumed to have the same diffusivity $\kappa_s$, this simple model is described by a salt diffusion equation,

$$\kappa_s \frac{\partial^2 C}{\partial x^2} = \frac{\partial C}{\partial t},$$

where $C$ is the mass concentration of salt per unit volume of pore liquid, $x$ and $t$ are the space and time coordinates, and $x$ is measured positive downward from the sea bed. This equation is assumed to apply in the thawed layer; $C$ is assumed to remain zero in the frozen region. Initially,

$$C = 0 \quad t = 0$$

$$X = 0 \quad t = 0$$

where $X$ is the thickness of the thawed layer. After $t = 0$ the condition at the sea bed,
the upper boundary of the thawed layer, is
\[ C = C_0 \quad x = 0, \ t > 0 \]
where \( C_0 \) is the salt concentration of sea water. At the lower boundary of the thawed layer (\( \tilde{X} = X \))
\[ \frac{dX}{dt} = -\kappa_s \left( \frac{1}{C} \frac{\partial C}{\partial x} \right) \]
where the subscript \( X \) implies evaluation at this boundary. This boundary condition is the statement that as the ice melts, salt diffuses into the liquid so formed. (The flux is \( -\kappa_s \frac{\partial C}{\partial x} \), and the total \( C \) dX has to be transported in time dt.)

The concentration \( C_x \) at the lower boundary of the thawed layer must also be specified to determine the problem. Since as already noted the thawing rate must be very slow in this model, there should be adequate time for heat conduction to maintain the temperature of this thawed-frozen boundary at about the same value as the sea bed temperature \( T_0 \). Because two phases are present here, the salt concentration \( C_X \) is determined in terms of the temperature by the requirement of chemical equilibrium:
\[ \frac{C_X}{C_0} \approx \frac{T_0}{T_f} \]
where \( T_f \) is the freezing temperature of sea water (which has concentration \( C_0 \)). It is assumed that the phase equilibrium concentration varies approximately linearly with temperature.

The governing equation, the initial conditions, and the conditions at the upper boundary are satisfied by a solution of the form
\[ \frac{C}{C_0} = 1 - A \text{erf} \frac{x}{\sqrt{4\kappa_s t}} \]
(\( \text{erf} u \equiv \frac{2}{\sqrt{\pi}} \int_0^u e^{-v^2} \, dv \)), with the thickness \( X \) of the thawed layer increasing as
\[ X = \sqrt{4\kappa_s} \lambda \sqrt{t} \]
where \( A \) and \( \lambda \) are constants. \( \lambda \) could also be defined to include the factor \( \sqrt{4\kappa_s} \), but this form is more convenient and makes \( \lambda \) non-dimensional. \( A \) and \( \lambda \) are found from the conditions at the thawed-frozen boundary. Equation (2) gives
\[ A = \frac{(1 - T_0/T_f)}{\text{erf} \lambda} \]
so
\[ \frac{C}{C_0} = 1 - \frac{(1 - T_0/T_f)}{\text{erf} \lambda} \text{erf} \frac{x}{\sqrt{4\kappa_s t}} \]
(4)

Equation (1) gives
The function $\phi$ is defined for convenience. The solution for $\lambda$ can be easily read off a plot of $\phi$ against its argument, once the ratio $T_f/T_0$ is specified.

We shall consider numerical values representative of Prudhoe Bay, where field studies are in progress. Temperatures are $T_f = -1.8^\circ\text{C}$ and $T = -1.0^\circ\text{C}$, from which $\lambda = 0.567$ by equation (5). The salt diffusivity $\kappa_s$ is not known but $\kappa_s = 1.0 \times 10^{-2} \text{ m}^2 \text{a}^{-1}$ ($a$ is an abbreviation for year in SI units) is reasonable; this is 6.4 times the value for NaCl in free liquid at $0^\circ\text{C}$. Other values of $\kappa_s$ might be chosen (Stoessell and Hanor, 1975; Li and Gregory, 1974, for example), but our conclusions would be unchanged. Equations (3), (4) and (2) become

\begin{align*}
x &= 0.1135 \sqrt{\epsilon} \\
C/C_0 &= 1.000 - 0.770 \text{erf} \left( \frac{X}{0.200\sqrt{\epsilon}} \right) \\
\frac{C_X}{C_0} &= 0.555
\end{align*}

in meter-year units.

**MODEL WITH THERMAL PROCESSES INCLUDED**

Although the mechanism of permafrost thawing in a negative sea bed temperature regime is well illustrated by the previous model, further insight can be gained by generalizing it to include thermal processes as well. Solutions can still be found in closed form, if prior to $t = 0$ the permafrost is at the constant temperature $T_i$ at all depths. In the thawed region ($x < X$) the temperature $T$ and concentration $C$ are described by

\begin{align*}
\kappa_1 \frac{\partial^2 T}{\partial x^2} &= \frac{\partial T}{\partial t} \\
\kappa_s \frac{\partial^2 C}{\partial x^2} &= \frac{\partial C}{\partial t},
\end{align*}

and in the frozen region ($x > X$) by

\begin{align*}
\kappa_2 \frac{\partial^2 T}{\partial x^2} &= \frac{\partial T}{\partial t} \\
C &= 0.
\end{align*}

$\kappa_1$ and $\kappa_2$ are the thermal diffusivities in the thawed and frozen regions, respectively, and $\kappa_s$ is the diffusivity of salt. Initially

\begin{align*}
T &= T_i & t &= 0 \\
X &= 0 & t &= 0
\end{align*}

where $X$ is the thickness of the thawed layer. After $t = 0$ the condition at the sea bed, the upper boundary, is
where $T$, and $C$, are temperature and salinity at the sea bed. At the lower boundary of the thawed layer ($x = X$), the thawed-frozen interface,

$$T = T_0 \quad x = 0, \ t > 0$$
$$C = C_0 \quad x = 0, \ t > 0$$

where $T_0$ and $C_0$ are temperature and salinity at the sea bed. At the lower boundary of the thawed layer ($x = X$), the thawed-frozen interface,

$$\frac{dX}{dt} = -\frac{K_1}{h} \left( \frac{\partial T(x<X)}{\partial x} \right)_x + \frac{K_2}{h} \left( \frac{\partial T(x>X)}{\partial x} \right)_x$$

(7)

where $K_1$ and $K_2$ are the thermal conductivities in the thawed and frozen regions respectively, and $h$ is the volumetric latent heat released on melting. Also at the thawed-frozen boundary,

$$\frac{dX}{dt} = -\kappa_s \left( \frac{1}{C} \frac{\partial C}{\partial x} \right)_x$$

(1)

as before, and

$$\frac{C_X}{C_0} = \frac{T_X}{T_f}$$

(8)

which is similar to equation (2), except that the boundary temperature $T_X$ is no longer assumed to be equal to the sea bed temperature $T_0$. In what follows, it is convenient to replace the thermal properties $(\kappa_1, \kappa_2, \kappa_3)$ by $(\beta, \gamma, \alpha)$:

$$\sqrt{\kappa_1} = \alpha \sqrt{\kappa_s}$$

(9a)

$$\sqrt{\kappa_2} = \beta \sqrt{\kappa_s}$$

(9b)

$$\kappa_3 = \gamma \kappa_1$$

(9c)

If the temperature $T_X$ and therefore the concentration $C_X$ at the thawed-frozen boundary should remain constant as thawing proceeds, the problem would be easily solved. The temperature solution would be essentially that of Stefan and Neumann (Carslaw and Jaeger, 1959, Chapter XI), and the concentration solution essentially that of the previous section. Although it does not seem obvious on physical grounds that $T_X$ and $C_X$ should be constant, it turns out to be so, as can be verified a posteriori.

The Stefan-Neumann temperature solution is of the form

$$T = T_0 + \left[ \frac{(T_X - T_0)}{\text{erf} \frac{X}{\alpha \sqrt{\kappa_s} t}} \right] \text{erf} \frac{x}{\alpha \sqrt{\kappa_s} t}$$

(10a)

in the thawed layer ($x < X$), and

$$T = T_i - \left[ \frac{(T_i - T_X)}{\text{erfc} \frac{X}{\beta \sqrt{\kappa_s} t}} \right] \text{erfc} \frac{x}{\beta \sqrt{\kappa_s} t}$$

(10b)

(\text{erfc} u = 1 - \text{erf} u) in the frozen region ($x > X$), with the thickness of the thawed layer $X$ increasing as

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\[ X = \sqrt{4\kappa_S} \Lambda \sqrt{t}. \]  

(11)

\( T_X \) and \( \Lambda \) are constants, \( T_X \) being the thawed-frozen boundary temperature. As in the previous section the factor \( \sqrt{4\kappa_S} \) is included for convenience. The quantities in square brackets are also constants, determined by the requirement that \( T(x < X) = T(x > X) = T_X \) at the boundary. \( \Lambda \) is determined as a function of \( T_X \) by equation (7), which, after some manipulation, yields

\[
\frac{T_X^* - T_0^*}{\frac{1}{2} \phi(\Lambda/a)} + \frac{\gamma (T_X^* - T_i^*)}{\psi(\Lambda/\beta)} = \epsilon
\]

(12)

where non-dimensional temperatures, denoted by an asterisk, are defined in terms of the freezing temperature \( T_f \) by

\[ T^* \equiv \frac{T}{T_f}, \]

a non-dimensional parameter \( \epsilon \) is defined by

\[ \epsilon \equiv \frac{h\kappa_S}{(-K_1 T_f)}, \]

and two functions \( \phi \) and \( \psi \) are defined by

\[
\phi(u) \equiv \sqrt{\pi} u e^{u^2} \text{erf} u + 2u^2 \text{if } u << 1
\]

(13a)

\[
\psi(u) \equiv \sqrt{\pi} u e^{u^2} \text{erfc} u - \sqrt{\pi} u \text{if } u << 1.
\]

(13b)

These definitions permit equation (12) to be written in its fairly simple form, and make all quantities in it non-dimensional.

In the Stefan-Neumann problem, \( T_X \) is specified, and equation (12) can be solved for \( \Lambda \) to complete the solution of the problem. In our problem, however, \( T_X \) is an unknown. However, from the concentration solution,

\[
C/C_0 = 1 - \frac{1 - T_X/T_f}{\text{erf} \Lambda} \text{erf} \frac{x}{\sqrt{4\kappa_S} t}
\]

(14)

\[
\phi(\Lambda) = \left( \frac{1}{T_X} - 1 \right),
\]

(15)

which is found just as in the previous section except that \( T_X \) is not assumed to be equal to \( T_0 \), we obtain another relation, equation (15), between \( \Lambda \) and \( T_X^* \). Equations (12) and (15) can now be solved simultaneously for \( \Lambda \) and \( T_X^* \). Eliminating \( T_X^* \) we find

\[
\phi(\Lambda) = \frac{1 + \gamma (\alpha/\beta)^2 \frac{\psi(\Lambda/\beta)}{\psi(\Lambda/\beta)} - 1}{T_0^* + c\alpha^2 \phi(\Lambda/a) + \gamma (\alpha/\beta)^2 \frac{\phi(\Lambda/a)}{\psi(\Lambda/\beta)} T_i^*}
\]

(16)
Once this is solved numerically for $\Lambda$, $T_X^*$ is given by equation (15),

$$T_X^* = \frac{1}{\phi(\Lambda) + 1}$$  \hspace{1cm} (17)

and the solution is complete.

The Stefan-Neumann theory must follow as a limiting case of our theory. If $\kappa_1 \rightarrow 0$, no salt can enter the thawed layer, so at the thawed-frozen boundary $C_x = 0$, and therefore $T_x = 0$ by equation (8). Solution of equation (12) with $T_y = 0$ thus yields the Stefan-Neumann result. That the dependence on $\kappa_1$ is gone, as it must be, can be verified by replacing the variable $\Lambda$ by $\kappa_1 / \kappa_s$ $\Lambda'$, say.

As in the previous section we shall use numerical values representative of Prudhoe Bay. The salt diffusivity $\kappa_s \approx 1.0 \times 10^{-2} \text{ m}^2 \text{ a}^{-1}$ as before. The thermal diffusivities $\kappa_1$, $\kappa_2$ and conductivities $K_1$, $K_2$ are estimated using the method described by Gold and Lachenbruch (1973), assuming a volumetric water content of 40% and a rock conductivity of $24 \times 10^7 \text{ J m}^{-1} \text{ a}^{-1} \text{ deg}^{-1}$. The latter value is characteristic of randomly-oriented quartz, which is the dominant mineral in the sediments of interest (Osterkamp and Harrison, in preparation). The results are $\kappa_1 = 28$, $\kappa_2 = 69 \text{ m}^2 \text{ a}^{-1}$ and $K_1 = 8.3 \times 10^7$, $K_2 = 14.5 \times 10^7 \text{ J m}^{-1} \text{ a}^{-1} \text{ deg}^{-1}$. The volumetric latent heat $h = 1.34 \times 10^8 \text{ J m}^{-3}$, which reflects the assumed water content. The temperatures are $T_c = -1.8$, $T_o = -1.0$, $T_i = -9.0 \text{°C}$. From equation (9) it follows that $\alpha = 52.9$, $\beta = 83.0$, $\gamma = 1.749$, $\epsilon = 0.897 \times 10^{-2}$, $T_0 = 0.555$, $T_i^* = 5.00$.

For these values it is valid to use the small argument approximations for $\phi$ and $\psi$ (equations (13)) in equation (16), which becomes

$$1 + \frac{2}{\sqrt{\pi}} \frac{\alpha}{\beta} \Lambda T_X^* \frac{2}{\sqrt{\pi}} \frac{\alpha}{\beta} T_i^* \Lambda + 2e^{-2} - 1$$

This resembles the corresponding equation (5) in the previous section when thermal processes were neglected. In fact, one might estimate that as a first approximation $\Lambda \approx 0.567$, the value determined in the previous section. $\Lambda$ can then be replaced by this value on the right hand side of equation (18), which can then be solved for a better value of $\Lambda$ from a plot of $\phi$, just as in the previous section. Iterating once or twice one finds $\Lambda = 0.514$. Then by equation (17), $T_X^* = 0.612$.

Given the numerical values under consideration, equation (10a) can be simplified. Because $x < X$ and $X = \sqrt{4\kappa_s \Lambda} \sqrt{\varepsilon}$, $x / \sqrt{\alpha} \Lambda \ll 1$. Therefore $erf \left( \frac{x}{\sqrt{\alpha \kappa_s t}} \right) \approx \frac{2}{\sqrt{\pi}} \frac{x}{\sqrt{\alpha \kappa_s t}}$, using the small argument approximation for the error function. Equations (11), (10), (17), (14) and (8) finally become

$$X = 0.1028 \sqrt{\varepsilon}$$

$$T \approx -1.000 - 1.01 \frac{x}{\sqrt{\varepsilon}}$$

$$T = -9.00 + 7.96 \text{ erfc} \frac{x}{16.61 \sqrt{\varepsilon}}$$

$$T^*_X = -1.104$$

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\[
\frac{C}{C_0} = 1.000 - 0.724 \text{erf} \left( \frac{x}{0.200/\ell} \right) \\
\frac{C_x}{C_0} = 0.614
\]  

in meter-year-degree units. These are plotted in Figures 1 and 2. It is understood that although numbers are given to several figures in equations (6) and (19), they are subject to large uncertainties.

DISCUSSION

Comparison of equations (6) and (19) shows that the thawing rate and the concentration in the pore water are not greatly changed by the inclusion of thermal processes. The decrease in the thawing rate is about 10%. Therefore, for the particular sea bed temperature of \(-1^\circ\text{C}\) considered, the thawing rate is predominantly under chemical rather than thermal control. In general, the situation varies from total chemical control at sea bed temperatures close to the freezing point of sea water, to total thermal control at sufficiently positive temperatures. In the former limit only the mass transport property \(k\) enters the results; in the latter, only the thermal properties \(\kappa_1, \kappa_2, \kappa_3, \kappa_4\) and \(h\). The thermal limit can be calculated by conventional Stefan-Neumann theory with thawed-frozen boundary temperature \(T = 0^\circ\text{C}\). Figure 3 shows how the freezing rate and boundary temperature \(T_e\) depend on the sea bed temperature. At a sea bed temperature of \(+1^\circ\text{C}\) the rate is about six times faster than at \(-1^\circ\text{C}\), \(T_e = 0^\circ\text{C}\), and the thermal limit applies. The predictions of the \(T_e = 0^\circ\text{C}\) Stefan-Neumann theory are indicated by the broken line; this theory predicts the same thawing rate at a sea bed temperature of \(+0.1^\circ\text{C}\) as our theory predicts at \(-1^\circ\text{C}\).

The simple model considered here illustrates the coupling between heat and mass transfer processes. In many such processes this coupling appears in the governing equations as advective terms; for example, a velocity times temperature gradient term in the heat equation. These terms are negligible in this model, which includes diffusion only, and the coupling is via the phase equilibrium condition relating temperature and concentration at the thawed-frozen boundary.

One obvious factor that has been neglected in order to get a solution in closed form is geothermal heat flow. This means that the behavior of the temperature solution is incorrect at large depths, although it should be reasonably good for fairly small depths and times. Our approach would obviously break down if geothermal heat were to entirely melt the permafrost from the bottom over the period of interest, which is probably less than 10,000 years near Prudhoe Bay, since it is unlikely that the near-shore areas have been inundated any longer. During this time geothermal heat should thaw only the bottom 100 m or so of the 600 m total permafrost thickness (Gold and Lachenbruch, 1973); the thickness of the thawed layer at the \(-1^\circ\text{C}\) sea bed is 10 m in our model. It is not likely that geothermal heat affects this thickness much, since it is primarily controlled by the rate that salt diffuses down from the sea bed. The geothermal effect eventually must change the thawed-frozen boundary temperature somewhat from that calculated in our model, but in such a way that the heat flux into the boundary is roughly the same.

The rate of thawing from the top is very small in this simple diffusion model, being only 10 m in 10,000 years. Experiments now underway indicate that the thawed layer is typically much thicker than this. Our diffusion model therefore suggests that we should look for other mechanisms of salt transport such as liquid motion. Although this would be an important change in the physics, the theory could easily be generalized to include it. Numerical solution would be required.
Figure 1. Depth dependence of temperature at 1,100; and 10,000 years after ocean transgression. The position of the thawed-frozen boundary at each time is indicated by the symbol b. The temperature at the boundary is constant at -1.10°C. Seasonal temperature variations are neglected.
Figure 2. Depth dependence of salt concentration of pore water in thawed layer calculated from the coupled model. $C/C_0$ is the ratio of the concentration to that of sea water. The position of the thawed-frozen boundary is indicated; $C/C_0 = 0.614$ there. The depth scale applies at a time of 100 years after ocean transgression. To find the dependence after 1 year, multiply the depth scale by 1/10; after 10,000 years, by 10.
Figure 3. Dependence of thawing rate parameter $\Lambda$ and thawed-frozen boundary temperature $T_x$ on sea bed temperature $T_0$. The thawing rate is proportional to $\Lambda$. The broken line is the $\Lambda$ obtained by conventional Stefan-Neumann theory when salt transport is neglected ($T_x = 0$).
Our simple theory is really a stepping stone to a more comprehensive one, which is best developed as more data become available. Nevertheless, the simple theory (1) indicates how the problem should be formulated, (2) is easily generalized, (3) illustrates the important role of salt, (4) suggests that pore liquid motion is important, and (5) can be used to check numerical computation schemes as they are developed.

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REFERENCES


GEOPHYSICAL INVESTIGATIONS OF SUB-SEA PERMAFROST
IN THE CANADIAN BEAUFORT SEA

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ABSTRACT

The presence of extensive frozen sediments beneath the sea-floor of the southern Beaufort Sea has been predicted on theoretical grounds, interpreted from seismic surveys, and confirmed by drilling at several locations.

Theoretically it has been predicted that permafrost is probably present throughout the shelf area but probably does not exceed 75 m in thickness at water depths in excess of 80 m. At shallower water depths permafrost thickens shorewards to as much as 600 m in near-shore areas. In contrast with the permafrost underlying deep water, which is probably largely in equilibrium with bottom-water temperatures, permafrost underlying shallow water is believed to have grown during periods of emergence and in consequence both its presence and thickness are dependent on the surface history and morphology of the previous land surface.

Reflection and refraction seismic surveys have been used extensively to detect the presence or absence of frozen horizons and the configuration of the top of these horizons. Studies of the attenuation of refracted signals can give a general idea of thickness. The seismic results illustrate very well the local variability and heterogeneity of the frozen horizons.

Observations made as part of offshore drilling programmes have confirmed the seismic interpretation. Sub-sea bottom temperatures have been measured as a part of the drilling programmes. These have given insight into the thermal character of the permafrost indicating its marginally sub-zero temperatures, the presence of unfrozen horizons and the probable presence of local ground-water movement.

The existence of substantial thickness of generally degrading frozen sediments of high ice contents and temperatures marginally below zero creates many additional concerns for offshore resource development in northern waters.

*Earth Physics Br. Contribution.
INTRODUCTION

Sea-bottom sediments with mean annual temperatures less than 0°C are defined to be permafrost materials whether or not inter-granular ice bonding is present. This study concerns itself with the nature of permafrost in the southern Beaufort Sea shelf, and examines:

a) the areal and vertical distribution of permafrost and ice-bonded materials, their relationship with known geological and geomorphological features, and

b) thermal models applicable to this particular sub-seabottom environment based on the thermal history of the shelf and measured thermal parameters.

The existence and nature of sub-seabottom permafrost must be taken into consideration in advance of exploratory drilling for hydrocarbons; the ambient temperature conditions and the extent of thermal degradation or aggradation of ice-bonded sediments and the response of those sediments to man-induced thermal perturbation must be known for proper design of casing programs and positioning of sub-seabottom structures. Further, the occurrence of gas hydrates in association with permafrost is well known on-shore in the Mackenzie Delta area. The possible existence of such materials in association with permafrost areas offshore cannot be overlooked (Hitchon, 1975). In some cases occurrence of both gas hydrates and permafrost have led to unforeseen technical problems in exploratory drilling.

THE BEAUFORT SEA STUDY AREA

The study area under consideration extends from Hershel Island on the west to the Baillie Islands on the east and from the shoreline to the 200 m bathymetric contour line at the edge of the Beaufort shelf.

As seen from Figure 1, a bathymetric chart of the Beaufort shelf, the study area is far from being monotonously flat. The largest feature is the Mackenzie Canyon which is thought to be glacial in origin and is presently being filled by Mackenzie River sediments in the Shallow Bay area. The eastern boundary of the Canyon can be traced inshore from Garry Island to Tununuk.

Other N-S trending seabottom troughs exist to the northeast of Garry Island, north of Kugmallit Bay and north of Atkinson Pt. These are thought to be erosional expressions of old river valleys from some earlier time period of lower sea levels (J. Shearer, pers. comm.).

Further offshore, large areas containing numerous pingo-like features have been outlined. These features are found in water depths exceeding 20 m, primarily in the central and eastern portions of the study area.

OCEANOGRAPHY OF THE BEAUFORT SEA - BOTTOM WATERS

Essentially, beyond the influence of the Mackenzie outflow the water masses present in the southern Beaufort Sea are those identified by Coachman (1963) and O'Rourke (1974) as:

- Arctic water: 0 to 200 m, low salinity, cold (-1.5°C)
- Atlantic water: 200 to 900 m, high salinity, warm (>0°C)
- Bottom water: >900 m, high salinity, cool (-0.3°C).

The detailed results of Herlinveaux (1973) for a profile across the continental shelf north of Tuktoyaktuk suggest a depth of 250 m for the transition from Arctic to Atlantic water. Sub-zero bottom-water temperatures and hence sub-bottom permafrost is essentially confined to the shelf.
Figure 1. Bathymetry of the southern Beaufort Sea (depth in meters)
Seabottom temperature and salinity data for the southern Beaufort Sea were collected from the Marine Environment Data Service and Fisheries and Marine Branch, Department of Environment, and from unpublished surveys carried out by GSC and Earth Physics Branch, EMR. Figure 2 and 3 show the bottom temperature and salinity conditions. Although the data is somewhat scanty, the near-shore mean annual seabottom 0°C isotherm is between the shoreline and 20 m water depth. A further winter and summer 0°C isotherm follows the edge of the Beaufort Sea shelf; hence, most of the shelf area experiences permafrost temperatures. The central portions of the map area, north of Kugmallit Bay and the Mackenzie Delta, are subjected to temperatures below -1.5°C. The winter and summer seabottom salinities are closely related to the bottom temperatures. In summer, warm Mackenzie River water can be found on the bottom along the shallow coastal areas of the Tuktoyaktuk Peninsula. Although much of the river discharge is towards the west into Mackenzie Bay, cold saline water is found at seabottom in the deeper portions of the Mackenzie Canyon.

OBSERVATIONAL EVIDENCE OF PERMAFROST

Early Drilling and Marine Surveys

The first evidence for the existence of offshore and sub-seabottom permafrost in the Beaufort Sea was obtained from shallow engineering drilling by the Arctic Petroleum Operator's Association, reported by MacKay (1972). Figure 4 shows the locations of the holes drilled including several containing frozen ground. Ice-bonded frozen material was obtained in four of these holes at depths less than 50 m below seabottom. In-hole temperature measurements were not made and it is not possible to determine how many of the other holes may have encountered permafrost (sub-zero temperatures).

Seabottom sampling during the Beaufort Sea portion of the Hudson-70 cruise (B. Pelletier, GSC, pers. comm.) encountered evidence of frozen materials on the seabottom. Samples taken in the vicinity of a point 30 km north of Cape Bathurst showed evidence of ice lensing in both fine-grained and coarse-grained materials.

Shearer et al. (1971) describe numerous underwater seabottom mounds mapped on the outer portions of the shelf by shallow reflection seismic studies. These have been interpreted as pingoes formed in the bottom of ancient lakes subsequently inundated by rising sea levels. Formation of pingoes would require both seabottom temperatures below 0°C and sediments containing large quantities of low salinity water. Typical pingo-like seabottom features are shown in Figure 5. Some of these submarine peaks rise 30 m to within 15 m of the surface.

McDonald et al. (1973) have examined shot-hole data from near shoreline seismic surveys along the Tuktoyaktuk Peninsula. The top of ice-bonded permafrost was mapped in detail in unconsolidated sediments primarily composed of silty sands. Wherever the sea was frozen to bottom in winter, i.e., in shallow water, ice-bonded permafrost was found within 5 m of the seabottom. However, the occurrence of a small amount of water below the ice cover could be correlated with considerable depths to the top of ice-bonded permafrost. A typical section from McDonald's work is shown in Figure 6. Permafrost rises to surface beneath offshore spits as abruptly as at the mainland shore. Local unfrozen zones were encountered within the ice-bonded permafrost section.

Since the time of these early surveys, organizations such as A.P.O.A. and the individual resource companies such as Imperial Oil have drilled extensively, particularly in the shallow water areas, but their results have not yet been made public.

Temperature Measurement

To the authors' knowledge, none of the early drilling programs included the measurement of temperature in drillholes. Subsurface temperature is a very basic parameter in the assessment and understanding of permafrost conditions. Most of the problems arising in
Figure 2. Bottom water salinities in summer and winter (units in parts per thousand)
Figure 3. Bottom water temperatures in summer and winter (temperatures in degrees C)
Figure 4. Locations of APOA shallow drillholes (after Mackay, 1972)
Figure 5. Reflection profiles across pingo-like features in the southern Beaufort Sea
Figure 6. Permafrost profile at a Beaufort Sea shoreline (after McDonald et al., 1973)
permafrost engineering are due to changes in the physical properties of subsurface materials accompanying the phase change of ice to water or vice versa. In onshore locations a variety of techniques have been devised to measure subsurface temperatures and from them derive other physical information (Judge 1973, Taylor and Judge 1974). The techniques of long-term observation used very successfully at 25 onshore locations in the Mackenzie Delta are not easily adapted to offshore, except from permanent or semi-permanent platforms such as artificial islands. Even to date, subsurface temperatures relevant to sub-seabottom have been measured in only three of the nine wells drilled from artificial islands. Sun Oil Company Ltd. installed shallow thermistor strings at Unark L-24 (Richards Island near-shore) and Pelly B-35 (Pelly Island near-shore), the results of which were reported by Brown and Barrie (1976) and Imperial Oil Ltd. installed a deep cable at Adgo P-25 (near Garry Island) which has been read by EMR (Taylor and Judge 1976).

At present, with these few exceptions, our offshore programs have used bottom-hole temperatures measured during halts in the drilling and extrapolated to equilibrium. Such techniques work best in a secure hole such as a hollow-stem auger offers, but have been conducted to depths of 300 m in offshore diamond-drill programs and to depths of 100 m using shot-hole rotary rigs.

**Kugmallit Bay Drilling**

In the spring of 1974 the Geological Survey of Canada supported an offshore drilling program in Kugmallit Bay near Tuktoyaktuk, NWT (Judge et al. 1976) to

a) confirm the nature of the seismic refractor,

b) recover frozen and unfrozen cores to determine their lithology and engineering properties, and

c) determine the temperature profiles beneath the seafloor and determine the thermal properties of the seafloor materials.

Four holes were drilled at the locations shown in Figure 7 with a portable and skid-mounted Helli-Drill on contract from Big Indian Drilling Co. Ltd. On-site logging, sampling and drill hole temperature readings were performed by EMR personnel.

Core samples were taken with modified split barrel (spoon) and Shelby carbide samplers to prevent sample contamination by the drilling mud. The samples were later tested in the laboratory for grain size, thermal conductivity, moisture content and pore-water salinity. Downhole temperature observations were made as frequently as the coring program and the state of the hole allowed. The equipment used for the measurements was similar to the "portable mode" described by Judge (1973). The accuracy of the sensor used was ±0.01°C and accuracy in determining sensor depth ±0.05 m. Mud temperatures were monitored in drillholes 1 and 2 using similar equipment and a sensor accuracy of ±0.1°C.

The four drill holes in Kugmallit Bay penetrated varying thicknesses of clays and silts before encountering sands and gravels. The stratigraphy for each of the drill holes is shown in Figure 8.

The fine-grained sediments in the upper sections of the boreholes represent the more recent deposits in the Bay. They are supplied primarily by the East Channel of the Mackenzie River and partially by coastal erosion of the surrounding Pleistocene deposits. Immediately under the silts and clays are Pleistocene glacio-fluvial and deltic deposits of sands and gravels (Mackay 1963). Their exact age is questionable, but it is probable
Figure 7. Locations of G.S.C. drillholes in Kugmallit Bay showing positions of geophysical profiles.
Figure 8. Lithological descriptions of Kugmallit Bay drillholes
that they belong to pre-classical Wisconsin or classical Wisconsin time (Rampton 1972).

In the third drill hole, an interesting geological feature was observed when a saltwater-sand stream was encountered below the permafrost table. Samples from the 3.5 m deep channel gave temperature readings between +0.8°C and +1.8°C. Deeper sampling and temperature recordings were impossible as mud circulation was lost in the free-flowing saltwater-sand stream.

In-hole temperature and mud temperature measurements indicated that permafrost was present in holes #1, 2 and 3. No sub-zero temperatures were encountered in borehole #4. The shallowest permafrost temperatures were measured at depths of 56 m, 84 m and 63.5 m below the surface respectively in the first three boreholes. Closer estimates of the depth to the top of permafrost were possible by interpolation and, in the case of #1 and #2, by observation of the temperatures of the circulated drilling fluid. In #1, for example, the output temperature of the fluid fell by 1.5°C between depths of 46 and 61 m and remained constant at greater depths. The depth at which output temperatures started to fall is interpreted to be the depth at which frozen sediments were first encountered. Interpolation between observed downhole temperatures places the top of permafrost at 43 m. The combined use of bottom-hole temperatures, core temperatures and fluid circulation temperatures indicate depths to the top of permafrost of 45 ±2 m, 67 ±3 m and 61 ±2 m in the three holes in which permafrost was encountered. Extrapolation of temperatures in #4 indicates that permafrost could not be present at depths of less than 73 m. Depths to the top of the frozen section of the permafrost as determined by drilling speed were 54 m, 68 ±2 m and 64 ±1 m, respectively.

Thermal conductivity, moisture content and pore-water salinities were measured on each of the 34 recovered samples of core and cuttings. Thermal conductivities were measured on the divided-bar using perspex cells to contain the samples. The most apparent feature of the results is the distinct difference between the sands and gravels as one group, and the silts and clays as another. Average thermal conductivities for the 27 samples of sand and gravel were 2.61 ±0.29 Wm⁻¹K⁻¹ and for the seven samples of silts and clays 1.47 ±0.10 Wm⁻¹K⁻¹. Corresponding volumetric moisture contents were 31 ±6% and 50 ±2% in the unfrozen material. The two frozen sand cores had substantially higher moisture contents of 44% and 70%. Thus, excess ice was certainly present in the second core and probably in the first, although cubic packing of sand grains could result in interstitial porosities of up to 48%.

Salinity of the interstitial waters in the sediments was measured for 29 of the recovered samples, 24 of which were core samples. The method used was to add distilled water to the sample until a sufficient volume of water could be extracted to enable the use of a Yellow Springs salinometer.

No sea-water salinities were measured during the drilling period, but Kelly (1967) quotes values of up to 29% under the ice of Kugmallit Bay, so there could be some contamination of samples by the sea-water used in mud circulation resulting in higher values for the interstitial water than in situ values. Mean salinity values were lowest in the two boreholes, #1 and #2, closest to the shorelines and highest in #3 in which the sand stream was encountered. Grouping the results by lithology indicated the lower salinities of the pore fluids of the sands and gravels compared with the silts and clays.

Shallow Drilling at a Proposed Offshore Well Site

Engineering drilling at one of the two proposed Canmar offshore well sites was completed during the 1975 season. A hole was drilled at the Tingmiarvik site 80 km north of Tuktoyaktuk in a water depth of 30 m to a depth of 60 m below seafloor (C. O'Rourke, Canmar Ltd., pers. comm.). The hole encountered medium sand to a depth of 43 m below seafloor and
silty clay from that point to the bottom of the hole.

Ice-bonded permafrost was encountered from 34 m to 43 m below bottom. The sediment temperature at the bottom of the hole was -1.6°C, indicating a total permafrost thickness considerably in excess of that depth although no further ice-bonding was encountered.

SURFACE GEOPHYSICAL SURVEYS

Seismic Refraction Methods

The shallow refraction seismic technique is perhaps the best geophysical method available for mapping of sub-seabottom frozen ground in the Beaufort Sea. The successful application of this method depends on the difference in the seismic properties of unconsolidated water-saturated sediments between the unfrozen and frozen states. A typical plot of seismic compressional velocity with temperature below 0°C is shown in Figure 9, after Aptikaev (1964). For coarse-grained materials (sand and gravel) saturated with low-salinity interstitial water, most of the water is in the form of ice at temperatures slightly below 0°C. This results in an abrupt change in seismic velocity that is closely associated with the permafrost temperature boundary. For fine-grained muds and silts, the change in compressional velocity with decreasing temperature below 0°C is much more gradual. This effect has been explained by Nakano et al. (1971) as resulting from only partial freezing of interstitial water while part of the water is held by weak chemical bonding to clay minerals. At very low temperatures (-23°C), Nakano et al. report that only 50% available moisture is in the form of ice. Hence, seismic velocities of frozen materials when compared to the unfrozen state can be used as an indicator of the relative amounts of ice contained in the material, as well as an indicator of the grain size. In fine-grained materials, Frolov and Zykov (1971) have shown that velocity indicates the amount of excess ice in the soil if the temperature of the material is known.

The existence of ice-bonded permafrost at depth beneath the seabottom, from the point of view of seismic surveying, lends itself to interpretation by the refraction method. The basic criterion for the successful application of this technique is the continual increase of seismic velocities with depth in the geological section. The velocity model applicable to the Beaufort Sea consists of a low velocity water layer (1420 ms⁻¹) overlying unfrozen bottom sediments (1420-2000 ms⁻¹) which in turn overlay ice-bonded frozen materials (2000-4500 ms⁻¹).

Unfrozen consolidated sedimentary rock (shale, sandstone) exhibits velocities in the range of frozen ice-bonded unconsolidated materials. However, much of the southern Beaufort Sea shelf area is thought to be underlain by a thick sequence of unconsolidated Tertiary and Quaternary muds, silts and sands (Hofer and Varga 1972). Generalized velocity sections from analysis of reflection seismic records of the Mackenzie Bay area indicate more than 1000 m of sediments with velocities less than 2000 ms⁻¹. Hence, for most of the area, no ambiguity in the interpretation of ice-bonded permafrost should occur; a special case is made for the shoreline area of the southern Mackenzie Bay and it will be dealt with in a later section.

The seismic refraction method has been described by Hunter and Hobson (1974). Seismic rays radiating from a point source pressure pulse (dynamite or air-gun array) are refracted through the water, sediments and ice-bonded permafrost. Refracted energy from the two underlying high-velocity layers is continuously radiated upwards into the water column. At large distances from the source, energy from these high-velocity layers constitutes the first arrival event at the surface detectors. Analysis of the travel times of first arrival events along the detector array yields velocities of the underlying layers. These, in conjunction with the "break-over" distance (the shot-detector distance denoting the onset of first arrival energy from a lower layer) can be used to compute the depths to

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Figure 9. Variation of compressional wave velocity with temperature for various lithologies (after Aptikaev, 1964)
the layers using standard refraction computations as outlined by Dobrin (1970).

The Geological Survey of Canada has developed a marine refraction array for use in mapping of shallow high-velocity refractors. Details of the method have been given by Hunter and Hobson (1974) and Hunter et al. (1974). The array consists of 24 hydrophones placed in-line at 15 m intervals. A dynamite source is detonated off one end of the array, yielding a "single-ended" refraction profile. The accuracy involved in this type of refraction shooting in the case of dipping refractors and the errors involved in detector array curvature is described in Hunter and Hobson (1974).

The GSC refraction array has been operated from a variety of small vessels in various near-shore areas of the Beaufort Sea and the results have been published by Hunter (1973), Hunter et al. (1974), Hunter and Hobson (1974) and Carson et al. (1975). As well, surveying continued in the 1975 field season from the M.V. Pressure Ridge and the C.C.G.S. Nanhidik. Figure 10 is a compilation map showing locations of all GSC refraction lines from 1971 to 1975.

The GSC array is limited in depth of penetration by the shot-detector distance. The depth limitation is also, in part, dependent upon velocity contrasts between refractors; for contrasts found in the Beaufort Sea, the approximate limit of penetration is 130 m from sea surface.

Frequency response of seismic signals depends upon the initial spectrum of the source, the distance travelled by the pulse, the attenuation coefficient of the medium and, in the case of thin refracting layers, the layer thickness and velocity contrast. For this array the observed centre frequency of recorded pulse is approximately 100 hertz at the farthest shot-detector distance; hence, thin ice-bonded permafrost layers (5-10 m thick) can be detected by the array.

**Seabottom Refraction Profiling**

In addition to the sea-surface array outlined above, the GSC has been experimenting with a refraction array that can be deployed on the bottom. This array is 180 m long with a 15 m spacing between hydrophones. Small explosive sources are detonated off each end of the array; positioning of the hydrophones with respect to the source can be done by measuring the water-wave arrival time. The array was successfully deployed through leads in the sea ice during April 1975 at eight sites in the offshore shelf area, including both Kopenoar and Tingmiark proposed drill sites.

**Compilation of Industry Seismic Data**

The bulk of our seismic information on the occurrence of sub-seabottom frozen ground comes from records obtained from marine seismic reflection surveying by companies involved in oil and gas exploration. Since long multi-hydrophone cable configurations are used, the first arrival refraction information can be obtained from monitor records and unprocessed tape playbacks. The various exploration groups involved in seismic surveying in the Beaufort Sea were canvassed by the GSC and the approach was met by overwhelming response. Over 40,000 line-miles of data at an approximate record spacing of 1/4 mile have been examined for the occurrence of high velocities. The areal extent of frozen ground is reported by Hunter et al. (in press 1976) and shown in Figure 17.

An example of the results obtained from industry data is shown in Figure 11, from the Mackenzie Delta area of the Beaufort Sea (sections G1, G2 and G3, spaced two miles apart).

Below seabottom, the upper unfrozen layer has an average velocity of 1550 m/s overlying a high velocity layer in excess of 3000 m/s. The topography on the upper boundary of the high velocity unfrozen layer appears to be somewhat rugged because of the vertical
Figure 10. Locations of G.S.C. seismic refraction profiles in the southern Beaufort Sea
Figure 11. Interpretation of the depth of permafrost along several refraction profiles
exaggeration of the sections; however, in general, dips on the refractor are less than 10°. Depths to the top of the frozen layer vary between 15 m and 200 m below sea-bottom on the sections.

There are some indications that the refraction data obtained from the industry reflection records are not sensitive to fine scale frozen structures. A recent shallow hole drilled by CanMar Ltd. at a site approximately 80 km north of Tuktoyaktuk, encountered frozen sand and the top of permafrost at a depth of 34 m below sea bottom (water depth was approximately 30 m). The layer of frozen icy sand was approximately 10 m thick. The hole bottomed at 60 m below sea bottom at a temperature well below 0°C, but in unfrozen silt. The seismic interpretation based on industry data gave a depth to top of the frozen layer at 130 m below sea bottom. It is suggested that the seismic refraction energy travelling along the thin and frozen lens attenuated rapidly and was not detected on industry data. Subsequent shooting over the drill site by the G.S.C. using a shorter array confirmed the presence of the shallow frozen layer and the rapid attenuation rate of the refractor. Further, in adjacent areas where the industry data indicated no frozen ground, profiles shot with the GSC refraction array indicated thin near-surface layers of frozen ground. It is suggested the interpretation based on industry data served only as an indicator of the presence of thick frozen ground (in excess of 10 m).

Since the interpretation of frozen ground by seismic means depends upon the presence of interstitial ice, areas of ice-poor materials such as clay and materials with saline interstitial water at temperatures below 0°C will probably not be interpreted as frozen and may be misinterpreted as non-permafrost areas.

Seismic Reflection Methods

Other marine geophysical techniques may be of use in mapping sub-seabottom permafrost. Where velocities and refraction depths are known, detailed topography of the top of frozen ground can be obtained by high-resolution reflection techniques using sparker, boomer, or air gun sources.

Electrical Methods

Direct current resistivity soundings made on the sea bottom by Scott (1975) in the Kugmallit Bay area of the Beaufort Sea showed evidence of a highly resistive layer which could be interpreted as ice-bonded material at depth below sea-bottom.

Downhole Seismic Surveys

Several exploration wells have been drilled in the shoreline area and offshore on artificial drilling-islands. The drill logs of most of these holes still remain confidential information at the date of this writing. However, two of these wells with non-confidential status are of interest: these are Imperial Nuktak P-59 (Hooper Island) and Imperial Immerk B-48 (situated between Hooper and Pelly Islands), both of which had "crystal cable" seismic logs run on them. In the Hooper Island well the section frozen and ice-bonded with relatively high seismic velocities is from surface to a depth of 685 m. The Immerk well on the other hand contains ice-bonded material from only 150 m to 380 m below surface. No temperatures are available for the upper section and it is not possible to determine whether the upper section is below 0°C.

THEORETICAL STUDIES

A General Thermal Model

Failing the availability of extensive deep offshore subsurface temperature information, thermal simulation models must be used to assess the thermal character of the permafrost. The distribution of permafrost, its growth and its degradation, cannot be understood.
without a knowledge of the subsurface temperatures and the factors controlling them. Observations of the temperature distribution suggest that over much of the earth the dominant mechanism of heat transfer is conduction. In the absence of other mechanisms of heat transfer, this distribution in the upper kilometer of the earth is determined uniquely from a knowledge of the temperature distribution at the solid-fluid interface (both at present and in the past), the terrestrial heat flux, and the thermal properties of the medium present. Because of the nature of the phase properties of water, it is necessary to know the moisture content of the medium and the distribution of ice and water in the pore spaces over a range of temperatures. The freezing temperature is a function of dissolved solids in the interstitial water and the grain size distribution of the medium. There is some limited evidence that the mass transfer processes may play a role in determining the subsurface temperatures in some offshore areas. Evidence of the role of such processes in locally modifying the onshore thermal regime in permafrost areas is cited in the works of Balobaev et al. (1973), Oberman and Kakunov (1973) and Van Everdingen (1974). In the offshore areas, where extensive degradation of permafrost is probably occurring and permafrost is generally warming, conditions may be considered analogous to onshore areas of discontinuous permafrost.

Shearer et al. (1971) have described pingo-like features in the Canadian sector of the Beaufort Sea as indicative of hydrological processes acting at present or in the past. Judge et al. (1976) have described a saline water-sand stream encountered beneath the top of a frozen layer in Kugmallit Bay. The role of the diffusion of salt is hard to assess at present since there are few field observations bearing upon it (Molochuskii, 1973). Extensive observations of the basic thermal information both on- and offshore are the key to resolving these problems. At present, onshore subsurface temperature information is limited to Barrow (Lachenbruch and Brewer, 1959), Prudhoe Bay (Cold and Lachenbruch, 1973), and the Mackenzie Delta (Taylor and Judge, 1974, 1975, 1976). Offshore subsurface temperature information in the Beaufort Sea has been given by Lewellen (1973, 1975) and by Judge et al. (1976).

As a consequence of the A.P.O.A. drilling, Mackay (1972), through the application of the Neuman analytical solution of heat conduction in the presence of a phase boundary, suggested that ice-bonded permafrost may underlie much of the Beaufort shelf area. He also predicted that, as a result of past sea-level lowering and low mean annual surface temperatures, relic permafrost with a thickness up to 450 metres may exist.

A one-dimensional finite difference heat conduction model has been used by Judge (1974) to estimate the approximate thickness of remanent permafrost which might exist in the area north and east of the Mackenzie Delta (the area which underwent long periods of emergence during the Wisconsin ice age). Similar simulations have been used to determine the rate at which the subsurface thermal regime in this area will change as a terrestrial surface is incorporated into the offshore by recession of the coast line (Hunter et al., 1976).

For these calculations, subsurface thermal properties were derived from onshore wells such as Reindeer D-27 and from the Kugmallit Bay measurements; that similar lithologies occur both onshore and offshore has been verified by Hofer and Varga (1972). Times of emergence and submergence have been taken from Mackay (1972), and the surface temperatures used are -2°C during submergence and -9°C during emergence. The emergence temperature of -9°C is similar to surface temperatures in northern Richards Island today. This may be warmer than surface temperatures in front of a continental ice sheet. A bottom-water temperature of -2°C may be too low, since measured bottom-water temperatures seem to range from -1.5°C to -1.8°C in deep water, and may be 0°C or above in water <20 m deep.

In spite of the uncertainties in the input data, Fig. 12 taken from Judge (1974) illustrates the general permafrost thickness to be expected in the sea-bottom beneath dif-
Figure 12. Remanent permafrost thickness in Beaufort Sea sediments (Times of emergence and submergence are shown together with the depth of the 0°C isotherm. The depth of the equilibrium position of the isotherm denoted by E.; B.S.L. refers to depth below current sea-level; Emg. means surface emergent prior to 85,000 years ago; sub. means surface submerged prior to 115,000 years ago.)
ferent water depths (namely, 20, 40 and 60 m) and how that thickness changed with time in response to the time-dependent surface temperature history. The calculated permafrost thickness, the value on the depth axis, is still several times the equilibrium value at a water depth of 60 m. The effect of latent heat in retarding the establishment of thermal equilibrium is pronounced as is illustrated by comparison with the calculations of Lachenbruch (1957). As an illustration of how dependent the present distribution is on the pre-Wisconsin Pleistocene history, the present thickness below 20 m of water is shown for both an emergent and a submerged land surface prior to the Wisconsin glacial period. Permafrost thickness predicted by the two models is 500 and 350 m respectively.

Of particular interest and directly amenable to measurement in drill holes are the differences in heat flow which result. In this example, in which permafrost is degrading at present and in which the equilibrium heat flow below the base of the frozen layer is 63 mWm⁻¹, the heat flow exhibited within the permafrost ranges from 25 mWm⁻¹ in 20 m of water to 50 mWm⁻¹ in 80 m of water.

As mentioned elsewhere (Judge, 1975), the western delta exhibits very different thermal characteristics reflecting the very different thermal history. Using appropriate thermal input data for this area, simulations were used to place limits on permafrost occurrence and thickness and thus to explain anomalous seismic velocities in Mackenzie Bay, discussed in the following section. In Mackenzie Bay and Shallow Bay deltaic islands are emerging from the waters, creating a new land surface beneath which permafrost is aggrading anew.

The true picture in each of these areas is, of course, far more complex than these simple simulations allow. If the present Tuktoyaktuk Peninsula can be used as an example, much of the present offshore area may have been covered with lakes during emergence. In such areas permafrost will begin to aggrade when submerged beneath below-zero waters. Other complexities arise from the true nature of shoreline recession - the effects of rapid sedimentation, of shifting river channels, of complex shoreline geometries, etc. At present, knowledge of even the times of emergence and submergence and of ice-sheet boundaries remains somewhat speculative.

**COMBINED STUDIES**

**Anomalous Shallow Seismic Velocities in Mackenzie Bay**

A detailed study was made of 250 km of industry seismic records northeast of Shingle Point in the "Mackenzie Canyon" area.

A high velocity refractor with an average velocity of 3000 m s⁻¹ was observed over most of the area. Near the northeast offshore ends of the survey lines the refractor velocity decreases to 1800 m s⁻¹. An average velocity of 1520 m s⁻¹ was used for the combined water-unfrozen sediment layer to calculate depths to the high velocity refractor. A contoured map of depths to the refractor is shown in Fig. 13; the average depth is about 200 m, but decreases to about 100 m near the shoreline. From a generalized velocity-depth function for Mackenzie Bay given by Hofer and Varga (1972), the high velocity refractor observed at these shallow depths is unusually high for this area of the Delta.

To seaward of the study area, Shearer (1972) has mapped the base of the buried Mackenzie scour channel from high resolution seismic reflection profiling. The depths obtained for the base of the channel are in the same range as the depths computed to the high velocity refractor in the study area.

Included in Fig. 13 is an interpreted section beginning at the shoreline formed by recent deltaic sediments; the high velocity refractor apparently correlates with the bottom of
Figure 13. Depths to high velocity refractor (below sea-level) in southern Mackenzie Bay (contours in meters)
the scour channel offshore, but departs from it in the inshore region. Where the present shoreline coincides with the edge of the scour channel, the top of the high velocity layer generally conforms to the base of the scour channel. If the sediment type forming the bottom of the scour canyon is uniform and is below 0°C, then the seaward decrease of refractor velocity may result from increasing temperatures in frozen ice-bonded material.

Pursuant to a permafrost interpretation in the vicinity of the shoreline, the top of the frozen section would rise and the total thickness of it would probably increase due to the proximity of a cold northern shoreline of mean surface temperature of -10°C compared with seafloor temperatures close to 0°C. However, the wedge shape of the frozen ground persists to too great a distance offshore for an explanation based solely on the effect of the shoreline. A simple explanation of this elongation offshore could be found in a rapidly receding shoreline. At present the shoreline in the area is aggrading, as is shallow permafrost evidenced by a thin near-surface high velocity layer. Permafrost in the area is apparently very complex in nature. Aggradation is occurring at the present shoreline both from the surface and from an older shoreline, producing a thin permafrost layer in the near-surface and a wedge-shaped section at depth extending out under the sea to a distance of several kilometers from the present coast. Hence, permafrost underlying the seabed remote from the shoreline would be relict in nature and degrading at present, since present seafloor temperatures are too high to support deep permafrost in thermal equilibrium. Even if present seafloor temperatures in Shallow Bay were below 0°C, the terrestrial heat flux could support only a current permafrost thickness of 100 m. That same flux is sufficient to have also melted a maximum of 200 m of relict permafrost in the past 10,000 years.

Thus, it is difficult to explain the high velocity refractor by the presence of the permafrost unless conditions were suitable in the past for thicknesses in excess of 300 m to have accumulated on the south side of Shallow Bay. As discussed by Judge (1975), the Mackenzie Canyon area of which it forms a part underwent a very different history to that of the offshore north of Richards Island. For much of the Wisconsin it was covered by an ice sheet (Mackay et al., 1972) and thus exposed to less severe surface temperatures than the unglaciated parts of the Beaufort Sea. Consequently, permafrost at the end of the Wisconsin period was, for example, probably thinner than that on Richards Island today. At the time of recession of the ice sheet 14,000 to 16,000 years B.P., sea-level was as much as 70 m below present sea-level. Therefore, the rate of inundation of the area by the sea depends on the rate of accumulation of post-glacial sediments. Assuming an ice-base temperature during the Wisconsin of -20°C and an exposed land surface to have existed between glacial recession and the time of stabilization of sea level 5000 years B.P., as much as 200 m of permafrost could have aggraded. Decreasing the Wisconsin ice-base temperature increases this thickness by approximately 35 m per 1°C decrease. For the top of the frozen section now to be at a depth of 200 m, either very high rates of degradation or very rapid burial is necessary. Neither explanation seems likely.

In the central and northern parts of the Mackenzie Canyon area, seismic velocities indicate unfrozen sands at the base of the scour channel. These results are confirmed by subsurface temperature measurements made in the onshore portions of Shallow Bay (Taylor and Judge, 1974). At each of these sites permafrost is relatively thin (60 to 150 m) and commences at the surface, consistent with young sediments in which permafrost is aggrading. Positive temperature gradients of 20 to 45°C km⁻¹ indicate no relict permafrost at depth. These observations place severe limits on the total amount of permafrost which could have accumulated in the Mackenzie Bay area.

It is perhaps worth reiterating that permafrost and hydrofrost (gas hydrates) may possess similar seismic velocities in coarse-grained materials. Once again, however, the limited subsurface temperature measurements in the onshore portions of Shallow Bay would tend to rule out the presence of gas hydrates under equilibrium geothermal conditions unless they are of high specific gravity (>0.6). Under highly non-equilibrium...
conditions, thick, shallow gas reservoirs might remain at present in the hydrate form almost anywhere in Shallow and Mackenzie Bays. A more reasonable history for the area, such as that suggested in the permafrost interpretation, could result in gas hydrate deposits at depths exceeding 100 m in areas that formed the margins of the Wisconsin ice lobe (the edges of the Canyon?).

Alternatively, the high refractor velocity may result from an older geological formation which has been structurally uplifted.

In summary, no definite interpretation of the seismic refractor can be made. It may be ice-bonded permafrost, although from what is known of the surface history of the area it is difficult to explain its presence remote from the shoreline. It may be hydrofrost under suitable conditions which could pertain to certain parts of Mackenzie Bay or, yet a third alternative, it could represent an older uplifted formation. A final solution must await the acquisition of deep subsurface temperature profiles in the area.

A Comparison of Seismic and Drilling Results, Kugmallit Bay

The results of shallow refraction shooting with the GSC array near Base Summit in Kugmallit Bay are compared with the drilling results from GSC drill hole #1 in Fig. 14. Close to shore the top of a high velocity layer is detected at a depth of 20 m, but becomes apparently discontinuous (beyond the range of the shallow refraction array), and disappears to seaward. The depth to the top of ice-bonded permafrost given by the seismic refraction results are reasonably close to that given by temperature measurements in the drillhole.

Fig. 15 shows a refraction section across Kugmallit Bay shot with the GSC array along with an interpretation of air gun shallow reflection data (J. Shearer, GSC, pers. comm.) for a line approximately one mile north of the refraction line, from Toker Pt. to the middle of the Bay. The data indicates that the interpreted upper boundary of ice-bonded permafrost is considerably shallower towards either shore. In the centre portions of the Bay, the interface drops to a depth of 90 m and becomes discontinuous in some areas. The discontinuities in permafrost are seen on both reflection and refraction records and suggest that the top of ice-bonded permafrost is either quite deep (100 m) or absent. Alternatively, since clay at temperatures of -10 to -20°C is usually indistinguishable seismically from the unfrozen state, the areas lacking high velocities could be interpreted as thick clay deposits in the frozen state. The average velocity of permafrost interpreted over the section is 3290 ms⁻¹, suggesting that the material is frozen coarse-grained silts or sands. A good correlation exists in general between the top of ice-bonded permafrost as determined by the seismic work and the top of permafrost as given by temperature measurements. DH #2 was placed in an area where seismic data suggests that the top of ice-bonded permafrost is rising sharply, hence a navigation error of 100 m and possible computation error of 10% resulting from the dipping interface would more than account for the difference in interpreted depths to the top of permafrost.

Seismic evidence for the existence of the thin permafrost lens at DH #3 can be found from a close examination of the seismic records. Fig. 16 shows the measured "first break" amplitudes for the permafrost refractor for record #188 shot over DH #3 and for record #192 shot approximately 2400 m away in an area which is interpreted as thick permafrost. Studies on the thin layer problem by Risnichenko and Shamina (1957), Rosenbaum (1965), Donato (1965), Poley and Nooteboom (1965) and others suggest that the refracted wave along a thin, high-speed layer is characterized by a low amplitude and a high attenuation rate compared to that of a refractor in a thick layer. If proper assumptions are made for mathematical constants, approximate formulae given by Rosenbaum and Donato can be employed to determine the layer thickness. With no information on which to base these assumptions, however, it is still possible to qualitatively assess the relative thickness of the permafrost layer from examination of "first-break" amplitude-distance curves since the attenuation rate increases with decreasing layer thickness.
Figure 14. A seismic refraction profile across DDH #1 of G.S.C. drillholes in Kugmallit Bay
Figure 15. Summarized geophysical and drilling results for a profile across northern Kugmallit Bay
Figure 16. Amplitude: distance curves for seismic slots in Kugmallit Bay
Marine DC resistivity soundings made by Scott (1975) in the Kugmallit Bay area indicated that a highly resistive layer lies at depth below the sea-bottom. Depths determined to the top of this layer are given on Fig. 14 and Fig. 15. The depth to permafrost given by the soundings is, in general, somewhat deeper than that given by the seismic and drilling. This is probably in part a result of the larger area over which the resistivity sounding is made. It is interesting to note that permafrost is interpreted to be present at considerable depth below DI #4. No indication of permafrost was found on the seismic reflection or refraction records, although the interpreted depth is at the limit of the range of both seismic systems employed.

CONCLUSIONS

As a result of the studies conducted to date, the authors conclude that

1. sub-seabottom permafrost exists over much of the Beaufort Sea shelf. (The results are summarized in Fig. 17);
2. ice-bonded permafrost is confined to the eastern half of the survey area;
3. excess ice has been observed in shallow offshore drillholes, although not to the same extent as onshore;
4. sea-bottom morphological features can be associated with ice-bonded permafrost;
5. maximum permafrost thickness varies from 600 m at the shoreline to 100 m or less in the deep waters of the shelf;
6. the upper boundary of ice-bonded permafrost can occur at depths between sea-bottom and 200 m below sea-bottom;
7. because of non-equilibrium thermal conditions, permafrost is generally aggrading downwards from the sea floor in water depths of 20 m or more but in the inshore region permafrost present is degrading downwards from the sea-floor;
8. temperatures of sub-seabottom permafrost are much warmer than those occurring nearby on land and thus ice-bonded sediments present are more susceptible to thermal degradation, as a consequence of a small heat input, than the nearby onshore sediments.

REFERENCES


Figure 17. Distribution of ice-bonded permafrost in the southern Beaufort Sea as determined by seismic observations.


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DETERMINATION OF WAVE-INDUCED PRESSURES IN THE SOIL MEDIA CONTIGUOUS TO A BURIED PIPELINE

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ABSTRACT

The stability requirements for burying an offshore pipeline to protect it from the dynamic effects of waves is considered. Presentation is made of two numerical models based on the finite difference and finite element methods as a means for obtaining the time varying pressure distribution in the soil media surrounding a buried pipeline. The theoretical basis, development, and application of the models is made. Results are presented that show a significant alteration of the dynamic pressure field in the soil media close to the pipeline can take place and may contribute to the failure mechanisms of interface erosion and soil liquification.
INTRODUCTION

Pipelines laid on the ocean bottom for conveyance of gases and liquids are subject to the potentially destructive forces produced by both waves and bottom currents. Each year, substantial financial losses are sustained throughout the world due to damage produced by the waves accompanying severe storms. During the period 1964-1965 losses of over 200 million dollars in pipeline facilities were sustained in the Gulf of Mexico alone as a result of two major hurricanes.

An additional degree of protection from wave and current action may be gained by burying the pipeline. As ocean waves can induce fluid velocities and dynamic pressures in the bottom sediments even in water of considerable depth, the rational determination of the depth to which a pipeline must be buried is of basic importance to the designer. This requires a fundamental understanding of the interaction process that occurs between the pipeline, the marine sediments that surround it and the fluid motion associated with wave action. At present, burial requirements are based largely on empirical information and designer experience, as a complete mechanistic picture of the interaction process that can lead to unburying the pipe and its eventual failure are still undefined.

It is considered possible that the dynamic pressure and velocity field variation at frequencies associated with surface waves, may contribute to interface erosion and to alteration of the sediment structure so as to reduce its effective strength, thus contributing to possible instability (as would be associated with short duration, intermittent soil liquefaction).

It is the long term objective of the work presently being performed at Texas A&M University to attempt to understand the interaction processes so that adverse effects may be minimized and improved design criteria for constructing offshore pipeline facilities to withstand the environmental conditions existing at a given site may be established.

HYDRODYNAMIC CONSIDERATIONS

Consider Figure 1, which shows a two-dimensional sinusoidal wave in a fluid layer of mean depth $h$, overlying a homogeneous porous medium of infinite depth. A pipe of diameter $D$ is embedded in the porous medium at a burial depth $d$. The surrounding fluid has a kinematic viscosity $\nu$, and the porous medium a permeability $K$ and porosity $n$. We shall be concerned with small amplitude, progressive plane waves acting as the hydrodynamic loading on the system. The study follows earlier investigators in this field (4,6,7,8) in using a boundary value approach to solve an assumed potential flow problem. As waves pass over the porous bed, pressure fluctuation on the fluid-bed interface induces a viscous flow within the porous medium, causing dissipation of mechanical energy and thus damping of the waves. This pressure field associated with the wave will be further altered to some degree if an impervious object such as a pipe is placed in the upper region of the bed. For viscous flow in the porous bed, the Reynolds number will be less than unity and Darcy's Law becomes applicable. The following is the form of Darcy's equation utilized by Reid and Kajiura (7)

\[
\frac{1}{n} \frac{\partial u}{\partial t} = - \frac{\nu}{K} u - \frac{1}{\rho} \frac{\partial p}{\partial x} \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (1)
\]

\[
\frac{1}{n} \frac{\partial v}{\partial t} = - \frac{\nu}{K} v - \frac{1}{\rho} \frac{\partial p}{\partial y} \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (2)
\]

Where $u$ and $v$ are velocity components in the $x$ and $y$ directions, $p$ is the dynamic pressure which is the object of concern, and $\nu$ and $\rho$ are the kinematic viscosity and density respectively of the fluid. It is assumed that the porous medium is homogenous, so that the permeability $K$ and porosity $n$ are constant and non-directional. For unaccelerated
Fig. 1 - Definition Sketch

Fig. 2 - Grid System Around a Circular Pipe

Fig. 3 - Finite Difference Scheme on Circular Pipe Surface
flows Equations 1 and 2 can be reduced to the more common form of Darcy's equation

\[ u = -\frac{K}{\nu_p} \frac{\partial p}{\partial x} \]  
\[ v = -\frac{K}{\nu_p} \frac{\partial p}{\partial y} \]  

If the fluid is considered incompressible, the flow in the porous medium must satisfy the continuity equation of the form

\[ \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0 \]  

Combining Equations 1, 2 and 5 gives the Laplace Equation

\[ \frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} = 0 \]  

which governs the distribution of pressure \( p \) in the porous medium.

**NUMERICAL MODELS**

Earlier investigators tended to solve this Laplace equation within the porous region with various complicated analytical approaches without actual success (2,5,7). The addition of a foreign body—the buried pipeline—tends to make the problem more complex. Numerical models are introduced in this study which use the finite difference and finite element technique to simulate the interaction of the pipeline—soil media—wave system.

**Finite Difference Model**

The Laplace equation and its different boundary conditions can be approximated by finite difference equations and solved by iteration techniques. A grid system is shown in Figure 2 which is designed to have a finer mesh immediately around the pipe, offering smaller spacings and thus more points of interest in that region. A more detailed picture of the pipe surface is shown in Figure 3 which shows the approximation of the normal derivative boundary condition

\[ \frac{\partial p}{\partial n} = 0 \]  

on the circular surface. The finite difference form of the Laplace Equation is solved by an iterative technique called the relaxation method. Consider the finite difference equation for any point \((i,j)\)

\[ p_{i,j} = \frac{(p_{i-1,j} + p_{i+1,j-1}) + 4ER(p_{i-1,j} + p_{i+1,j} + p_{i,j-1} + p_{i,j+1})}{2ER} \]  

A set of equations can be written for the whole network of grid points within the problem region of concern. Unknown values in the \(i+1\)th row are specified in terms of known values in the \(i\)th row by a single application of the above expression. If there are \(N\) unknown values in the \(i+1\)th row, the general equation above must be applied \(N\) times across the length of the row. The resulting system of \(N\) simultaneous equations specifies the \(N\) net values implicitly. A complete derivation of the method can be found in Reference 3.
Finite Element Model

In the finite element representation, a variational, extremum approach, valid over the whole region of concern is used and the solution is the one minimizing some quantity \( I \) which is defined by suitable integration of the unknown quantities over the whole domain. Thus the Laplace equation (6) subject to prescribed value \( p = p_b \) on the outer boundary and the normal derivative boundary condition, Equation (7), on the pipe surface inside the region can be mathematically shown by using the Euler-Lagrange Equation that this is equivalent to finding a function \( p \) which satisfies the boundary conditions and minimizes

\[
I = \iint \left[ \frac{1}{2} \left( \frac{\partial p}{\partial x} \right)^2 + \frac{1}{2} \left( \frac{\partial p}{\partial y} \right)^2 \right] \, dx \, dy \tag{9}
\]

For the approximate solution we shall assume the region to be divided into finite elements, Figure 4, in each of which

\[
p = \begin{bmatrix} N_1 & N_2 & \cdots \\
\end{bmatrix} \begin{bmatrix} p_1 \\
p_2 \\
p_j \\
\end{bmatrix} = \begin{bmatrix} N \end{bmatrix} \begin{bmatrix} p \end{bmatrix}^e \tag{10}
\]

where \( \{p\}^e \) represents a column of pressure values at the element nodes. Combining contributions from the three node points \((i,j,k)\) of the triangular element in conjunction with Equation (9) gives

\[
\begin{bmatrix} \frac{\partial I}{\partial p} \end{bmatrix}^e = \begin{bmatrix} E \end{bmatrix} \begin{bmatrix} p \end{bmatrix}^e \tag{11}
\]

for each element. \( \begin{bmatrix} E \end{bmatrix} \) thus formed is the Element Stiffness Matrix (ESM). By summing up all the elemental contributions

\[
\begin{bmatrix} \frac{\partial I}{\partial p} \end{bmatrix} = \sum \begin{bmatrix} E \end{bmatrix} \begin{bmatrix} p \end{bmatrix}^e = 0 \tag{12}
\]

a Global Stiffness Matrix (GSM) is formed.

\[
\begin{bmatrix} \frac{\partial I}{\partial p} \end{bmatrix} = \begin{bmatrix} GSM \end{bmatrix} \begin{bmatrix} p \end{bmatrix} = 0 \tag{13}
\]

The above equation is then solved by the Gaussian Elimination Method, and the pressure values \( p \) can be found at each of the node points.

In this study, elements of triangular shape are used. These elements can be graded in shape and size to follow arbitrary boundaries and to allow for regions of rapid variation of the function sought. Figure 5 shows how the flow region can be subdivided into finite elements. Essentially two systems of triangular elements are utilized—an outer region of regular-sized elements and an inner region of finer, irregular-sized elements immediately surrounding the pipe. The curved pipe boundary can be approximated by small triangular elements constituting a multi-sided polygon. The size and number of these inner elements is determined by the accuracy required in the problem. Different systems of elements can be defined for the region depending on the size of the pipe and its burial depth.

**SOLUTION DEVELOPMENT AND RESULTS**

As part of this study, a validation check of the numerical models with existing
Fig. 4 - Problem Region of Finite Element System

Fig. 5 - Finite Elements Around a Circular Pipe
analytical and experimental results was undertaken. Analytical results were available for the dynamic wave pressure distribution in the soil without a buried object in the region. Most are the results of Liu (4) and Sleath (8) in regard to their wave damping studies. An experimental study of the pressure distribution about a buried pipe was conducted as an integral part of this overall project (1). Shown in Figure 6 is a comparison of finite element solutions with Liu's analytical results by using wave and soil parameters listed in Table 1. In this comparison, a zero value bottom boundary condition is specified in the numerical model at a depth of one wave length to simulate the damped wave pressure. Test runs showed that the numerical models can produce results which agree extremely well with Liu's analytical solution. In addition, comparisons were made with experimental laboratory wave channel data measuring the dynamic pressure that occurs in the soil media, Figure 7. Modification of the finite element region was necessary to simulate the impervious bottom boundary corresponding to the physical conditions of the experimental study in the numerical model. As seen in Figure 7, curves obtained from the finite element model were shifted to the right of the experimental results. Better agreement is evident for the curves corresponding to the smaller waves. It is believed that this shifting was largely due to the difference in the top boundary values appearing in the two approaches, which may have been caused by the irregular sand ripples formed in the course of the experiment.

However, curves generated by both methods thus follow much the same damping pattern. Better agreement could be achieved if actual, measured, boundary values were used as input in the numerical model.

Some of the results obtained from the finite difference model are shown in Figure 8 and Figure 9 by using wave and soil parameters in Table 2.

In Figure 8 are plotted the pressure distributions in the soil region immediately contiguous to the pipe surface. Also plotted are values in the same region that occur without the addition of the buried pipe which causes a slight increase of pressure at the top of the pipe and a decrease at the bottom. Figure 9 shows the pattern of the wave pressure damping along axis A-B, and the effect of the embedded object on the pressure distribution.

In the finite element model, the same wave and soil conditions in Table 2 were applied. Figure 10 shows a series of plots of the pressure distribution along the pipe surface at different instances of time so as to simulate the action of a progressive surface wave. A similar result was obtained as in the finite difference model, only that the increase in pressure at the top of the pipe is somewhat smaller in this case, and the decrease at the bottom is more pronounced.

The generally good agreement obtained by both models to existing analytical solutions and experimental data supports the extension and use of these models as a means for evaluating dynamic pressure distribution about an embedded object. It appears that both the finite difference and finite element models are adequate although the finite element approach may have some distinct advantages. Better accuracy is expected in the finite element approach since the normal derivative condition on the pipe surface can be introduced naturally in the problem, while the finite difference model depends on an approximation of the normal derivative equation, which again depends on mesh sizes which have to be comparable to the boundary layer thickness surrounding the embedded pipe.
Table 1 - Wave and Soil Parameters

<table>
<thead>
<tr>
<th>Case</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (in.)</td>
<td>3.2</td>
<td>2.0</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>T (sec.)</td>
<td>1.19</td>
<td>1.09</td>
<td>1.48</td>
<td>1.62</td>
</tr>
<tr>
<td>L (ft.)</td>
<td>4.5</td>
<td>4.0</td>
<td>6.17</td>
<td>6.25</td>
</tr>
<tr>
<td>h (ft.)</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>K (ft.)</td>
<td>$5.62 \times 10^{-10}$</td>
<td>$5.62 \times 10^{-10}$</td>
<td>$5.62 \times 10^{-10}$</td>
<td>$5.62 \times 10^{-10}$</td>
</tr>
</tbody>
</table>

Fig. 6 - Comparison of Finite Element Method With Liu's Analytical Solution

Fig. 7 - Comparison of Finite Element Method with Experimental Results
Fig. 8 - Pressure Around a Circular Pipe at $t = 0$
(Finite Difference Method)

Table 2 - Wave and Soil Parameters

<table>
<thead>
<tr>
<th>H (in.)</th>
<th>T (sec.)</th>
<th>L (ft.)</th>
<th>h (ft.)</th>
<th>K (ft.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>1.04</td>
<td>4.69</td>
<td>11.2</td>
<td>4.83x10^{-5}</td>
</tr>
</tbody>
</table>
Fig. 10 - Pressure Around Circular Pipe (Finite Element Method)
REFERENCES


NOTATION

D = pipe diameter
p = wave induced pressure

d = pipe burial
pb = boundary pressure value

ESM = Element Stiffness Matrix
T = wave period

GSM = Global Stiffness Matrix
t = time

h = still water depth
u = fluid particle velocity component in x-direction

I = finite element functional
v = fluid particle velocity component in y-direction

K = permeability
x = horizontal coordinate

L = wave length
y = vertical coordinate

n = porosity
\rho = mass density of water

v = kinematic viscosity
NEAR SHORE PERMAFROST IN THE VICINITY OF PT. BARROW, ALASKA

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ABSTRACT

Shallow seismic refraction studies near Pt. Barrow, Alaska, are reported. These studies, which depend upon a significant seismic velocity difference between frozen and non-frozen soils, delineated areas of frozen and non-frozen material beneath the active layer along Pt. Barrow. In general, the P wave velocities in frozen materials were observed in the range of 2500 m/sec to 3000 m/sec while velocities in the non-frozen materials of similar composition to the frozen samples display velocities from about 1500 m/sec to 2000 m/sec.

Good correlation was found between the occurrence of surface vegetation (scattered grasses) along Pt. Barrow and the underlying ice-bonded permafrost. However, areas lacking such vegetation generally were not underlain by ice-bonded permafrost. Seismic refraction surveys along the spit between Pt. Barrow and Plover Point failed to locate fast layers associated with permafrost to depths of at least 46 meters. Additional work on Tapkaluk Island approximately 20 kilometers east of Pt. Barrow indicated no bonded permafrost to depths of about 44 meters. It is concluded that if continuous ice-bonded permafrost exists in these locations it is below these depths.

Additional refraction lines run across selected portions of Elson Lagoon indicated no continuous bonded material to depths of at least 160 meters. Further work along the edge of the lagoon just southwest of Pt. Barrow did not indicate bonded permafrost beneath this water. It is estimated that these results are only applicable above an imaginary plane which dips beneath the waters of the lagoon from the Pt. Barrow shore with a dip angle of at least 30° below the horizontal. Thus, it may be possible that continuous bonded permafrost exists beneath the lagoon edge if it were below such a steeply dipping surface.

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INTRODUCTION

The work described herein was performed during the months of August and September 1974 in the vicinity of Pt. Barrow, Alaska.

Figure 1, an area map of the Pt. Barrow vicinity, indicates the two principal areas of study. Area A is immediately on Pt. Barrow while Area B is approximately 15 miles to the east in the Tapkaluk Islands.

![Map of Pt. Barrow, Alaska, vicinity. Two areas of seismic refraction studies, "A" and "B", are shown. The Barrow community is approximately five miles southwest of Brant Point along the coast.](image)

The work involved measurement of seismic velocities by refraction methods in the subsurface materials. These velocities are related to the frozen or non-frozen state of the materials, with high velocities corresponding to the frozen state and low velocities corresponding to the non-frozen state. Although most of the refraction lines were run on land or right along the beach, some lines were run beneath Elson Lagoon and on the Tapkaluk Islands. The results of these measurements give us a preliminary idea of the ice-bonded permafrost distribution along the edge of Elson Lagoon at Pt. Barrow.

INSTRUMENTATION AND METHOD

The instruments used for this survey were an SIE 6-channel reflection/refraction unit and a Bison signal enhancement hammer seismograph. In order to examine the limitations and constraints of the method and therefore put the results in proper perspective, it is valuable to review briefly the seismic refraction technique.

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Figure 2 is a diagram of a simple seismic refraction measurement. The seismic source indicated sends seismic energy in all directions and receivers 1 through 7, which may be hydrophones or geophones, receive a portion of this energy. In this case we have assumed two horizontal layers with homogeneous velocities - layer 1 with velocity $v_1$ which is less than velocity $v_2$ in layer 2. The refraction technique used depends upon the time of the first energy arrival at a particular receiver. In this sketch we can see that receiver 1, receiver 2, receiver 3 and receiver 4 receive energy that travels at velocity $v_1$ just beneath the surface directly from the seismic source to each respective receiver. However, some energy which leaves the seismic source and travels to the

![Two Layer Earth Seismic Energy Paths](image)

Figure 2. A sketch of an ideal two layer earth showing the ray paths from the source to receiver points 1 through 7. Note that for points 1 through 4 the first energy received at the geophones travels through Layer 1. However, for points 5 through 7 the seismic wave which travels down through Layer 1 to the geophone is faster than the direct wave which travels through Layer 1 only. Thus it is possible to determine the velocity in Layer 1, the velocity in Layer 2, the depth to Layer 2 and whether the interface between Layers 1 and 2 is sloping with respect to the surface of Layer 1. (Dobrin, 1960)

interface between the layers at velocity $v_1$ is refracted according to Snell's Law along the interface of the two layers and travels with a higher velocity $v_2$. While it is traveling, energy is constantly leaking both upward and downward, and ultimately, a distance from the source is reached at which the first energy to arrive at a particular receiver has been returned from layer 2. The point at which this happens depends upon the thickness of layer 1, the direction and magnitude of the dip of the interface between the layers, the velocities of the layers, and the length of the receiver array. In the sketch, the crossover is arbitrarily shown to occur at receiver number 5 and all subsequent receivers will also receive their first energy from layer 2. The time of activation of the energy at the source, and of energy arrival at each receiver are recorded, as is the distance from source to receiver. These data are then plotted to give travel-time curves such as shown in Figure 3. A single equipment set-up as shown in Figure 2 will provide the two line segments sloping upward toward the right. The slopes of the two straight-line segments have units of inverse velocity corresponding to the velocities of the two layers. By reversing the profile, that is, by setting off
Figure 3. Reversed profile on southeast side of Barrow spit parallel to beach (approx. 12 m to beach).
a shot at each end of the receiver array, the two additional lines which slope upward to the left are obtained. The absence of symmetry in this plot in Figure 3 indicates that the interface between the layers is not horizontal. The dip angle of this boundary, the true velocity in layer 2, and the depth to the refracting horizon can be calculated from the data shown.

It is assumed that the layers are composed of homogeneous materials so that the velocity is the same at any location in each layer. A simple example of this figure applied to permafrost would be the case of a sandy material that is not frozen near the surface, but is frozen at some depth. The velocity in the frozen material is significantly different than the velocity in the non-frozen material, and this is the key to the detection of permafrost by seismic refraction methods. Note that if layer 2 is not entirely frozen, but only partially so, then the average velocity in layer 2 will be somewhat higher than the velocity in layer 1, but not as high as if layer 2 were completely frozen. As the volume of frozen material in layer 2 is further decreased, its velocity will approach that of layer 1 and in the limit it would be impossible to detect a boundary, and the two layer model no longer applies. Thus, we see that the technique is an averaging technique and it is therefore possible to have small inclusions of frozen materials in an otherwise non-frozen matrix that are not detected by observation of seismic velocities. These inclusions may be small and uniformly distributed or, provided their size is an order of magnitude or so smaller than the receiver spacing, they may be isolated inclusions yet not detectable.

RESULTS

Figure 4a shows an area on the tip of Pt. Barrow where several seismic lines were run.

Figure 4a. Pt. Barrow area (Area A of Figure 1).
Line G, seen in Figure 4b and approximately 30 meters long, indicates a non-frozen upper layer overlying a frozen lower layer at a depth of approximately 2 meters.

![Seismic refraction lines in the Point Barrow area.](image)

The time-distance plot for line "G" is given in Figure 3. Time in milliseconds is plotted on the vertical axis and distance in meters is plotted along the horizontal axis. The velocity of the unfrozen layer is taken as the average of the two values shown on the plot of 1676 meters per second. The lack of symmetry of the plot again indicates that the boundary between the layers is dipping. Using values of the slopes of the upper curves, it can be calculated that the dip of the boundary is 1.6° to the northeast, and the velocity of the ice-bonded layer is 3138 meters per second. The depths indicated by the z values on the figure are the vertical depths to the top of the ice-bonded layer at the shot points. An auger hole at the southwest end of the line recovered fresh-water ice-bonded permafrost from a depth of about 1.8 meters in close agreement with the depth predicted by the seismic data (see Figure 5). The dispersion of the data points may be due to several factors: lack of known distances between the geophones, scaling errors, irregular boundary between the permafrost and the overburden, and inhomogeneous layers. The last two are probably the most significant.

Velocities determined along this profile are typical of those found along other lines in the area, as shown in Figure 6, a plot of measured velocities along a vertical axis. Note that there is a group of velocities between 1600 and 2000 meters per second and a similar group between 2500 mps and about 3300 mps. The first group indicated corresponds to frozen material. The highest velocity shown is 3300 mps, but most of the
frozen material velocities are between 3000 and 2500 mps. Also, not plotted are a third group of velocities of a few hundred meters per second, which were characteristic of unconsolidated sand and gravel in the surface layer of the barrier islands.

We can summarize the results of the refraction measurements near Point Barrow by referring to Figure 4b. Lines A, B, C, G, F, E, and L on the lagoon side gave low velocity upper layers and high velocity lower layers and, as noted above, our augering indicates the low velocities are non-frozen coarse sands while the high velocity lower layers are frozen sands. Line D, a 137 meter long line run toward the southeast into the lagoon, gave a velocity along the path of 1800 meters per second, typical of unfrozen material.

Using an assumed velocity of 2500 mps for frozen material, a conservative estimate, we can calculate the minimum depth to any continuous horizontal frozen layer along line D as approximately 30 meters. This is in marked contrast to the observed frozen layer at a depth of approximately 2 meters on shore. Further, assuming a dipping planar interface between the frozen and unfrozen material extending under the lagoon, and using the velocity values just given, it can be shown that if such a surface exists it is dipping...
Figure 6. Seismic velocities of the refraction layer (taken from Table I, Column 4). Group I is indicative of non-ice bonded materials while Group II is indicative of the ice-bonded permafrost.
into the lagoon at no less than 30 degrees. The depth to the top of ice-bonded permafrost would then be about 80 meters at the end of line D, 137 meters from shore.

Alternative configurations for the frozen-unfrozen boundary in this area are suggested by the results from shots along lines E and B. Line E is a revised profile 182 meters long with a velocity of about 2700 mps for the frozen layer. Along this line seismic energy was almost completely attenuated at distances beyond 120 meters from the shot point. As discussed by Hunter, this suggests that the ice-bonded layer is underlain by a zone of material with a lower velocity, which in this case implies that it is not frozen solid. If this is true, then the dip of the surface at the top of the ice-bonded layer may reverse at the shore of the lagoon and dip back under the land.

Line B was about 80 meters long with velocity typical of frozen material. There was similar attenuation at the last receiver only, so that it is possible the relationships implied along line E also hold here, but we hesitate to conclude this without first re-shooting the line.

These possible configurations of the interface are shown in Figure 7. Note that if the attenuation along line B is accepted, then the boundary would extend back under the shore so that the ice-bonded permafrost body would assume a lens shape.
Returning to the location map of Figure 4b, line H is a 728 meter long refraction line run beneath Elson Lagoon. The average velocity observed on this line was 1770 m/s, a velocity characteristic of non-frozen material. Applying the parallel interface calculation, we obtain a minimum depth to a hypothetical frozen layer along this line of 160 meters. We conclude that although sub-bottom temperatures below the freezing point have been reported to depths of 12 meters in the west end of Elson Lagoon by Lewellen these temperatures are not associated with continuous ice-bonded permafrost. Although isolated ice-bonded permafrost bodies cannot be ruled out along line H, it is apparent that only a small percentage of the 728 meter long line could contain bonded material. Note the wavy line extending northerly from Elson Lagoon to the Beaufort Sea in the upper right hand part of the figure. West of this line the surface consists of small patches of tundra, mudflats, areas of gravel with scattered clumps of grass, and with frost polygons common throughout. To the east of the line, however, the surface consists only of sand and gravel, typical of the rest of the spit which extends to the east. The line thus appears to represent the boundary between an old land area to the west and the southerly migrating sands and gravels of the spit.

Line J on the spit is a reversed profile which indicated non-frozen materials to a depth of at least 48 meters. Assuming this to be characteristic of the spit environment, we anticipated that the wavy line would also mark the eastward limit of ice-bonded permafrost at shallow depths. To test this hypothesis, line I was run at a right angle to the assumed boundary. The results are shown in Figure 8.

Line I was 210 meters long, and was composed of 7 continuous 30 meter lines run with a hammer seismograph. The slide shows the velocity below the first refracting horizon encountered in each of these lines. The abrupt change in velocity at 110 m is obvious and does appear to indicate an eastward limit of shallow, ice-bonded permafrost.

Finally, refraction lines were run along one of the Tapkaluk Islands; Area B, Figure 1. The results of a profile 230 meters long, run along the axis of the island, are shown in Figure 9. The velocity of the refracting layer was 1700 m/s, a velocity associated with non-bonded materials. Thus we conclude there is no continuous ice-bonded permafrost within about 50 meters of the surface of the island at that location. Further, this island along with the area east of Point Barrow discussed earlier is representative of the barrier island environment. Accordingly, it seems reasonable to conclude that there is no continuous shallow ice-bonded permafrost in the environment, where shallow is taken to mean less than, say, 50 meters.

SUMMARY

Our observations in the vicinity of Pt. Barrow have not indicated continuous ice-bonded permafrost beneath the west end of Elson Lagoon nor along the barrier islands. However, Barrow Spit was found to be underlain by continuous ice-bonded permafrost. These results must be weighed with an eye to the averaging nature of the technique and the observational depth limitations inherent in the line lengths used.

REFERENCES


Acknowledgements - This work was sponsored by the Alaska Sea Grant Program, supported by NOAA Office of Sea Grant, Department of Commerce, under Grant Number 04-5-158-35.
Figure 8. The seven seismic lines that make up line I of Figure 4b. Line 11-11 is the most western portion while Line 11-9 is the most eastern portion of Line I. The approximate transition from vegetated to barren ground is seen to coincide approximately with the transition from high velocities (frozen) to low velocities (not frozen).
TAPKALUK IS.

Figure 9. Composite reversed profile in the Tapkaluk Islands.

V = 1654 m/sec
T = 0.0060 + 0.000604 d
S.D. = 0.025

V = 1727 m/sec
T = 0.0055 + 0.000579 d
S.D. = 0.017

Z = 20 m
SECTION 11

NORTHERN OIL AND MINERAL

Etchegary, L. M. and W. Hindle
Polar Gas Project: Natural Gas Pipelines in the Arctic Environment - Engineering Research
(TransCanada Pipelines, Ontario, Canada)

O'Donnell, J. P.
Pipelines and the North Sea
(Pipeline Consultant, Texas, United States)

Palmer, H. E. and W. Hindle
Polar Gas Project: Natural Gas Pipelines in the Arctic Environment - Environmental Research
(TransCanada Pipelines, Ontario, Canada)

Simpson, O. G.
Unique Problems of Oil Development in the Arctic
(Atlantic Richfield Company, Alaska, United States)

Splettstoesser, J. F.
Mining in Antarctica: Survey of Mineral Resources and Possible Exploitation Methods
(University of Nebraska, United States)

Strain, H. J.
Offshore Drilling from Artificial Ice Platforms
(Panarctic Oils Ltd., Alberta, Canada)
POLAR GAS PROJECT  
NATURAL GAS PIPELINES IN THE ARCTIC ENVIRONMENT  

PART I  
"ENGINEERING RESEARCH"  

Walter Hindle  
Group Vice-President and Executive Engineer  
TransCanada Pipelines  

and  

L. M. Etchegary  
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ABSTRACT  

The Polar Gas Project is one of the most challenging and fascinating undertakings being contemplated in the world today. It was formed in late 1972 with the mandate to plan the research and engineering for the transportation of large volumes of frontier natural gas from the Canadian Arctic Islands to markets in Canada and the United States.


It can be seen from the reference map (Fig. 1) that the main sources of gas are located well within the Arctic Circle in two general areas; namely, the Sabine Peninsula on Melville Island and King Christian and Thor Islands lying off the western side of Ellef Ringnes Island.

According to the latest published report of the Geological Survey of Canada, ultimate recoverable reserves of natural gas are estimated to total some 240 trillion cubic feet. To date, over 13 trillion cubic feet of these reserves have been established, an amount fast approaching the minimum required threshold volumes of 20 to 30 trillion cubic feet.

The challenge facing Polar Gas is to devise a means of economically moving trillions of cubic feet of Arctic Islands natural gas reserves up to 2,000 miles southward and across up to 170 miles of Arctic ocean channels while minimizing impact on social or biophysical environments.

Research programs are currently underway, to obtain answers to the remaining technological and environmental questions needed to enable feasibility determination and detailed planning, for example, is developing a number of methods which would permit large diameter pipe to be laid during the winter, across channels averaging 600 feet in depth.

Polar Gas currently plans to file regulatory applications to construct the line in 1977. This date could be advanced should threshold gas reserves be established as a result of 1975 drilling programs in the Arctic Islands.

NOTE: The author has requested this paper be read in conjunction with the related paper, "Part II - Environmental Research", by W. Hindle and H. E. Palmer (p. 1109).
Figure 1. Polar Gas reference map.
THE PIPELINE SYSTEM

In investigating a pipeline to deliver natural gas from the Arctic Islands, Polar Gas is facing a unique technological challenge -- the construction of deepwater channel crossings in the Arctic Islands.

Faced with the challenge of major pipeline crossings across Arctic Island channels frozen for as much as 11 months of the year, Polar Gas construction engineers developed a unique approach -- one which was based upon building the pipeline during a four-month winter construction schedule. The intent is to utilize the strength and stability of the ice as an extension of the land surface whenever possible, preferably by modifying conventional pipeline laying techniques (Fig. 2).

![Figure 2. Frozen channel.](image)

The proposed Polar Gas pipeline system would probably be constructed in two phases; the first running south from Melville Island and the second extending north at a later date to other discoveries in the area of King Christian and Ellef Ringnes Islands. Depending on routing, the system will involve some 2,200 to 3,200 miles of up to 48-inch diameter pipe to deliver in the range of 2 to 4 billion cubic feet of gas per day. Marine crossings will comprise a minimum of two lines of up to 36-inch diameter pipe.

The major emphasis at this time is directed towards a 48-inch diameter pipeline running from Melville and King Christian Islands down the west side of Hudson Bay to markets in central Canada, which would take maximum advantage of the available economies of scale. With a capital requirement of 7½ billion 1974 dollars, this system would require reserves in the order of 30 trillion cubic feet to support a minimum throughput of approximately 3 billion cubic feet per day.

An alternative route does exist via east Hudson Bay; however, this would entail an additional 170 miles of water crossings, an average of 630 feet deep, which, due to insufficiently strong and land fast ice, would have to be constructed using lay barge techniques. It is probable that even if a marine pipelaying system were to be designed that could...
cope with the water depths, currents and ice problems of the northern Hudson Bay area, the result could be delayed timing and substantially greater cost for the Project (Fig. 3).

Figure 3. Proposed alternative routes.

A smaller 42-inch system from Melville Island, connecting with TransCanada's system near Winnipeg, is also being considered. The minimum throughput level for such a system would be in the order of 2 billion cubic feet per day, with capital requirements of approximately 4½ billion 1974 dollars and threshold reserve volumes of about 20 trillion cubic feet.

Total compression requirements for a fully powered system would approach the 3 million horsepower level. System optimization studies of the compression facilities along the line are in progress with the planning for compressor stations being directed towards the use of turbines of high thermal efficiency (about 33 percent) in the 20,000 to 35,000 horsepower range. In the continuous permafrost zone in the north, it is generally planned to cool the gas to 27 degrees F. by refrigeration systems and in the south, outside permafrost zones, aerial cooling systems will be used to maintain discharge temperatures of below 100 degrees F.

Low compression ratios will be used for stations adjacent to channel crossings so that unit additions can be made in order to increase the throughput capacity of the marine pipelines in the event of loss or delay in the installation of one of the lines.

ROUTE DESCRIPTION

The Canadian Archipelago lies north of the Canadian mainland to the polar ice shelf between Alaska and Greenland, comprising approximately 500,000 square miles. Three of the islands found in the Archipelago are among the 10 largest in the world.

As previously outlined, the main gas supply sources are located on Melville Island, King Christian and Thor Islands. Lateral lines from these areas will be constructed to meet
at a junction point on either Bathurst or Cornwallis Island and from this point, the main line will proceed via the west or east Barrow Strait to Somerset Island and then overland to southern markets.

Contrary to popular belief, the islands are not perpetually ice-bound. In fact, less than 8 percent of the land area is permanently ice covered. The central and western islands form a frozen desert with low relief and from 2½ to 5 inches of annual precipitation. In the east, Ellesmere, Axel Heiberg and parts of Devon Island are mountainous with higher elevations covered by permanent glaciers.

For one to three months in the summer, more than 90 percent of the land is bare of snow. Temperatures range from 70 degrees above zero on the odd day during the summer to more than 60 degrees below zero in the winter. Example of mean temperatures at Resolute Bay on Cornwallis Island are minus 28.5 degrees F. in February and plus 40.3 degrees F. in July.

The ice increases in thickness to the end of May but breaks up sufficiently in the east to permit ocean shipping with icebreaker support from July 15th to October 15th. Ice conditions become progressively worse to the west. Polar Gas' Rea Point base camp is generally accessible by sea between August 25th and September 30th (Fig. 4).

Surface characteristics vary considerably along the route, as might be expected with the pipeline passing through several physiographic regions. In the Arctic, the land surface consists mainly of sedimentary bedrock outcrops and unconsolidated materials. From Somerset Island to Chesterfield Inlet, the terrain is rough and hilly with a high frequency of bedrock outcrops. Permafrost and glaciation characteristics of this region are much in evidence (Fig. 5).

South of Chesterfield Inlet, the line follows the margin between the wave-washed sediments which extend eastward to Hudson Bay and non-marine glacial deposits to the west. At the Seal River, located in north Manitoba, the zone of discontinuous permafrost is entered; this also marks the beginning of the tree line. In some areas of northern Manitoba, ponds
and small lakes account for about 70 percent of the surface area. Alternating wet and rock depressions persist further south into southern Ontario.

These terrain conditions are well known to Polar Gas through the experience gained by TransCanada Pipelines. The latter company has constructed large diameter pipelines during very cold weather and through rock and muskeg. In fact, TransCanada has actually been able to take advantage of such conditions to lay pipelines faster and more economically than in the summertime (Fig. 6).
INTER-ISLAND CROSSINGS: RESEARCH AND INVESTIGATION PROGRAMS

In order to demonstrate the technical feasibility of constructing the channel crossings, it has been necessary to collect and analyze extensive data. To date, three major engineering research and investigation programs have been conducted as follows:

a) An ice research program in the Spring of 1973;
b) A marine research program in the Summer of 1973; and
c) A second-stage ice research program in the Spring of 1974.

A. Ice Research Program - 1973

The principal objective of the 1973 programs was to begin collecting ice and bathymetric data relevant to pipeline routing and design, and to prove the feasibility of using the ice as a support platform for winter marine pipelaying operations, including verification that a conventional form of ditching machine could be used to excavate a trench across the sea ice. Such a trench is a prerequisite for several possible crossing techniques and in total, 30 miles were successfully trenched during the trials.

The crossings chosen for this research were the Byam Channel (Melville Island to Byam Martin Island) and Austin Channel (Byam Martin Island to Bathurst Island) (Fig. 7).

![Map of channel crossings.](image)

Weather conditions during the program were typical of those which could be expected. In both April and May, the combination of temperature and wind produced wind-chill temperatures of minus 80 degrees F. These, however, are comparable to those experienced in pipelining across Northern Ontario.

In March, 1973, a base camp for the program was established at Rea Point, Melville Island, near the base camp of Panarctic Oils (Fig. 8).

During the month, men and equipment were flown to the site from Yellowknife, N.W.T., in preparation for trenching, which began in mid-April.
The equipment used for the trenching was basically standard land pipelaying trench equipment except that the diameter of the bucket wheel was increased to provide a capability for cutting ice up to a thickness of 15 feet.

When excavating through 6½ foot thick ice, (the average thickness of sea ice) the maximum progress achieved by the machine in a twenty-four hour day was approximately 2 miles (Fig. 9).

Figure 8. Polar Gas and Panarctic Oils base camps.

Figure 9. Ditching machine cutting a trench through the ice.
While these trenching operations were in progress, several other research and investigation activities were being carried out. They included, for example:

a) Ice thickness measurements in several channels (Fig. 10);

b) Ice strength tests designed to provide basic parameters for further laboratory testing and analysis. These were full-scale load tests using large water tanks (Fig. 11);

c) Bottom sampling with piston gravity corers (Fig. 12);

d) Current measurements (Fig. 13);

e) Bathymetry, sub-bottom profiling and side-scan sonar observations through the trench (Fig. 14);

f) Scuba diving observations of the channel bottom to water depths of up to 200 feet (Fig. 15); and

g) Environmental observations (Fig. 16).

Figure 10. Augering through the ice to measure thickness.

In general, the results obtained from this overall program were extremely encouraging and established the feasibility of cutting trenches through sea ice with only minor modifications to conventional equipment. Moreover, the data obtained during the program served as an excellent base for continuing research.

B. Marine Research Program - 1973

This program was designed with the main objective of providing marine data for the design of potential pipeline crossings in the Arctic Islands and northern Hudson Bay. The major part of this program was carried out in August, September and October from ice strengthened, 2400 gross ton vessel, the Percy M. Crosbie. The vessel had a total complement of 53 research staff and crew members (Fig. 17).

Prior to commencing the program, the vessel was modified to meet the special requirements of the survey work to be undertaken. It was also equipped with two specially built 40 foot aluminum survey launches, helicopter and helicopter landing pad.
Figure 11. Testing ice loading characteristics. Large tanks are filled with water to measure vertical deflection of ice.

Figure 12. Collecting core samples.
Figure 13. Vehicle and equipment used to perform bottom profiling and side scanning of ocean floor.

Figure 14. Installing current meters.
Figure 15. Scuba diver.

Figure 16. Environmental observations on ocean floor.
The launches were designed to handle the sophisticated electronic equipment required for bathymetry, sub-bottom and side-scan sonar data as well as the precise navigational systems required for research of this type (Fig. 18).
Aided by this equipment, the following detailed activities were undertaken:

a) Acoustics depth soundings to determine bathymetry and bottom roughness;

b) Seismic profiling to determine sub-bottom stratification;

c) Side-scan sonar surveys to detect ice scour, reefs, outcrops and other obstruction which could interfere with the pipeline;

d) Current metering, wave recording and tidal observations; and

e) Collection and analysis of bottom samples.

The Percy M. Crosbie travelled 15,000 miles during the program and obtained data at selected locations in 19 different channels in the Arctic Islands and Hudson Bay areas. The program was aided by an excellent season from the standpoint of the great amounts of open water available for research. However, this was offset by a more than normal amount of fog which caused difficulty in the use of air-borne transportation of personnel to support the program.

Of particular interest was an independent air-transportable system which was put together to collect similar basic data on short notice in channels which only became ice-free for short periods and which were inaccessible to the major vessel because of the intervening ice conditions. The system comprised three 17-foot aluminum launches coupled together, each being sufficiently light for transportation from location to location by means of a small helicopter.

Ice reconnaissance patrols were also carried out from both helicopters and fixed-wing aircraft. In addition to providing short-term ice forecasts at designated crossings, valuable records were obtained of the sea ice conditions prevailing in the channels during the break up season (Fig. 19).

![Figure 19. Air-transportable survey vessel.](image)

The data obtained during the Percy M. Crosbie's voyage included approximately 800 miles of bathymetry, sub-bottom profiling and side-scan sonar records. Most of the data collected during this highly successful program has been analyzed and it formed the basis for
technical assessment studies of pipeline installation methods which have been carried out on selected channels. These marine studies also highlighted the gaps in present knowledge and helped identify areas where research and investigation should be concentrated in the future.

C. Ice Research – 1974

Though this was the general name of the program, it encompassed far more than the title suggests and included work on bathymetric and sub-bottom data collection from the ice. This is another area of innovation of our program at Polar Gas. In the past, as far as is known, such investigations on a production type basis have, up to this time, only been carried out in open water conditions.

Our general objectives in this program were briefly as follows:

a) To conduct a research on ice strength in order to confirm and evaluate parameters predicted by laboratory tests and mathematical model computer studies.

b) To research methods of preparing a right-of-way across the ice surface.

c) To collect ancillary data on currents, bottom soils, ice thickness, ice movements and ice scour.

d) To develop and test directly from the ice surface and obtain data in Byam Channel (Melville Island to Byam Martin Island) and West Barrow Strait (Bathurst Island to Prince of Wales Island).

For bathymetry and sub-bottom research programs, a variety of sophisticated equipment was used which enabled us to acquire both data simultaneously through the ice. They are:

a) A 3.5 Khz high resolution system placed in a helicopter which was fitted with precise navigational equipment. Readings were taken at 300-meter intervals to provide quick evaluation of the bottom profile (Fig. 20);

b) A similar high resolution system was placed in an enclosed housing on a large tracked vehicle which took readings at 15-meter spacings (Fig. 21);

c) A Sparker 1500 Hertz system was placed in another enclosed housing on a large tracked vehicle which provided more detail for the interpretation of sub-bottom profile in sand and/or rock areas (Fig. 22); and

d) A precise navigational system placed in a small totally enclosed tracked vehicle which was fitted with flotation tanks as a safety measure (Fig. 23).

Information from all these systems was collected on magnetic tape and fed into a computer housed at the Rea Point camp. This enabled the bathymetric data to be accurately plotted within hours of the data arriving from the field. It was then checked by a pipeline design engineer to assess its suitability for pipelaying operations, particularly as to expected stresses induced in the pipe by the bottom roughness. A total of 62 miles of detailed bathymetric data (bathymetry profile, sub-bottom profile and sub-bottom analyses) was collected in this manner during the program. A complementary program of collecting data, consisting of both through the ice and marine, provided maximum flexibility in bathymetric research.

The ice strength testing part of the program basically comprised a continuation of the initial work started in the previous year. Eighteen full-scale load tests with water tanks and 89 cantilever tests, under both long and short term loading conditions, were carried out.
Figure 20. S-62 helicopter fitted with resolution system and navigation equipment.

Figure 21. FN-110 Nodwell fitted with resolution system.
Figure 22. FN-110 Nodwell equipped with Sparker system.

Figure 23. Navigation system in closed tracked vehicle with flotation tanks.
For the right-of-way preparation part of the program, the work included:

a) Observations on the rate of build-up of snow on cleared ice surfaces;
b) Thickening an ice sheet by flooding with sea water to obtain a stronger and smoother surface (Fig. 24);
c) Evaluation of the effects of snow fencing as a means of keeping the ice surface snow free;
d) Raising the surface of the ice sheet by adding compacted snow in order to assess the effectiveness of keeping the surface free from drifting snow by wind action; and
e) Measurements of the tensile forces which can be sustained by anchors frozen or mechanically attached to the ice surface (Fig. 25).

Figure 24. Flooding the ice with sea water.

Ancillary data collected during the overall program included such items as current velocities, bottom soils samples, ice thickness measurements, ice movement and ice scour observations. The techniques used to obtain these data were essentially the same as those employed during the previous year, with minor modifications and the addition of new pieces of equipment to improve efficiency.

Most of the data collected during the programs has now been analyzed and is being utilized in the on-going technical assessment studies of a number of pipeline installation techniques for Arctic channels.

INTER-ISLAND CROSSINGS: CONSTRUCTION TECHNIQUES

Depending on the width, depth, ice conditions and other characteristics of the channels to be crossed, a number of pipelaying methods could be required. In general, however, they fall into two basic categories:

1. Those using the ice as a platform from which to lower the pipe (somewhat in the manner of a lay barge) (Fig. 26), and
2. Those based on the conventional "bottom pull" technique - either from the ice surface or directly from one shore to the other (Fig. 27).
Figure 27. Model showing "bottom pull" from the ice technique.

An additional option for one of the channels which could be faced is to construct the pipeline crossing using a ship to pull the pipe during the summer season. As currently conceived, the "bottom pull" through the ice system would involve the building of a series of thickened ice platforms or "ice islands" from which winches will pull the pipe assembled on shore. This method could involve one or more "tie-ins" or welds of several mile long sections of pipe which have been pulled into position. The technology to carry out "tie-ins", working in chambers lowered to the bottom of the channel or by lifting to the surface the two ends of pipe to be joined, is available.

On shorter channel crossings, the "bottom-pull" shore to shore technique would involve moving the pulling station from the surface of the ice to the opposite shore.

OVERVIEW

The engineering research activities of the past three years have been undertaken with the main aim of confirming the technical feasibility of the pipeline. The results to date have confirmed, to a high degree of confidence, that a natural gas pipeline from the Arctic Islands is technically and economically feasible.

In the future, research programs from a technical standpoint, will be directed towards terrain and metallurgical studies, route optimization analyses, the refinement of construction methods and techniques and data collection for final pipeline design. Needless to say, these programs present additional opportunities for innovation and improving existing technology.
Laying pipelines in the North Sea becomes increasingly difficult as operations move northward. The first North Sea discoveries were made just north of the 51st parallel. The most recent have been north of the 61st parallel.

The move northward has been marked by greater water depths, more severe surface conditions, greater distances from markets and, most significantly, by more prolific fields.

Five "giant" fields, i.e., those with oil reserves of more than one billion barrels, have been found in the North Sea. Three of these, including Statfjord, the largest North Sea discovery, are in the vicinity of the 61st parallel.

The North Sea's far north fields are generally at depths of about 500 feet. In a few instances, depths well below 500 feet have been encountered on some pipeline routes.

Most of the Northern fields are in the East Shetland Basin, an area 90 to 120 miles east and north of the Shetland Islands. Storms that can generate winds of 80 mph and seas of more than 20 feet can build up in this area in as short a time as 4 1/2 hours. This area is also exposed to powerful swells rolling in from the north Atlantic.

The longest pipeline in the southern part of the North Sea is the 86 mile, 28 inch line from the Viking field to Theddlethorpe in Britain's east coast. By contrast, the 34 inch crude oil line from Ekofisk to Teesside, England is 220 miles long and the gas line, 36 inches, from Ekofisk to Emden, West Germany is 268 miles long.

Both of these lines are shorter than the projected 300 mile natural gas line from the Brent field to Peterhead, Scotland. That line, in turn, will be dwarfed by still another proposed gas pipeline that would stretch 600 miles from the Statfjord field to West Germany.

The ability of available offshore pipelaying equipment to meet these conditions is under question. As a result, contractors are building larger and more sophisticated lay barges which, they believe, will be able to contend with the more severe conditions now being encountered.

Two totally new pipelay vessels that exemplify this development are making their debuts this year. One is the E.T.P.M. 1601, a pipelay ship, and the other is the Jersey Piper,
The 1601's size will contribute to its seaworthiness and will give it two other advantages. Its length will enable it to handle double-jointed pipe and thus make it possible for it to move ahead in 80 foot rather than in 40 foot increments. Its deck space will enable it to carry a much larger volume of weight-coated pipe and thus lessen its dependence on supply vessels.

The Jersey Piper will have the same advantages as the 1601 plus several others. Its semi-submersible design is credited with making it capable of operating in 15 foot seas. In addition it has two distinctly peculiar advantages:

1. It has a retractable stern ramp which replaces the conventional stinger. When conditions become sufficiently severe to halt pipelaying operations, the ramp is retracted to wait out the bad weather. On a conventional barge, the stinger is abandoned at the onset of foul weather and recovered and rehitched when the weather moderates. This can be a very time consuming operation.

2. It will have a computer assisted, 14-point mooring system which is very sensitive to vessel motion. The mooring system includes unusually long anchor cables which will lengthen the interval between the repositioning of anchors.

Several other lay barges of approximately the same size as the 1601 and the Viking Piper are under construction. Their cost will approach $100 million, several times that of their predecessors, which will result in much higher operating costs.

A revolutionary type of pipelaying vessel may see service in the North Sea in a couple of years. This is the Santa Fe International Apache, which would be the world's first self-propelled, reel-type pipelaying ship. It would be capable of laying continuous strings ranging from 99 miles of 8 inch to 7 miles of 34 inch in water as deep as 3,000 feet. It could be the vessel that will lay the first line across the 1,200 foot deep Norwegian trench.

New types of bury barges, supply vessels and service ships are also making their appearance in the North Sea.

Two new bury barges are using reels to mount their high pressure hoses. One is using the eductor principle to dispense spoil. Another, using a "fluidization" principle, is proving effective in very sandy sea bottoms where it is impossible to maintain ditch walls.

The rigors of the North Sea made it impossible to supply lay barges with line pipe from barges. It is being done with fast, powerful, highly seaworthy supply vessels now. Since their loads are limited, a supply vessel capable of carrying 10,000 tons on deck, is now in the design stage. An unusual hull would give it superior motion characteristics. It would also be capable of laying pipe.

An unusual service ship, the "H.T.S. Coupler I", was designed for the sole purpose of making underwater repairs. It is equipped for saturation diving, has heavy lifting equipment which can be operated through a moon pool or over the side and hydraulically operated alignment frames equipped with tools that may be operated from the surface.
Most of the cost undertaken to meet North Sea pipelaying problems has gone into new vessels, particularly the new lay barges. Considerable sums, however, are going into other areas as well.

These include higher quality line pipe, upgraded tensioning devices, improved stingers, automatic welding, very close control of weight coating, the use of buckle detectors and buckle arresters, etc. All have been undertaken with one objective: To do a better job in less time.

North Sea costs are very high. A conventional lay barge operating in the relatively mild southern area costs around $120,000/day. The cost of replacing a buckled joint may range from $1 million to several million. The cost of the proposed 300 mile Brent gas line is estimated at $1 billion and that of the proposed Statfjord, West Germany line, $2.5 billion.

Sea and weather conditions combine to interrupt North Sea pipelaying operations frequently and, at times, for long intervals. Because of the extremely high costs, taking maximum advantage of favorable conditions is critical in North Sea operations.

ABSTRACT ONLY AVAILABLE
Since 1974 Polar Gas has been undertaking field studies of the High Arctic (north of Spence Bay) to assess the environmental implication of building a chilled gas pipeline from fields on the Sabine Peninsula of Melville Island to the mainland of Canada. This research is a part of a phased program of environmental field studies which will eventually cover the entire Polar Gas route from its northern to its southern extremities (Fig. 1).

Field studies were carried out in 1974 on the Boothia Peninsula and on Somerset, Prince of Wales, Cornwallis, Little Cornwallis, Bathurst and Melville Islands under the broad categories of Landscape, Land Mammals, Birds, Fisheries and Marine Ecology. Studies of Land Mammals, Birds, Marine Mammals and general Marine Ecology are continuing in 1975 to further define the important biological parameters of the area. In addition, 1975 field studies of Landscape, Land Mammals, Birds and Fisheries have been extended into the region south of Spence Bay to the Northern border of Manitoba.

This paper outlines some project-related environmental factors presently under consideration in the High Arctic. Although continued refinement of our data base is necessary for detailed problem definition and solution, we presently have a sufficient grasp of the important physical and biological parameters to identify the key areas of interaction between the pipeline and the environment.

NOTE: The author has requested this paper be read in conjunction with the related paper, "Part I - Engineering Research", by W. Hindle and L. M. Etchegary (p. 1085).
ENVIRONMENTAL OVERVIEW

Climate

An extreme cold continental climate prevails over the study area once the channels are frozen over. During winter, temperatures are typically lower than -30°F (34°C) and the weather tends to be calm and clear. In the summer period the weather becomes unstable when the sea ice starts to melt, giving rise to cool maritime weather and the frequent fog and low cloud. Summer temperatures are typically in the 40°F (4°C) range, although they may be somewhat warmer on the Boothia Peninsula.

Precipitation is typically low, the mean annual total precipitation being only 5.35 inches (13.59 cm) at Resolute on Cornwallis Island. About half of the annual precipitation falls as snow, mainly in the months of May, September and October. Normally the land is snow and ice free only in the months of July and August.

Landscape

The study area can be divided into an igneous region which forms the central portion of Boothia Peninsula, the western portion of Somerset Island and the east coast of Prince of Wales Island, and a sedimentary region which forms the remainder of the study area. The landscape can be roughly divided into six broadly defined land systems as follows:

a) Coarse igneous rock uplands with or without frost-shattered rock fragments (Figs. 2 and 3);

b) Coarse sedimentary rock uplands with or without frost-shattered rock fragments (Figs. 4 and 5);

c) Continuous shallow tills and till veneers having gravelly and sandy surfaces with abundant angular rock fragments originating from underlying bedrock (Fig. 6);

d) Deep till and reworked till with rounded boulders of glacial origin (Figs. 7, 8 and 9);
e) Coarse gravelly and sandy deposits (outwash valleys, eskers, kames, hummocky disintegration moraine) (Figs. 10 and 11); and
f) Marine and lacustrine deposits (Figs. 12, 13 and 14).

Figure 2. Coarse igneous rock uplands - Boothia Peninsula (ground view). Land system a.

Figure 3. Coarse igneous rock uplands - Boothia Peninsula (aerial view). Land system a.
Figure 4. Coarse sedimentary rock upland with Felsenmeer - Somerset Island (ground view). Land system b.

Figure 5. Coarse sedimentary rock upland with Felsenmeer - Somerset Island (aerial view). Land system b.
Figure 6. Sedimentary upland with thin till veneer - Somerset Island. Land system c.

Figure 7. Reworked till with typical elongated net pattern of vegetation - Somerset Island. Land system d.
Figure 8. Aerial view of reworked till with typical elongated net pattern of vegetation - Somerset Island. Land system d.

Figure 9. Heavily vegetated till pocket. Land system d.
Figure 10. Hummocky disintegration moraine - south of Sanagak Lake, Boothia Peninsula. Land system e.

Figure 11. Aerial view of hummocky disintegration moraine - Boothia Peninsula. Land system e.
Figure 12. Well vegetated sedge meadow on raised delta - Boothia Peninsula. Land system d.

Figure 13. Closeup of sedge vegetation on raised delta - Boothia Peninsula. Land system f.
The major portion of the route crosses land system types a, b, and c. Land systems d, e, and f constitute only a minor portion of the total landscape.

Vegetation cover on land systems a, b, c, and e ranges from sparse to extremely sparse. Plant cover, excluding crustose lichens, seldom exceeds 5 percent. Much of these areas would be classified as polar desert.

Marine and lacustrine deposits represent the opposite end of the vegetation spectrum. On these, vegetation cover may reach 100 percent. The communities are dominated by sedges and may also contain a significant component of mosses. Occurrence of thermokarst lakes is largely confined to these areas of fined-grained soils.

The vegetation of till and reworked till deposits which lie in areas intermediate between the sparsely covered uplands and the densely vegetated lowlands is the most difficult to characterize. Depending upon such factors as soil humidity, gradient, drainage, aspect, degree of cryoturbation and activity of erosive forces, the composition and percentage cover of vegetation may vary considerably. Lichens, mosses, deciduous shrubs (willows) and evergreen shrubs (dryas, saxifrage) form components of varying importance. Plant cover generally exceeds 20 percent and may reach values greater than 50 percent in moist, protected areas.

Freshwater Systems

The sedimentary portion of the study area contains a few large lakes and only limited areas of extensive thermokarst pond development. The majority of the lakes are shallow enough to freeze to the bottom. The igneous portion of the route contains numerous small and large lakes, many of which are deep enough to have unfrozen water in the winter.

Both sedimentary and igneous regions have numerous streams. Most stream runoff is derived from the annual snowmelt and there is consequently a marked dropoff in streamflow by late
summer. Most, if not all, streams freeze to the bottom but it is possible that some streams, such as the Union River, which are lake regulated may have some winter flow.

A number of lakes were tested in the spring of 1974 for winter dissolved oxygen. It was found that most lakes had adequate oxygen for fish survival. However, a few instances of oxygen depletion were noted.

The bottom fauna of streams and lakes consisted almost completely of species of larval midges (chironomids). A relatively minor component of stoneflies and mayflies gives the bottom fauna of Boothia Peninsula a somewhat greater diversity.

Marine Systems

It will be necessary to cross Byam and Austin Channels to reach Bathurst Island. From Bathurst there is a choice of crossing by a west route via West Barrow Strait, Baring Channel and Peel Sound or by an east route via Crozier Strait, Pullen Strait and East Barrow Strait.

The Channels of the east route break up earlier than those of the west route. In some years, such as 1974, Barrow Strait may open as far west as Griffith Island, as early as March or April. In more typical years such as 1975, a stable ice front forms from the north-east corner of Somerset to the south shore of Devon Island and the deterioration of ice in East Barrow is a more gradual process taking place in June and July. Breakup of the ice on West Barrow, Austin and Byam Channel usually occurs late in the summer.

The presence of ice cover is a major factor influencing primary productivity. An important phytoplankton community utilizes the concentrated nutrients and limited light at the water-ice interface. Large zooplankters (amphipods) graze on the phytoplankton at the water-ice interface and are in turn grazed upon by ringed seals and seasonally by ducks and seabirds (through ice leads).

Terrestrial mammal fauna is extremely simple, consisting of a total of only nine species for the whole study area. The following species are present; caribou (peary and barren ground), muskox, arctic ground squirrel, brown lemming, arctic fox, arctic hare, collard lemming, arctic wolf, and ermine (Figs. 15 through 18).
Figure 16. Adult cow muskox - Bathurst Island.

Figure 17. Lemming.
Species diversity is lowest on Somerset Island with six species and highest on Boothia Peninsula with eight species.

Suitable mammal habitat is restricted to those areas where soil and moisture conditions permit the development of significant vegetative cover. Such conditions are usually found in areas of deep till, marine and lacustrine deposits. Within suitable vegetated areas, the thickness and physical properties of the snow cover greatly affect the availability of vegetation to grazing mammals. Consequently, windblown vegetated slopes and south-facing slopes are important winter and spring habitats (Figs. 19 and 20).

Marine Mammals

The channels of the study area are frequented by ringed seals, bearded seals, harp seals, beluga whales, narwhal, polar bears, and, to a small extent, by walrus. Ringed seals and polar bears (inhabiting wide areas of fast ice), are permanent residents of the study area. The remaining marine mammals migrate into the area during the open water period. Their movements are greatly influenced by highly variable breakup events.

Perhaps the most spectacular phenomenon involving marine mammals is the annual summer gathering of thousands of beluga whales and their calves in estuaries such as Cunningham Inlet (Fig. 21).

Birds

From May to August, many birds utilize terrestrial, freshwater and marine environments during some part of their life cycle.

In the spring the most important areas for birds are the early leads which open up in the sea ice. During this period, large concentrations of snow geese, elder ducks and oldsquaw and a variety of seabirds utilize the limited available habitat. During the nesting season the limited areas of lacustrine and marine deposits with their associated thermokarst
Figure 19. Peary caribou on well vegetated river valley slopes - west coast of Somerset Island.

Figure 20. Muskox grazing in wet depression with sedge vegetation - Melville Island.
development are critical for nesting waterfowl and shore-birds. After nesting, waterfowl concentrate for moulting, at which time they are flightless. Our information on moulting birds in the high arctic is presently incomplete and the subject is being pursued intensively during the 1975 program (Fig. 22).

Figure 21. Concentration of beluga whales - Cunningham Inlet.

Figure 22. Concentration of 400 moulting snow geese in thermokarst pond - Creswell River area, Somerset Island.
Seabirds require cliff nesting sites close to marine areas where there are consistently recurring open leads, while peregrine falcons generally utilize cliff-nesting sites along streams. The numbers of important cliff nesting sites are limited and many of these have already been identified.

The general statement can be made that very limited parts of our study area are capable of supporting abundant bird life. As a result there is a very strong "oasis effect" and a judicious combination of landscape analysis and field checks of promising looking areas usually enables a fairly precise pinpointing of such sites.

Fish

Arctic Char is the most significant species in the study area. In the islands it is the sole species present, but on the Boothia Peninsula it is joined by lake trout and nine-spine stickleback (Fig. 23).

![Figure 23. Arctic Char.](image-url)

It is known that arctic char must overwinter in freshwater as they are not capable of enduring winter seawater temperatures less than 32°F (0°C). Some char spend their entire lives in freshwater, whereas others spend their summers in the sea and return to freshwater in late summer. Our study area contains only a few streams with sufficient late summer flow to sustain anadromous runs. Significant exceptions are the Union River on Somerset Island and the Lord Lindsay River on Boothia Peninsula. Most char found in the study area are of the landlocked variety.

All char, whether anadromous or landlocked, require lakes of sufficient depth to prevent freezing to the bottom. This allows for the survival of fish and eggs. Such lakes are fairly frequent on the igneous portion of the Boothia Peninsula but are rarer on the sedimentary Arctic Islands portion of the study area.
APPROACH TO ENVIRONMENTAL PROTECTION

We are using a rather simple but workable approach to environmental protection consisting of four steps which can be summarized as follows:

1. Identify the nature of potential disturbances;
2. Determine the locations where potential problems may exist;
3. Where feasible, modify routing, scheduling or procedural measures to avoid potential problems; and
4. If potential problems cannot be totally avoided, carry out further research to determine the magnitude of the problem and measures to minimize disturbance.

ENVIRONMENTAL CONCERNS

Landscape

Landscape disturbance factors which must be considered include:

a) Location and design of access roads, right-of-way and permanent facilities.
b) Excavation and backfill operations.
c) Effect of vehicular traffic.
d) Alteration of subsurface drainage by the frost bulb of an operating gas line.
e) Long term erosion problems.
f) Disposal of solid waste.

All these concerns overlap with basic geotechnical investigations and engineering design. In many cases the optimum engineering design results in the optimum environmental design. However, this is not always true because the environmental input introduces the added complexities of biological, cultural and aesthetic factors.

In most cases the types of foreseen changes in the landscape would not likely have a significant influence on the functioning of ecosystems.

Freshwater Systems

The basic objectives is to maintain the freshwater environment in such a state that it will continue to support the existing fauna. The principal concerns that have been identified are:

a) Siltation;
b) Pollution;
c) Excessive water extraction (from fish overwintering areas);
d) Snow removal (causing freezing of fish overwintering areas); and
e) Blockage of winter flows.

As indicated in the landscape section, some long term siltation effects would be possible if pipeline construction caused significant terrain instability within the drainage area of a lake supporting overwintering char. Our general analysis to date has indicated that the problem will be rare.

Siltation of streams will occur as a short term effect during actual stream crossing and it could occur as long term problems if pipeline activity promotes terrain instability. Siltation is not considered as a major problem in the vicinity of our routes. Siltation could result in localized changes in relative abundance of larval midges (chironomids) which form the bottom fauna. This would not necessarily have any major effect on productivity. Regardless of the theoretical effects of increased siltation, the actual increase in stream siltation would likely be minimal because of the engineering precautions that would be taken to minimize cases of severe erosion.

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We do not foresee any serious problems with pollution from poisonous substances. There are few probable uses for acutely toxic substances. Bulk storage of fuel for equipment and methanol for pip testing (if used) will be protected by diking and drainage control. Fuel transportation risk can be minimized by prearranged procedures for containment, recovery and disposal.

Water extraction from small lakes which are just barely deep enough to support char should be controlled to avoid damaging drawdowns and possible freezing to the bottom. Similarly, snow removal from such lakes should be minimized to prevent increasing the depth of ice cover. Potential problems areas will be easily identified with a minimum of field work at a more advanced stage of the engineering design.

In the likely small number of streams with winter flows, care must be taken to ensure that the frost bulb around the operating chilled pipe does not create a blockage of streamflow, resulting in icing and the freezing to the bottom of downstream reaches. Where this problem potentially exists, it will be necessary to do more detailed thermal studies to determine whether special measures such as deep burial or special insulation are required.

Marine Systems

The following concerns have been identified with regard to the effect of a pipeline on the marine environment:

- Siltation resulting from nearshore trenching activities or runoff from major onshore work sites.
- Indicental fuel spillage associated with shipping or onshore work sites.

Field work is being carried out to establish the nature of bottom sediments and their relationship to the distribution of benthic fauna. When this work is completed, it will be possible to identify any potential problem areas. If this initial work indicates the existence of such areas, further work will be undertaken to establish the magnitude, duration, extent and importance of the siltation effects. On the basis of this information the environmental and engineering merits of alternative crossing sites or construction schedules and techniques will be examined.

Fuel spillage from transfer at dockside will be controlled by the application of strict standards for fuel handling. Hazards at sea are not unique to the project but are pertinent to all northern shipping. Effective solutions to the problem include programs to increase safety of navigation in Arctic waters and centrally coordinated emergency procedures.

Atmosphere

Inversions during the winter months could result in ice fogs and locally high concentrations of NOx emissions in the vicinity of compressor stations. It will be necessary, prior to final location and design, to have a more detailed understanding of local atmospheric conditions so that compressor station sites can be located in sites favorable for the dispersion of largely water vapour and carbon dioxide emissions.

Terrestrial Mammals

The following concerns related to terrestrial mammals have been identified:

- Disruption of normal behaviour patterns by noise, vehicular approach and human approach;
- Habitat destruction;
- Road kills; and
- Increased hunting pressure.
Disruption of behaviour patterns or large ungulate is being carefully examined. Two mechanisms of disturbance occurring simply or in combination are being investigated:

a) Frequent disruption could interfere with feeding, procreation and rearing of young leading to population decline.

b) Frequent disruption could lead to higher stress levels increasing energy demands of animals.

One has available the alternative strategies of either avoiding encounters or minimizing the impact of encounters.

The most promising approach to mammal protection is through procedural modifications to minimize impact of encounters. Such measures as activity restrictions at critical times of the year (such as during the calving period), rules to prevent the following of animals with aircraft and ground vehicles, controlled blasting during construction and other such measures applied at the source of disturbance can go a long way to minimizing the impact of disturbance.

Animal density is an extremely important parameter in the assessment of overall impact of disturbance. In the High Arctic, mammal populations appear to be so small and dispersed that chances of encounters are low even during intensive pipeline construction activities. In many situations special protective measures may not be justified.

Road kills do not appear to be a major problem in view of the low mammal densities and the lack of definite migration paths across the right-of-way.

It is not yet certain whether total hunting pressure will increase or decrease as a result of pipeline activities. On the one hand, employment of people from northern communities on work related to the pipeline will draw them away from hunting. Since guns will not be allowed in camp, except for emergencies, there will be no hunting opportunities on the job. However, increased exposure to potential new hunting areas may stimulate hunting of these areas when workers leave the project.

Marine Mammals

The following concerns have been identified with regard to the effects of a pipeline on marine mammals:

a) Habitat alteration.

b) Direct damage to animals from shock waves from underwater blasting.

c) Disturbance of behavioural patterns.

d) Attraction of polar bears to campsites.

Habitat alteration has been discussed in the section of Marine Systems. Changes in the bottom fauna are of major relevance to marine mammals which directly or indirectly depend on this fauna for their food intake.

Shock waves due to blasting of foreshore trenches could have a localized effect on ringed seals in the winter or seals and other marine mammals in the summer. Most problems due to blasting can be avoided or minimized through scheduling or special blasting techniques.

The approach to minimization of disturbance will be essentially the same as that applied to land mammals.

Prompt and positive disposal of garbage will be necessary to discourage attraction of polar bears. Educational programs on polar bear behavior and training in special avoidance procedures and in the use of scaring devices will be important. Cooperative efforts with government agencies will be necessary to develop effective measures to ensure safety of both men and bears.
Birds

The following concerns related to birds have been identified:

a) Disruption of normal behavioural patterns.
b) Habitat alteration.
c) Increased hunting pressure.
d) Fuel spillage.

Mechanisms of disruption of behaviour patterns of birds are similar to those discussed for mammals. Disturbances during nesting and moulting periods may be the most critical. In view of the rather strong correlation between certain landscape features and breakup phenomena, total avoidance of specified areas during critical periods is feasible. Avoidance of important areas coupled with selectivity-times procedural modifications should result in minimal disturbance.

Habitat alteration could take the form of drainage changes in lowland habitats and siltation of marine feeding areas, subjects which have been previously discussed.

The discussion of hunting pressure in the section on Terrestrial Mammals applies equally to waterfowl. In addition to control of hunting, strict policing measures will be necessary to prevent the capturing of rare and endangered peregrine falcons for export to falconers.

Birds are vulnerable to direct mortality through contact with oil or fuel contaminated water surfaces. Measures discussed under Marine Systems should adequately resolve this potential problem.

Fisheries

The primary consideration for protecting fisheries in both the freshwater and marine situation is to avoid significant environmental alteration as discussed in the sections on Freshwater and Marine Systems.

In addition, precautions must be taken to minimize the following direct disturbances:

a) Blasting damage to fish;
b) Creation of barriers to fish migration; and
c) Overfishing.

It is known that fish are highly vulnerable to air bladder damage from blasting shock waves. Blasting activities will be timed in such a way as to avoid fish concentrations. Modifications in blasting techniques may also be employed in order to minimize the shock wave effect.

Care will also be taken in the design of temporary crossings of streams to avoid the use of undersized culverts which cause high stream velocities against which arctic char are incapable of swimming.

In view of the small char populations and their slow growth rates, overfishing is viewed as a major environmental concern. In even a small fraction of the pipeline employees were to exercise their legal right to fish and catch their legal limits, there would be a substantial local impact on the char fisheries. It may be necessary to institute project specific controls in order to prevent such an occurrence.

OVERVIEW

Over the past two years, Polar Gas has been conducting field studies in the High Arctic to assess the environmental implications of building a chilled natural gas pipeline from...
Melville Island to Southern Canada. Studies to date have been carried out on the Boothia Peninsula and on Somerset, Prince of Wales, Cornwallis, Little Cornwallis, Bathurst and Melville Islands. Research on Land Mammals, Birds, Marine Mammals and general Marine Ecology is continuing in 1975 to further define the important biological parameters of the area.

The research effort described in this paper is part of a phased program of environmental field studies which will eventually cover the full Polar Gas route.

The results of the environmental studies to date have provided Polar Gas researchers with a significant grasp of the important physical and biological parameters in the High Arctic. Analysis of these parameters has identified the key areas of interaction between the pipeline and the environment. It is believed that routing, design, construction techniques and schedules and operating procedures can be chosen which will result in an environmental impact that is reasonable and acceptable in return for the essential transportation system provided by the pipeline.
UNIQUE PROBLEMS OF OIL DEVELOPMENT IN THE ARCTIC

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ABSTRACT

Technology evolved by the oil industry for operations in harsh environments of other parts of the world has been applied to the development of oil resources in the Alaskan Arctic. However, there are several situations which are unique to the Arctic that require new technology. This paper describes some of the unique situations, field and laboratory tests and the application of test solutions to design and operation of the Prudhoe Bay facilities.

Remoteness of the area requires establishment of self-supporting working communities supplied by their own air, land and water transportation systems. Damage to the delicate tundra is minimized by centralized development and restricted offroad travel. Instability of the tundra surface requires development of construction designs which assure the integrity of structures and road systems.

Experimental field and laboratory programs were conducted to determine criteria for wells to be drilled and completed in the underlying permafrost.

Environmental studies sponsored by industry, government agencies, and scientific organizations have provided more complete data on the Arctic ecosystem applicable to the specific requirements of oil development. These studies continue to date.

Initial exploration and production in the shallow ice-infested waters of the Alaskan Beaufort Sea will rely largely on technology developed at Prudhoe Bay and in other ice-infested waters such as Cook Inlet, the Canadian Beaufort Sea, and Prudhoe Bay offshore. However, additional unique problems are already being recognized. Studies are ongoing both with industry and government to identify the best solutions to these problems.
INTRODUCTION

After several years of unsuccessful exploratory activity in the Alaskan Arctic, oil industry development operations commenced in earnest with the discovery and confirmation of very large oil reserves at Prudhoe Bay in 1968. During the subsequent development period much has been learned about the North Slope area. Further, industry has developed methods for operating in the unique and harsh environment which is characteristic of the area.

The North Slope of Alaska may be defined as that area which lies north of the Brooks Range Divide, about 76,250 square miles in extent. The Brooks Range crosses Alaska from east to west and is about 150 miles wide. North of the range is a treeless foothills province of moderate relief which is about 20 to 80 miles wide. The Prudhoe Bay area lies in the Arctic Coastal Plain and is bordered on the north by the Beaufort Sea. The Coastal Plain is characterized by minor relief with a slope so slight that it appears to be an endless flat. It is a tundra plain covered by thousands of lakes and swamps and poorly drained by meandering streams. Lakes are so numerous that in many places there are large areas which contain as much water as land. Almost all lakes and streams are entirely frozen each winter.

The climate of the North Slope is among the harshest in any area of the world where man lives and works. The cold average temperature, high winds and extended periods of darkness combine to create this severe climate. Temperature records indicate a range from 70°F in July to -60°F in the winter. The average wind velocity during a year is approximately 15 miles per hour with the maximum at times reaching some 70 miles per hour. Wind velocities are generally greatest in the winter months. Frigid winters are very long and the summers are brief. The average annual precipitation is around five inches and normally occurs in the form of heavy fogs, mist and snow.

In many instances, some of the same basic procedures which are common to other areas of the world are applicable in the Alaskan Arctic. Conditions, such as cold, high winds, snow, wide seasonal variations, rain, ice, fog and generally arid conditions have previously been encountered, and thus are not new. Industry has been faced, however, with several unique problems which did present new challenges requiring considerable study to reach acceptable solutions. These problems include:

1. Remoteness
2. Tundra Surface
3. Permafrost
4. Arctic Ecosystem

The purpose of this paper is to recognize the existence of the problems and to discuss some of the procedures that were used to find solutions. Additional unique problems associated with future operations in the Arctic Offshore area are also recognized.

Remoteness

Living Conditions

Establishing an operation or base of operation in the Arctic means providing a self-sufficient, totally independent environment for life support of the people involved.

Exploration activities for oil on the North Slope of Alaska are transient and temporary. Housing is modular and portable.

Development and operation of the Prudhoe Bay Field, however, required the establishment of a permanent camp-community environment for life support over a period of many years.
A semi-permanent camp was built in 1969 to support early construction activities. The A.R.Co./Exxon permanent camp facility at Prudhoe Bay was completed in 1970 and has been the base for development activities since that time. The permanent camp facility is currently being expanded to provide additional offices, living quarters, recreation facilities and service areas for the field operating staff.

Utilities included in the complex and connected by heated hallways are the Water Plant, Sewage Plant, Warehouse, Generator Plant, Automotive Shop and Vehicle Storage.

To provide a healthy and pleasant environment, the interior of the base camp has been designed to emphasize spaciousness, which is important in confined, Arctic living quarters.

Facilities for recreation include shuffle board, billiards, a library and movie room. The base camp expansion now under construction will add a basketball court, exercise and weight-lifting room, a sauna bath, and a new theatre-type movie room with 130 seats. The new living quarters will also provide one-person-per-room accommodations for added privacy.

The medical facility for the complex has an examining room with X-ray and electrocardiogram equipment and a four-bed infirmary. A physician's assistant is in attendance at all times.

The North Slope tundra in summer months provides an almost limitless supply of water. In contrast, with a maximum water freeze of over six feet in winter, most lakes and rivers freeze to the bottom.

The Sagavanirktok River has been a primary source of water at Prudhoe, and a water intake station was built in one of the deeper channels, approximately one mile from the base camp complex. The water is preheated a few degrees at the intake station and pumped into an insulated pipeline. The increased water temperature is then maintained by electrical impedance or resistance heating for delivery to the water plant. Processing at the water plant provides filtering, chlorination, and softening.

To have a large alternate source of water connected to the water system, a 30-acre lake was recently deepened to about 18 feet. Water will be pumped from beneath the ice during the months of maximum freeze, and the reservoir refilled from the river each summer.

Sewage treatment and disposal has always presented major concern in the Arctic. All water used in a life support facility needs a disposal outlet. Portable camps use portable sewage units. The permanent sewage plant at our Prudhoe Base Camp has used an extended aeration, secondary treatment process.

Chlorinated effluent from the plant is pumped through an insulated and heated pipeline to a lagoon where it is released to the bottom.

The present plant has operated quite successfully since 1970. To accommodate base camp expansions, however, and to increase capacity and efficiency, an Activated Biofilter system is currently being added. The resultant system should be one of the most advanced in existence.

A unique natural phenomenon in the Prudhoe Bay area has been used to accommodate the disposal of solid waste. Windblown deposits from the Sagavanirktok River delta created a large sand dune area. A sanitary landfill was developed in the sand dunes and all residue from the base camp incinerator, together with other disposable solids, are buried at this facility.
Logistics

With the critical need for materials and supplies to support all activities, plans for accommodating air, land, and water transportation had to be a first consideration. Some of the initial effort at Prudhoe Bay, therefore, was the construction of an airstrip, dock, and interconnecting road system.

To satisfy early needs for supplies, an airlift was established (with a peak in 1969) using primarily C-130 Hercules aircraft. The materials moved to the North Slope by air in 1969 totaled 36,000 tons, or the equivalent of about 2,000 Hercules flights. Much of the equipment had to be adapted or designed to fit into the airplanes. Included were a complete diesel plant, which was needed for fuel manufacture, and three complete drilling rigs.

All transportation of people has been by air.

The first constructed ground transportation corridor to the North Slope was a winter ice road built by the State of Alaska in the winter months of 1969 and 1970. However, with delays in construction of the Transalaska Pipeline, the need for materials diminished and the haul road was abandoned.

In the fall of 1974, Alyeska Pipeline Company completed the first phase of a gravel road to the North Slope. This road will serve as the first permanent land connection from Fairbanks to Prudhoe Bay, a distance of some 500 miles. When the bridge across the Yukon River is completed in 1976, year-round access will be available for pipeline-related activity.

The greatest quantity of materials and equipment, by far, has been moved to the North Slope by water. Ocean-going barges are pulled around the west coast of Alaska and from the Canadian Mackenzie River. Access to Prudhoe Bay by barge, however, is possible only when ice conditions along the Arctic coast permit. This is usually during the month of August. Considerable planning, therefore, is necessary to meet schedules of resupply and equipment movement during this brief period.

Each summer a dock and causeway receives the barges. During the period 1969 through 1974, barge freight was moved across an 1100-foot causeway and dock located on the south side of Prudhoe Bay. To handle the large quantities of freight required during the present and future barge seasons, a second causeway and dock has been constructed. Some 440,000 tons of equipment will have been moved across the Prudhoe Bay docks after the 1975 season.

Tundra Surface

Problems Recognized

The tundra layer provides insulation for the permanently frozen soils below. In many cases, this permanently frozen permafrost is rich in ice. Any damage of the tundra insulating layer can result in thermal erosion and can create severe environmental damage which is slow to heal. In the summer months, the tundra layer thaws and the terrain becomes like a swamp because the permanently frozen soil prohibits any circulation of the water into the ground system. This thawed layer will not support movement of conventional surface vehicles and again, disturbance to the tundra layer could result in melting of the permafrost.

Recognizing these unique characteristics of the tundra and permafrost, facility designs and operational practices have been developed which allow safe and effective operations, yet protect the delicate tundra system.
Centralization Concept

For Prudhoe Bay oil field development the centralization concept has been utilized. Centralized facilities are being used for personnel housing, oil processing facilities, gas handling facilities, staging areas and for electrical power generation. These facility sites are underlain and/or surrounded by gravel pads to provide all weather access. The centralization concept permits a concentration of facilities in a relatively small area, thus, minimizing environmental disturbance.

Oil wells are being directionally drilled from several multiple well drill sites which are placed in selected locations throughout the field. The sites consist of gravel pads with a minimum thickness of five feet. The gravel acts as an insulator and prevents thaw of the underlying permafrost. Initially, the average number of wells per drill site will be eight. With future requirements, these drill site areas may contain 20 or more wells.

Practices

Possible disturbance of the tundra surface is also avoided by use of an access road system which is being constructed to the various centralized field facilities. The access roads, which are constructed of gravel, generally have a 30 foot crown width and are of sufficient thickness to permit travel during the several week summer thaw season. In this connection, studies have been conducted in the field area utilizing temperature probes in and below gravel fill for development of thermal profiles versus time.

In addition to the road system for general field traffic, a gravel construction pad will parallel all pipeline systems in the field. These pads, which will provide all season access across the tundra without damage, will be used for pipeline installation and maintenance. Pipelines will be installed above ground on pipe supports which consist of steel piling with structural steel crossbeams. Pipelines will be insulated and will be of sufficient height above the tundra surface to prevent tundra thaw.

Permanent heated structures, such as living and facility areas, are supported on pile foundations with several feet of free air space below the building. Elevation of the building above the ground surface allows dissipation of heat from underneath the building, thereby preventing permafrost thaw. Air circulation around and under the building also helps minimize snow drift accumulation. Piling used for building foundations can be either wood or steel. They are placed in drilled holes, backfilled with a slurry mixture and allowed to freeze back.

Other Considerations

The gravel road system allows year around access to all facilities and the centralized development concept reduces the miles of road and number of facilities. However, there are situations where travel is required outside of the established road system. Movement of conventional surface vehicles over the tundra is restricted to the winter months. Rolagons, that is vehicles which travel on inflated rubber bags, have been used for off-road travel during the summer. These vehicles exert little pressure on the tundra, thus allow cross country movement without damage to the fragile vegetation cover.

The primary emphasis has been on the prevention of damage to the fragile tundra system. However, there are instances when the tundra will be damaged and quick, effective rehabilitation of the damaged area is required to avoid permanent scars.

The need to rehabilitate certain damaged areas of the tundra was recognized early in the development of the Prudhoe Bay field. Initial research programs defined rehabilitation techniques which were effective but required continued maintenance, for example, repeated
fertilization and reseeding. The present tundra rehabilitation research program sponsored by industry is aimed at long term rehabilitation using to the extent practical native seeds. The research is identifying seedable plant materials, planting techniques and land management practices. In addition, the research is intended to identify natural processes which might result from the rehabilitation activities. These natural processes could include the attraction of grazing animals, such as caribou, to the rehabilitation site.

The data obtained from both of these programs, that is, the short-term and long-term rehabilitation programs, has been obtained under actual field conditions on the North Slope of Alaska.

Permafrost

The production of petroleum reserves underlying thick permafrost required development of wells which could be safely operated. The thaw zone which melts out surrounding a producing well leads to thaw subsidence loading on the casing. The refreezing of the thawed permafrost during an interruption in the flow of the hot fluid causes an external pressure loading on the casing. Furthermore, during the "freezeback" of a well, freezable fluids in the well itself will freeze and create casing collapse pressures inside the assembled well. These specific permafrost operational problems are recognized design considerations for commercial petroleum production in a permafrost environment.

Freezeback

The freezeback problem was diagnosed during the early development drilling phase in Prudhoe Bay field. The potential of an external casing collapse was recognized as a result of the refreezing of a saturated thawed permafrost zone surrounding a well. Laboratory investigations indicated that significant pressures could be produced during refreezing of a thaw zone. Detailed soil mechanics data were developed and employed in a numerical simulation of the refreezing process. The results of the simulation were compared to two sets of independent full scale field test data. In the field tests, actual refreeze pressures were measured as a function of time and depth for various thaw conditions. Laboratory estimated pressures were verified by those observed in the field test. Maximum expected pressures were predicted, and casings with adequate collapse ratings were selected for the outer conductor and surface casing in the permafrost.

Internal freeze damage can be eliminated by removing all of the water from cased annuli in the permafrost of the well. Removal of water base drilling mud was investigated in a series of laboratory and prototype experiments. A two-step displacement procedure was developed. In this process, conditioned water base mud is washed from an annulus with fresh water. The water is then displaced by an oil base fluid. Efficiency of this displacement approaches 100 percent in actual practice. Potential freezeback in the internal strings, therefore, has been solved by replacing all freezable fluid with a nonfreezable oil base fluid.

Thaw Subsidence

The thaw subsidence or thaw consolidation process has been recognized as a severe problem for surface construction design in the cold regions of the world. The thaw zone associated with a hot oil well and the related "subsidence" are important design considerations in Prudhoe Bay well design. To determine the magnitude of subsidence loading on well casing in the permafrost, a major research and engineering program was initiated. The single most important task was operation of a thaw subsidence field test near the center of the Prudhoe Bay field. The field test thaw zone approximated the thaw zone of a mid-structure producer after twenty years production. The actual insitu
casing deformation was measured with a specially developed logging tool. Formation pressures were recorded throughout the test life. Numerous other types of data were collected during drilling and the twenty-one month thaw phase of the test. These data, along with extensive mechanical behavior data for naturally occurring soils provided input information for numerical simulation of the thaw subsidence process.

Subsidence simulation was calibrated with field test casing strain data from the test site. Hindcasting indicated the model could estimate observed strains as size of the thaw zone increased. The simulation was employed to predict maximum expected strain for a worst case subsidence condition in the Prudhoe Bay Field. The worst case was developed after a sensitivity study of simulation defined the critical factors for producing a subsidence strain. The worst case analysis was developed as the most adverse combination of these critical factors. The compressive strains in this case approached .7 percent. Expected strains in typical wells would be on the order of .2 percent. The maximum strain observed in the field test was .3 percent. Tensile strains of smaller magnitude were expected based on the simulation and actual observations in the field test.

The strain limit of 13-3/8 inch N-80 normalized casing connections was determined in an axial deformation test. A finite element simulation of the casing connection was employed to generalize the experimental data to a broader class of casing loading. Minimum strain limits were defined for the casing of interest. The 13-3/8 inch API buttress casing was shown to have a minimum ultimate compressive strain capacity of 2.3 percent and a minimum ultimate tensile strain capacity of 3.8 percent.

The most significant conclusion of the overall research and engineering effort is that the casing is commercially available which can withstand the expected maximum subsidence strains in an oil well completion through the permafrost interval with very acceptable design factors.

The significant result of the last three years of research and engineering study has been the development of a casing program that will safely produce petroleum resources in the Alaskan Arctic.

Environmental

All operators in the Arctic can be considered as environmentalists. For example, an operator is always aware of the environment around him in the Arctic and his primary concern is for safe and effective design and operations. This is the primary prerequisite for an environmentally sound program.

As activities began to accelerate in the Prudhoe Bay area in 1968, many of the unique characteristics of the Arctic environmental system were already recognized. These were known through a review of existing knowledge from the literature or from experiences in other cold regions. Also, the knowledge gained by the operations in the previous fifteen years in the Prudhoe Bay area was directly applicable. Recognizing the fragile and unique features of the Alaskan Arctic ecosystem, many engineering and operational steps were taken directly. Examples of some of these steps have already been mentioned.

In addition, it was realized that there were other unique features of the Arctic ecosystem that we were not necessarily familiar enough with. Initially, Atlantic Richfield Company hired Angus Gavin, an ecologist who had spent his lifetime in studies of the Arctic, to study the ecology of the Alaskan North Slope and recommend practices to ensure its maximum protection. Angus Gavin has now been in residence at Prudhoe Bay each year for seven years in the study of water fowl, fish and game and his observations are continuing into the development stage of the field. In the past two years, Gavin

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has been concentrating on the documentation of the effects of our existing facilities on caribou behavior. The behavior we observed is that caribou quickly become accustomed to man's presence and that our existing facilities are not a hindrance to the caribou in any way. Gavin is attempting to document this behavior in order to determine whether our observations are correct and whether the precautions we have taken to protect the caribou are adequate. One such precaution is illustrated in this slide of a pipeline crossing for caribou so as to allow free movement within the unit area.

The most comprehensive program which was aimed at understanding the total tundra ecosystem has been the Tundra Biome program. The Prudhoe Bay operators participated in the Tundra Biome program for five years of its existence. The primary tundra study site was at Barrow but with funds provided by the operators, a comparison site was established at Prudhoe. Last year was the final year of the biological program and was concerned primarily with the preparation of reports. However, several field studies are continuing to completion at Prudhoe in 1975. These are soils/vegetation mapping programs. This interdisciplinary approach to the understanding of the tundra ecosystem has been one of the more successful scientific endeavors. We feel that the results of these studies and our years of experience in the Arctic have provided us with a fair understanding of the tundra ecosystem.

What is learned from our experiences in Prudhoe Bay as well as in other areas of the world where we operate will be applicable to some degree in new areas of oil and gas exploration and production. Several years ago, the industry participated in an Arctic estuary study by the Institute of Marine Sciences, University of Alaska. This program concentrated primarily on the Colville River Delta, Harrison Bay and Simpson Lagoon. Last year the Prudhoe Bay operators sponsored an extension of the studies into Prudhoe Bay. These investigations were also supplemented by marine fisheries investigations of the Alaska Department of Fish and Game. The major components of this program include fishery studies, benthic biological studies, sediment studies, and hydrocarbon background analyses.

This extension was primarily associated with the construction of a 4,500 foot causeway and dock and drilling from stabilized islands in Prudhoe Bay. Prior to the causeway and dock construction and the drilling activity, our evaluation was that there would be no significant impact on the marine ecosystem. However, as we move further offshore, we apply what we have learned from previous operations. The studies of the Prudhoe Bay marine environment should provide us knowledge applicable to operations further offshore in the Beaufort Sea, i.e., whether the precautions now being taken to protect the environment are adequate for facilities further offshore.

Offshore Extension of Operations

The discussion thus far has related primarily to experience and problems on land. Much of this is certainly applicable to future operations in the offshore area, which is industry's next big technological challenge in the Arctic. The offshore area will carry many similar problems with the added dimension of operations in ice infested waters.

Industry is already preparing for operations in the Alaskan Offshore area through specialized research studies both in the United States and Canada. We will utilize results from these studies in conjunction with experience gained in the North Sea, Cook Inlet, Canadian Beaufort Sea, and in the Prudhoe Bay Onshore and Offshore area, in a continuing search for mineral resources. This is a new and exciting challenge which conceivably will be more costly than any area in which industry has operated to date. We are now ready to meet that challenge.
MINING IN ANTARCTICA: SURVEY OF MINERAL RESOURCES AND POSSIBLE EXPLOITATION METHODS

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ABSTRACT

A list of known occurrences of economic minerals in Antarctica is sparse at best, but the reason for this may be simply the result of inadequate exploration. Only about five percent of the continent is not covered with ice, and this greatly hampers direct observations of potential deposits. Another factor in limiting exploration for minerals is that the emphasis has been on basic science rather than commercial ventures. However, even though no present occurrences could be considered as ore bodies, many require further study to determine their potentials. Assuming that some deposits might eventually become economic, several exploitation methods could be considered, depending on whether a deposit is ice-free or ice-covered. In one example covered here, something would be introduced into the continent rather than being removed; namely, radioactive wastes would be stored within the ice sheet and left to disintegrate harmlessly. In the case of a deposit in the interior, transport to the coast would be a major problem. Coastal facilities for ship docking, initial concentrating of ores, and similar basic units would have to be constructed for virtually any exploitation operation. Offshore mining of manganese nodules and drilling for petroleum and natural gas are likely in view of present information on their occurrences. Other likely potential resources include coal and iron. Towing of icebergs to arid regions for water supply is considered to be feasible, but requires additional study before being implemented. It is an understatement to say, unfortunately, that difficult, but not insurmountable, technological problems will have to be resolved before any exploitation can begin. This may be a decade or more in the future and will involve large amounts of money. Political disputes over sovereignty of potential deposits loom as an issue that remains to be solved.
INTRODUCTION

A brief comparison of Antarctica and the Arctic shows several differences in geography and economic resources. The Arctic consists of an ocean centered generally at the North Pole with land masses around it. Arctic regions are relatively rich in mineral resources. Antarctica is a continent located at the South Pole with major oceans surrounding it, and has no known mineral resources that are presently economic. Historically, the polar regions are also different in that Antarctica was discovered only about 150 years ago. Only in this century has information been forthcoming about the geology of Antarctica and its mineral resources.

The purpose of this presentation is to review those mineral resources and offer some suggestions as to how they might be exploited if the market demand becomes favorable, technology becomes available for particular problems, and political issues can be resolved.

The geology of Antarctica was only generally known as recently as twenty years ago. With a few exceptions, only rock outcrops around the coast had been explored by the middle of this century. Figure 1 shows the extent of geologic information as of 1956 when preparations began for the International Geophysical Year, 1957-58. Information gained since that time on the geology and mineral resources of Antarctica has provided reasonably detailed geologic maps (Craddock et al., 1969-70) and a general assessment of resource potential (Potter, 1969; Smith, 1972; Wright and Williams, 1974). Studies conducted from research ships in the oceans surrounding Antarctica, particularly the Eltanin since 1962 and the Glomar Challenger since 1972, have contributed greatly to information on tectonics of the southern hemisphere, relationship of Antarctica to the other southern continents, marine geology, and distribution of manganese nodules. A compilation of 222 mineral species, subspecies, and varieties (Steward, 1964) includes several of potential commercial value.
However, one common factor stands out among all of these studies. To date, no commercial deposit of any resource has yet been found. Although there are several that deserve further exploration and evaluation, none can be classified presently as economic. Potential resources as of 1974 and their locations are generalized in Figure 2. A detailed version of this map can be found in Wright and Williams (1974), where individual deposits are described and located. These include sand and gravel, iron, copper, gold, silver, molybdenum, and coal, plus an assortment of miscellaneous metals and non-metals. Offshore potential resources include metalliferous nodules and oil and natural gas. In a few selected active volcanic areas, geothermal energy is indicated. Finally, the possibility of transporting icebergs to more northern latitudes for potable water and irrigation should not be overlooked.

There are sound geological reasons for the presence of mineral resources in Antarctica, according to Wright and Williams (1974). Based on known mineral occurrences in Antarctica and the relationships between the geological provinces in Antarctica and those of neighboring Gondwana continents, they have made estimations of mineral resources in Antarctica on a statistical framework. Figure 3 shows the reconstruction of Gondwanaland in order to show why this idea is plausible. The whole story is not quite this simple, but in general the presence of known ore bodies in other southern continents implies that they exist in Antarctica also, at least for those deposits formed before Cretaceous time, about 130 million years ago, when Gondwanaland began to separate. According to Wright and Williams (1974) the best discovery probability for a base-metal deposit in any part of Antarctica is in the Andean orogen, that area of Antarctica extending south from South America. The probability there is 75 in 1000. Parts of all the other southern continents are relatively rich in certain mineral deposits and, therefore, the same mineralization could (or should) extend into Antarctica.

Lassiter Coast

Copper mineralization has been reported (Rowley et al., 1975) at the base of the Antarctic Peninsula in the Lassiter Coast area (Fig. 4). In three field seasons of work, one area received detailed study that showed small, very low-grade mineralized zones that are not well known because of concealment beneath the adjacent ice cover. Most ore minerals occur as sulfides in propylitically altered rock. However, the average trace metal content of the rock in the area of greatest concentration is about 200 ppm Cu, 100 ppm Pb, and 50 ppm Mo, well below ore grade even in areas where mining is feasible. Further exploration could reveal greater concentrations and, possibly, an economic deposit.

Dufek Massif

The Dufek intrusion was discovered in 1957 when this area of the Pensacola Mountains (Fig. 4) was visited by a surface-traverse party. The intrusion, of Middle Jurassic age, is a large layered complex similar to the Bushveld in South Africa. Iron, the main metal present, occurs in the form of magnetite concentrations (70-80 percent magnetite) as much as several meters thick (Ford and Boyd, 1968). The thickness of the intrusive complex is estimated at 7 km, about 4 km of which is exposed. The minimum areal extent is estimated at 34,000 km², according to geophysical surveys, although most of the body is ice-covered. A diamond-drilling survey could delineate the extent of mineralization. Preliminary surveys have been made (Kovacs and Abele, 1974) to determine the possibilities of constructing a runway for wheeled aircraft in this vicinity.

Other Possibilities

It has been known for many years that coal occurs in numerous parts of Antarctica, including coastal areas where mining and transportation would be more feasible. However, coal resources in other parts of the world are sufficiently plentiful to preclude any development of Antarctic coals, except for possible use locally for heating or power production.
Figure 2. Known occurrences (solid circles) of potentially valuable minerals in Antarctica. The outermost dashed line is the limit of the continental shelf. In Ross Sea, G represents the presence of gas in holes drilled by Glomar Challenger in 1973 (1 is Hole 271, 2 is 272, 3 is 273) and one (D) drilled in 1975 as part of Dry Valley Drilling Project (at 68 m depth unconsolidated sediments contained 38% methane).
An iron deposit of apparent large dimensions has been reported in the Prince Charles Mountains (65° E) and will probably receive more attention because it is near the coast.

Most other reported occurrences are incompletely surveyed or are in areas that pose logistical problems that would prohibit any exploitation. For fuller discussions of additional occurrences see Wright and Williams (1974) and a report by the Fridtjof Nansen Foundation (1973) (especially the latter for problems posed by exploitation).

**OFFSHORE AND DEEP-OCEAN DEVELOPMENT**

Probably the most favorable area for economic development in Antarctic regions is offshore. The problems of land transportation systems and associated facilities for the mining of deposits on the continent would be avoided, although problems would be posed by sea ice, icebergs, and water depths.

**Oil and Natural Gas**

Estimates of oil and gas resources in Antarctica vary considerably, mainly because of the lack of direct information. The U.S. Geological Survey has reported that "the western Antarctic continental shelves alone could have potential resources of 45 billion barrels of oil and 115 trillion cubic feet (3.25 trillion m³) of natural gas" (Science, 1974), while another estimate (Moody and Geiger, 1975) of undiscovered potential for all Antarctica is 20 billion barrels of oil and 80 trillion cubic feet (2.27 trillion m³) of natural gas.
Figure 4. Map of Antarctica showing selected stations and physical features
Two broad areas present the best prospects for exploration: Weddell Sea and Ross Sea (Fig. 2). The Weddell Sea shows promise because of a structure similar to the Maracaibo basin (Deuser, 1971), among other reasons, but sea ice in the Weddell Sea poses a serious problem. The Ross Sea is encouraging because of the presence of gaseous hydrocarbons (McIver, 1975) in rocks of Miocene age in three of the four holes drilled there by the Glomar Challenger in 1973 (Fig. 2). The holes were on the continental shelf at water depths of 500 to 600 m and the maximum penetration (hole 272) was 443 m (Hayes and Frakes, 1975). Whether there is any correlation between those gas occurrences and the Gippsland Basin (Australia) and Terinake region (New Zealand) is unknown and may be completely coincidental. Whether there are liquid hydrocarbons at greater depths under the Ross Sea remains to be seen.

A hole drilled about 68 m in the sea bottom from a sea-ice platform in November 1975 (Fig. 2) over 120 m of water yielded unconsolidated core containing 38 percent methane.

Manganese Nodules

As a result of studies done mainly by the research ship Eltanin, the distribution of manganese (or ferromanganese) nodules is relatively well known in the South Pacific Ocean, Drake Passage, and Scotia Sea (Southwest Atlantic Ocean). A continuous belt of concretions up to 500 km wide lies beneath the circumpolar current at about 60°S. Less continuous fields lie north and south of the belt. These fields occur in water depths of less than 900 m to more than 4500 m. The principal Mn mineral is todorokite, with smaller amounts of birnessite (from summary of Goodell et al., 1971). In a study of both manganese nodules and ferromanganese oxide deposits (encrustations on rock outcrops) in the entire Atlantic Ocean, Cronan (1975) found the highest Fe concentrations (occasionally >40%, average 26%) in ferromanganese oxide deposits from the Drake Passage-Scotia Sea area and Mn lower (average 13%) than in any other area. Most manganese nodules and related deposits, however, would be mined for their accessory constituents, primarily copper, nickel, and cobalt. In the Drake Passage-Scotia Sea area, the abundances of these elements are 0.105%, 0.244%, and 0.179%, respectively (Cronan, 1975; based on 36 samples of ferromanganese oxide deposits, average water depth of 3696 m).

Because manganese nodules are more abundant in other parts of the world's oceans, and other areas have nodules with higher contents of desirable elements, the sea bottom around Antarctica will probably not be mined in the near future, but could become important at some future time. Water depth should be no problem, because technology already exists to conduct mining in water several kilometers deep (see Mining Engineering, April 1975-special issue on deep-ocean and offshore mining; also Cruickshank and Marsden, 1973). However, there is no policy to regulate this activity, even though it has been predicted that by 1985 there will be at least six companies actively mining manganese nodules. In Antarctic waters there would still be the problem of icebergs, even after the political boundary issue is resolved. (Some claims extend to 60°S; see Fig. 11 and section below on Territorial Claims.)

DIFFICULTIES OF EXPLORATION, DEVELOPMENT AND EXPLOITATION

Now that we have seen where known mineral occurrences are, why they are or might be there, and why there might be more if a more intensive exploration program were conducted, the following section discusses some of the obstacles presented to the exploitation of those resources.

Ice Cover

The distribution of mineral occurrences conforms generally with the distribution of rock exposures, as seen by comparing Figures 2 and 5. Less than five percent of the entire continent is ice-free. Only within the vicinity of outcrops could any practical or
Figure 5. Distribution of rock exposures in Antarctica
feasible mineral exploration and exploitation programs be conducted; in other words, where the ice is not present or is at least very thin.

This is an important point to be considered as far as minerals exploration and possible exploitation are concerned. Even though Antarctica covers an area of about 14 million square kilometers (about 1.5 times the area of the United States), only the five percent or less that is uncovered by ice can offer any possibility of mineral development. The remainder is more or less inaccessible under ice as much as 3000 m thick in some locations.

A related example of an ore body presently being studied is at Isua on the west coast of Greenland near Godthab. The deposit is partly ice-free but it also extends underneath the edge of the ice sheet. Mining such a deposit would be similar to the usual practice of removing the overburden to get at the ore. Unfortunately, in the case of Isua, the adjacent "overburden" (ice) would encroach slowly into the area of mining activity and would continue to be a problem for open-pit mining (Colbeck, 1974). It has been suggested (Ashton, 1975) that one possible means of slowing the rate of ice movement toward the pit would be to artificially cool the ice below its present temperature.

Remoteness and Environment

Antarctica is a difficult place to get to and a difficult place in which to work. There are no centers of civilization or populated areas, except for the few dozen research stations located on and around the continent. Figure 4 shows most of the stations as of a few years ago when about a dozen countries were active in maintaining them for mainly research purposes. For purposes of scale it is 4000 km from Christchurch, New Zealand to McMurdo, the main U.S. station, and 1350 km from McMurdo to the South Pole. Access to Antarctica is by aircraft or ship, but for only a limited time of the year. Because of its location, Antarctica is dark about half the year, making any activities outdoors hazardous. Combined with the world's worst weather, virtually nothing can be done outside in winter except for very brief and routine tasks. Extremely low temperatures can make some metals brittle, transform some fuels into a solid mass, and damage the lungs if one breathes deeply. The world's record low temperature of -88 °C (-127 °F) was recorded at Vostok Station (Fig. 4).

Many of the stations shown on Figure 4 have been occupied continuously since the IGY, beginning about 1957, for scientific programs at the stations, and also to get an early start in the spring on research projects that can be done outside. Early explorers had to do this by sailing to Antarctica during the short summer season, establishing a base for the winter and then beginning geographic exploration or research projects the next spring and summer. This has changed considerably with the advent of aircraft support, particularly for the U.S. and a few other national programs. However, the United States and other countries suspend aircraft operations during the winter and sea ice prevents ship operations as well, so the entire continent is essentially isolated for about half of each year. An exception is in the Antarctic Peninsula area, where the Argentine government can conduct year-round flights by C-130 aircraft between one of their stations and Argentina.

Just like the North Slope of Alaska, and many other remote areas, all supplies have to be brought into Antarctica (Fig. 6). There is no natural food supply. Figures 7 and 8 show McMurdo Station, perhaps the most accessible and certainly the largest and most sophisticated station in all of Antarctica. Lying directly south of New Zealand, it is the main base for U.S. activities; a station population of 1000 is not uncommon during the summer. A brief discussion of the McMurdo facilities is helpful as an example of the kind of development required for commercial activity in Antarctica. McMurdo is presently maintained by the U.S. National Science Foundation, sponsor of U.S. research in Antarctica, with logistic support provided by civilian organizations under contract to NSF, and by the U.S. Navy and U.S. Coast Guard.
Figure 6. LC-130 aircraft loading generators

Figure 7. McMurdo Station, Antarctica, showing natural harbor and ice wharf (arrow)
The buildings at McMurdo are typical for this kind of operation—prefabricated, easily constructed, and similar to those used in the Arctic. Because of the permafrost, essentially no subsurface installation is done. All water and sewer lines are above the surface and are kept from freezing by using electric tape wrappings around the pipes (Fig. 9). On a hill above the station (Fig. 8) is a seawater distillation plant that supplies fresh water for station needs. McMurdo is on the coast and this is not a problem. Fresh water in Antarctica has to be derived from seawater or by melting snow. At a few stations, freshwater lakes are tapped for a water supply. Most methods of producing liquid water in Antarctica are expensive and will have to be considered for commercial activity, although snow melting could be done cheaply if waste heat can be utilized from some other activity. Station power at McMurdo had been supplied by a nuclear reactor, adjacent to the seawater distillation building, from the early 1960’s until recently when a technical problem developed and the reactor was dismantled and returned to the United States. The station power is now supplied by diesel-fuel generators.

According to international agreement, the United States and other countries follow environmental practices in their operations in Antarctica, and field operations include removal, or in some cases incineration, of trash after field work is completed. This is another aspect to be considered for commercial activities.

Figure 7 also shows the natural harbor at McMurdo, allowing ships to approach shore for cargo transfers at a manmade ice wharf (Fig. 10) constructed in 1973 (Barthelemy, 1975). This kind of wharf could be constructed at coastal locations conducive to ship access and near ore-producing areas.
Figure 9. McMurdo Station, Antarctica, showing water and sewer pipe network.

Figure 10. Icebreaker next to ice wharf at McMurdo Station, Antarctica.
McMurdo is also served by aircraft that use landing strips on sea ice or shelf ice adjacent to the station. Both wheeled (C-130 and C-141) and ski-equipped (LC-130) aircraft can be accommodated. Few areas in Antarctica have the natural conditions required for permanent runways on land. Argentina presently maintains a station with such a runway and several feasible sites are under study by the United States (Kovacs and Abele, 1974).

McMurdo also maintains helicopters for research and reconnaissance purposes.

With regard to commercial development of the offshore area of Antarctica, the situation is perhaps no better as far as accessibility because of the great depth of water over the continental shelf and slope, the presence of sea ice and icebergs, and the horrendous logistical problems imposed. The only apparent optimistic feature is that there is essentially no threat of earthquake activity because Antarctica is aseismic.

**Territorial Claims**

In addition to the problems of distances, isolation, darkness, climate, and water supply, among others, as liabilities with regard to human presence and involvement in Antarctica the issue of political boundaries and claims must be mentioned.

Who owns Antarctica? Figure 11 shows that several countries do, and yet no one does, at least for 15 more years. Several areas have had claims made on them, but the Antarctic Treaty, signed in 1959 by the claimant countries and others working in Antarctica, holds all claims in reserve for the term of the Treaty, 30 years. The Treaty became effective in 1961. As shown in Figure 11 claims have been made by Chile, Britain, and Argentina, all three claims partly overlapping in the Antarctic Peninsula; Norway, between the British and Australian claims, but only the coast and undefined hinterland; Australia, the largest area, interrupted by the claim of France; and New Zealand. The large area of Marie Byrd Land (toward 90°W) is unclaimed. Note that the United States and the Soviet Union have no claims represented and have never claimed any of Antarctica, nor have many other countries working there. Offshore islands have also been claimed.

No stations are shown in the Antarctic Peninsula because they are too numerous to plot at the scale given. The United States operates Palmer Station, the Soviets operate Bellinghausen, and Britain, Argentina, and Chile have several stations each in this area. Also, the Soviet Union announced in 1975 (New York Times, June 29, p.8) that they intend to establish a new station on the coast near the Filchner Ice Shelf (Fig. 4—between Halley Bay and General Belgrano Stations; i.e., in the areas claimed by Britain and Argentina). Another point to mention here is that most claims extend north to 60° (the effective limit of the Antarctic Treaty), the British claim to 50°, and the Chilean northern limit undefined. This should be kept in mind for any offshore development on the continental shelf or in deeper waters.

The signatory countries of the Antarctic Treaty are working toward an amicable solution of resource exploitation, and held a meeting in Oslo in 1975 to discuss these issues. When the Treaty was drafted, the signatories avoided all reference to exploitation of mineral resources in Antarctica, but succeeding discoveries, particularly in offshore areas (see below), have made some kind of mutually agreeable policy mandatory. A U.S. polar advisory group (National Academy of Sciences, 1974) has recommended a program of "realistic but not exaggerated attention to the potential for mineral deposits in Antarctica and [to] share such information openly with other nations to do the same" (p. 22).

Legal problems related to exploitation of Antarctic mineral resources are discussed in depth in Schatz (1974), in a report by the Fridtjof Nansen Foundation (1973), and also by Slevich (1974).
SUGGESTIONS

Most of the preceding discussion has been pessimistic as far as any minerals exploitation in Antartica is concerned. Known occurrences are either below ore grade or else are unworkable because of logistics or technological difficulties. However, there are several reasons for anticipating a more active interest in minerals exploitation in the near future:

1. Mineral resources appear sparse because there have been no organized exploration programs. Most of the information on mineral resources is the result of basic science programs and not commercial ventures. Further exploration could yield much more promising resources, particularly in light of the reconstruction map of southern hemisphere continents (Fig. 3).

2. The depletion of known mineral resources in other parts of the world will throw more emphasis on resources in Antarctica. A more active market for certain metals or oil and natural gas could then outweigh the high costs of exploration and development, leading to an intense effort by industry.

3. Settlement of political issues is imminent, probably well before 1991 when the present 30-year period of the Antarctic Treaty is due to expire. Pressure from industry for leasing offshore areas has already begun, even though it is uncertain where the jurisdiction may lie to review, issue, or monitor leases, or what the terms might be.

4. Technology has not yet advanced to the stage required to deal with many of the problems that would be presented as a result of proposed development and exploitation, particularly in offshore areas. However, recent experiences in developing oil and gas resources in the North Sea and other harsh environments such as Alaska, Arctic Canada, and offshore Greenland and Labrador will be helpful in offshore Antarctica.

Development on the Continent

With the exception of the few occurrences mentioned earlier, mineral resources development on the continent shows little promise for the near future unless an active exploration program is initiated. Even if a major deposit is discovered, transportation would be a major problem. Because most of the continent is composed of a slowly moving ice sheet, surface transportation systems involving tracked vehicles or hovercraft would be the only likely means of bringing ore to the coast. Unless the ore payload has a high value, air transportation seems prohibitive. Coastal facilities would consist of ore-processing and concentration plants, ore-storage areas, and docks for ships to load the ore for transport to the north during the short summer season.

Disposal of Radioactive Wastes

This topic is mentioned here because the technology and transportation requirements are similar in some ways to mining activity, even though something would be introduced into the continent rather than removed. It has been proposed (Zeller et al., 1973) that the Antarctic ice sheet be used as a storehouse for radioactive wastes because the thicker parts of it would trap them for long periods of time, thus rendering them harmless as they decay. Packaged containers of wastes would be allowed to melt slowly from the surface downward. Various different disposal methods have been proposed for the ice sheet and the general concept has been discussed by international groups of scientists. In brief, it appears to be too risky in view of the lack of information about the ice sheet as a whole, particularly the characteristics of the bottom interface with bedrock. Furthermore, once entombed, there is essentially no way to retrieve the containers.
Figure 11. Map of Antarctica showing limits of territorial claims. Compare station locations with Figure 4.
Offshore

An early step in the development of offshore areas would be a systematic program of shipboard exploration to determine sub-bottom structures and other information, and then drilling. Capabilities for carrying out this phase already exist, even for Antarctic waters. What does not exist is a means of carrying out production activities in iceberg-ingested waters at depths greater than most of the world's continental shelves. The continental shelf around Antarctica (Fig. 2) averages about 500 m at the seaward edge, compared with about 200 m for the world's shelves. The continental slope is also steeper in Antarctica. However, oil production in the North Sea in water depths of 300 m is now feasible, and platforms and other structures are being designed for even greater depths. At the World Petroleum Congress in May 1975, a paper on "Deep water production methods" by M. Rederon et al. noted that "production may be initiated from fields in over 900 feet (275 m) of water by 1980 - by 1990, working water depths may be 10 times that great" (World Oil, June 1975, p. 73). According to a recent report, two dynamically positioned semi-submersible vessels designed for deep water appear to be suitable for Antarctic waters, as shown by model tests. Each was designed to work in all areas of the world, including iceberg zones, in almost any kind of weather. One vessel is designed for water depths of 1000 m and its propulsion units give a maximum still water speed of 10 knots. It could be in service by late 1978 (Offshore Services, April 1975, p. 54). The other vessel has a depth capability of about 1200 m (Cintract, 1975).

Design and tests have also been conducted on an offshore drilling system for use in the Arctic, featuring an air-cushion barge with modern offshore drilling equipment and an ice-melt positioning system (Anders and Nichols, 1973). The air-cushion system is capable of lifting the hull, complete with all drilling machinery, crew, and some drilling expendables, to a height if 2.6 m above any traverse surface, allowing a rig move during the frozen winter season. The external hull heating system is capable of melting a ledge of ice moving from any direction at a rate of 4.6 m per day using waste heat from three engines. The system maintains its position through a four-point mooring system with ice anchors attached to the ice pack. Tension can be applied on appropriate anchor lines opposing the direction of motion of the pack. This action forces the hot side of the hull to bear against the encroaching ice face, melting it at a rate equal to the motion of the ice. The unit is designed to operate in water depths from 0 to 180 m during the Arctic winter and in depths of 9 to 180 m during the summer, although an extending riser and mooring system will allow operations in water depths greater than 180 m. Drilling equipment and systems planned for the barge will be capable of drilling offshore wells to depths of nearly 5000 m. The entire system was designed for work in land-fast ice areas in the Arctic, but could have possible application for the Antarctic.

Another possibility of a system designed to withstand ice pressures while drilling in the Arctic is that of an ice-breaking drillship for offshore exploratory drilling (Jones and Schaff, 1975). Any of the units mentioned above could, therefore, feasibly conduct a drilling program in relatively deep water under pack-ice conditions. Dynamically positioned vessels have the maneuverability required to maintain their positions over drilling sites and avoid iceberg collisions in some cases. Such a vessel could also close off a hole, move away from a danger area, and reoccupy the site later. However, this could be costly depending on the frequency of occurrence. Large icebergs cannot be deflected and pose the biggest threat to offshore activity.1 (The largest Antarctic

1Icebergs as a Fresh-Water Source. Although icebergs are a hazard to offshore development, they can also be considered as an asset. Feasibility studies have been conducted on the subject of transporting Antarctic icebergs to arid regions of the southern hemisphere so that the meltwater can be mined for irrigation (Weeks and Campbell, 1973). Transportation seems to be an outstanding problem, because wind and currents dictate the motion of icebergs, and the energy required by tugs to tow them could be substantial. In-transit melting and related factors require more study.)
iceberg on record was 170 km long). Small icebergs could possibly be destroyed or reduced in size by explosives (Mellor, 1975), or else diverted by an icebreaker. Fortunately, in the deeper waters of the Ross Sea, icebergs could not become grounded or gouge the seafloor and thus would not be a threat to wellheads or other bottom systems. More will have to be known about regular iceberg movement patterns, if they exist.

Support vessels and aircraft could provide the surveillance required for iceberg movements, but the threat of icebergs for offshore development is a problem that remains to be solved, particularly for drilling vessels and production platforms. The answer to a production system maybe a seabed type designed by Lockheed Offshore Petroleum Systems (Doumani, 1973, p. 51), in which wellhead cellars, pipeline assembly, manifold center, gathering lines, subsea storage and separation center are all located on the bottom of the sea. The components are serviced by a manned capsule designed for a one-atmosphere environment at depths of 1200 ft (365 m), and the whole system can be extended to 2000 ft (610 m).

With the experience of offshore activities in the North Sea, West Greenland, and other areas, there is no reason to think that the problems of working in the waters around Antarctica could not be handled. Improvements in ice-breaking drillships, hovercraft, offshore structures, and other systems will make this possible. Even though the risks and cost may be great, the proceeds may be attractive. Recently the Danish Government granted licenses for exploratory drilling in David Strait, west of Greenland. Fixed platforms do not appear feasible because of icebergs, and the working season will no doubt be short, yet numerous companies are willing to take the risks involved. It has been reported that the cost per hole could be as much as $6 million. Furthermore, the Danish Government will retain a potential 80 percent of any profits. Offshore drilling in Antarctica in 10 to 20 years time would probably cost more yet, but the world's oil and gas supply will be depleted more.

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REFERENCES


After drilling 100 exploratory wells and 10 field delineation wells, industry has discovered six major gas fields and one oil field in the Canadian Arctic Islands. All six gas fields are known to extend offshore and seismic records indicate the presence of other prospective undrilled structures which lie entirely offshore. Horizontal ice movement was small and by using proper ice engineering methods it was found that the Arctic Ocean ice could be artificially thickened sufficiently to carry the weight of a 500-ton land rig. Two offshore gas wells were successfully drilled in 400 to 500 ft of water approximately seven miles offshore from Melville Island in 1974 and 1975. This technological breakthrough means that offshore fields can be discovered economically now rather than waiting several years for sophisticated semisubmersible drilling rigs to be developed which can operate in the severe ice conditions of the Canadian Arctic Islands.
INTRODUCTION

Several years ago Canada's reserves of oil and gas began to decline. As can be seen oil reserves peaked in 1969 and gas reserves peaked in 1971 (Fig. 1). The oil reserves graph excludes natural gas liquids. The gas reserves graph excludes Arctic Islands and Beaufort Basin discoveries because these fields are not yet connected to market by pipeline. To date then the reserves have come mainly from discoveries in the Western Canada Sedimentary Basin. This area has now been intensively explored for over 25 years. In the late 1960's it became obvious that hopes of significant new discoveries lay in three less accessible but promising frontier areas - The Beaufort Basin, the Arctic Islands and East Coast Offshore. (Fig. 2) Discoveries have now been made in all 3 areas. Of these areas the most difficult in which to mount a sizeable exploration effort was certainly the Arctic Islands.

THE ARCTIC ISLANDS

Reaching above Canada's mainland towards the North Pole, this region is remote from supply centres and inhospitable to the extreme (Fig. 3). Winter brings three months of total darkness. The Arctic Islands have no permanent settlements. There is no road or railroad access. The Arctic Ocean is frozen over for ten to eleven months each year. Figure 4 shows a rig on Ellesmere Island that drilled the world's most northerly wells above 80 degrees N Latitude. Supply by ship is only possible for a month or so each summer. Figure 5 is the Finnish tanker MT Falva working her way through the ice near Melville Island. Despite these difficulties, the oil industry has in seven years of operating learned how to proceed with exploration at a pace only slightly less efficient than in Western Canada. Expenses, however, are much higher. A mile of seismic or a foot of hole drilled costs 5 to 10 times more because of the logistics involved. For example a large drilling rig costs about $25,000 per day to support. A 30 man seismic crew operating with mobile camp and tracked vehicles costs about $20,000 per day.

REA POINT BASE CAMP

The main base of operations is on Melville Island at Rea Point, 3,300 miles by sea from Montreal and 1,800 miles by air from Calgary. (Fig. 2) As field headquarters for the Islands, Rea Point can accommodate up to 150 men at peak periods. It is adjacent to the sea and has a 6,000 foot airstrip which is useable year round. In August of each year, a convoy sails to this location from Montreal. Enough fuel and materials are delivered to supply the following year's drilling and seismic requirements. Ninety-five percent of all fuel is handled in bulk form. The sea going tankers offload at Rea Point into a fabric tank farm which holds 4 million gallons.

ARCTIC TRANSPORTATION

Aircraft

As there are no roads, heavy reliance is placed on air freighters, such as the Hercules C130. The large rear-loading Hercules has been the key to successful exploration in the Arctic Islands. With its capacity to haul complete drilling rigs it has opened up islands otherwise inaccessible by ship due to heavy ice. Other aircraft used extensively are four engine Lockheed Electras which handle the weekly crew change flights from Calgary. Twin Otters carry small freight and personnel between sites.
OIL
BILLIONS
OF
BARRELS

GAS
TRILLIONS
OF
CUBIC
FEET

PROVEN NATURAL GAS RESERVES

PROVEN CRUDE OIL RESERVES


FIGURE 1-REMAINING HYDROCARBON RESERVES OF CANADA

FIGURE 2-PRODUCING AND FRONTIER EXPLORATION AREAS OF CANADA
FIGURE 3 - SVERDRUP BASIN GAS and OIL FIELDS

FIGURE 4 - DRILLING NEAR EUREKA ON ELLESMORE ISLAND
Not every rig move is made by air. When the distance involved is 100 miles or less, rigs are often moved by truck (Fig. 6) along rough trails constructed on the frozen terrain or across the ice between islands. The ocean ice, usually measuring from 5 to 10 feet thick, is quite able to support the 50 ton loaded trucks. Such overland and overice moves are very economical. A 50 mile truck move can be accomplished for less than half the cost of an air move over the same distance.
ICE ROADS AND ICE MOVEMENT

It was this experience moving heavy trucks across the ice between islands and landing 75-ton Hercules aircraft on ocean ice airstrips that led engineers to consider the possibility of offshore drilling from an ice platform. Before this could be undertaken the single most critical factor was to know that the rig would not move in relation to the ocean floor during the drilling period. After several years of study commencing in 1971, it was determined that the ice sheet remains relatively stationary between islands during the months of January to June each year. Figure 3 shows about 20 various locations checked for horizontal ice movement over the years. Ice movement at the close-to-shore locations was monitored by surveying methods from on-shore observation stations. To get the far-from-shore ice movement information it was necessary to use an acoustic method which involved a pinger installation on the ocean floor as a reference point and a geophone pattern installed just below the ice to record distance and direction of any horizontal ice movement.

WATER DEPTH AND ICE THICKNESS

For ice platform drilling the limitation for horizontal ice movement is 5 percent of water depth. This limitation will tolerate 25 feet or so of movement. Water depth usually exceeds 1,000 feet within a few miles from shore (Fig. 3). Once it was established that horizontal ice movement was within acceptable limits the next step
was to design an ice platform that would carry the weight of a heavy drilling rig. For this project, a prominent ice engineering firm, Foundation Engineering of Canada, was retained in 1973. Calculations and experience with a small 150-ton rig showed that the natural ocean ice, 6 to 8 feet thick, would be insufficient to carry the concentrated load of a 500-ton drilling rig. Fenco's theory was that if the ice could be artificially thickened by flooding and freezing in one-inch to two-inch layers to a total thickness of about 12 to 16 feet in the center of a platform then the ice would safely carry the 500-ton rig.

**ICE PLATFORM DESIGN**

Hecla and Drake Point Gas Fields

Fenco's design was first used in 1974 to drill a gas well at Hecla N-52, 7 miles offshore from Melville Island (Fig. 7). Then in 1975, the design was used again to extend the Drake Point field 7 miles offshore on the east side of Sabine Peninsula. This design involved a 430-foot diameter ice platform 16 feet thick at the center and tapering to 6 feet natural ice thickness at the circumference (Fig. 8). The design was based on ice strength measurements and field testing conducted the previous winter. Figure 9 is a plan view of the platform showing the location of instruments used to monitor ice temperature and ice deflection under the drilling rig. The rectangular area in the center is the rig and rig shelter measuring about 150 feet by 30 feet. Figure 10 is a cross section which shows the way that the original ocean ice sheet deflected under the weight of artificial ice on top. It also shows how the platform tapers from 16 feet thick at the center to 6 feet thick natural ocean ice at the edges.
The three basic design considerations are buoyancy, shear and bending. Calculations showed that if the ice platform is strong enough in bending it is more than adequate for the other two considerations. For example, the total buoyancy of the 430-foot diameter platform constructed at Hecla was sufficient to carry a load 6 or 7 times the weight of the 500-ton drilling rig (Fig. 11).

Tensile Stresses

Nevertheless, due to bending stresses caused by the concentrated load, and in order to keep the tensile stresses below the 50 psi design criteria in the underside of the ice platform, it was necessary to have at least 12 feet of ice thickness under the rig itself. The as-built platform at Hecla had an added safety factor because it was made 16 feet thick so that calculated tensile stresses were only 25 to 30 psi instead of 50 psi. The 50 psi design criteria will likely be followed at ice platforms being built for offshore wells planned in 1976.
FIGURE 9
W. HECLA N-52 ICE PLATFORM SHOWING LOCATION OF
DEFLECTION AND THERMISTOR STATIONS

FIGURE 10-CROSS SECTIONS THROUGH W. HECLA N-52 ICE PLATFORM
The Arctic Ocean overlying the Sverdrup Basin is never ice free. However, it is usual for it to break up to some degree each summer with numerous cracks, leads and limited open water areas appearing during July, August and September. By early October the ice begins to consolidate again and the large pans of second year and multi-year ice become frozen in newly-formed first year ice. Access to the proposed offshore drilling site with light vehicles and flooding equipment is normally possible across one-foot to 2-foot thick first year ice by early November. At both Hecla N-52 and Drake I-55 a stable pan of multi-year ice about 5 feet thick covered the drilling location before ice platform construction commenced. The first step was to excavate a 4-foot deep hole in the natural ice using chain saws and ice picks. The moonpool or rectangular hole through which the well would later be drilled was then installed (Fig. 12). Flooding and freezing in successive 1-inch to 2-inch layers then got underway. A free flooding technique was used with 3 pumps located within 75 feet of the moonpool so that the flooded layers tended to taper to zero at the circumference of the platform. The portable screw type pumps used to 1974 delivered about 40,000 gal/hr each (Fig. 13). The 2-cycle gasoline engines on these pumps gave trouble in cold weather so in 1975 two electric driven submersible pumps were used instead (Fig. 14). The electric pumps were placed in insulated casings frozen through the ice and they performed trouble free. With either system 2 to 3 floods a
day could be accomplished. Average ice thickness buildup was 2-1/2 in/day at Hecla in 1974 and 3-1/2 in/day at Drake in 1975 (Fig. 15). Ignoring the shut down period the ten feet of artificial ice was built up in about 40 days at Drake in 1975.

ICE PLATFORM PERFORMANCE

The performance of the ice platform was monitored in several ways while drilling the well. The most critical measurement was vertical deflection with time under the weight of the rig (Fig. 16). Ice deflection was measured once a day along several radial lines emanating from the rig (Fig. 9). As long as the deflection rate was constant or decreasing there was no danger but an accelerating rate could have indicated that there was danger of failure. It turned out that the creep rate at Hecla in 1974 was steady and the deflections were quite small indicating a very safe design. Total deflection was slightly greater at Drake, about 0.8 feet compared to only 0.5 feet at Hecla (Fig. 16), however in both cases there was 1-1/2 feet of freeboard in the moonpool at the end of the drilling period. Freeboard is the distance from the top of the ice under the rig to the water surface. Ice temperature under the rig was also monitored. As can be seen the temperature in the ten feet of artificial ice remained at or below -10°C at both wells during drilling (Fig. 17). The upper few feet at Drake were colder than at Hecla because the ice platform was completed several weeks before the rig was moved on. The temperature increases from -10°C at the interface of the artificial and natural ice to about -2°C at the ice/water interface. The water temperature (Fig. 18) remains fairly constant of -2°C from surface to ocean floor 400 feet below the ice surface although a slight
warming tendency is noticeable commencing 30 feet above the ocean floor at Drake. Water salinity at Drake remained relatively constant at 30,000 ppm. No salinity data was taken at Hecla. Horizontal ice movement was also monitored. Only 2 feet of movement occurred at both wells.

**DRILLING PROGRAM**

The Arctic Ocean tide is very small in this area, about 1-1/2 feet at W. Hecla N-52 and about 2-1/2 feet at East Drake I-55. The drilling program was similar at the two offshore wells. The first string of casing was set about 1,000 feet below ice level and the second string at about 2,500 feet (Fig. 11). The gas sand was encountered at 2,700 feet in the Hecla well and at 3,500 feet in the Drake well. Total depth at East Drake I-55 was 3,900 feet. Both wells turned out to be excellent gas wells which significantly extended proven field limits and added several trillion cubic feet to the proven reserves in the Arctic Islands.

**FUTURE OFFSHORE DRILLING PLANS**

Many technological advances have occurred in the Canadian Arctic over the past 6 or 7 years but undoubtedly the Ice Platform drilling system is the most significant development ever made in the short history of Panarctic Oils Ltd. This system of offshore drilling is economical, safe and efficient. These two tests were drilled as expendable wells and were plugged and abandoned after evaluation of the gas reservoir. In 1976 plans are to drill 2 more offshore wells which should extend
the Hecla and Drake Point fields 20 miles offshore (Fig. 7). These wells will be drilled in about 1,000 feet of water. In 1977 it is possible that a gas well will be completed on the ocean floor and a flowline run to shore for extended production testing.

LIMITATIONS OF ICE PLATFORM DRILLING SYSTEM

This ice platform offshore drilling system does have some serious limitations because it can only be used for four or five months a year and cannot be used in areas where ice movement exceeds 5 percent of water depth. In addition, the 4 to 5 month time constraint limits drilling depth to about 10,000 feet maximum. Nevertheless, most of the gas fields discovered in the Arctic Islands are shallow. It is believed that this drilling system will discover several new gas fields while industry is developing a new drilling system that can operate year round.

ARCTIC DRILLING SYSTEMS OF THE FUTURE

Here are models of two Arctic drilling systems of the future. The first (Fig. 19) is an air cushion drilling barge designed by Arctic Engineers, a subsidiary of Global Marine. This system can be towed over water or ice and should be able to operate in the Arctic Islands for 6 or 7 months a year. A prototype air cushion vehicle capable of carrying 100 tons has already been field tested in the Canadian Arctic. The next system is one which should be able to operate 12 months a year (Fig. 20). This is a semi-submersible ice-cutting drilling vessel. It is a monopod
TIME FROM BEGINNING OF FLOODING

FIGURE 15 - RATE OF ICE BUILD-UP

FIGURE 16 - ICE PLATFORM DEFLECTION UNDER RIG LOAD

FIGURE 17 - ICE TEMPERATURE PROFILES UNDER RIG WHILE DRILLING

FIGURE 18 - SALINITY AND TEMPERATURE OF ARCTIC OCEAN WATER
FIGURE 19—PROPOSED AIR CUSHION ARCTIC DRILLING SYSTEM

design with the lower hull below the ice level and the upper hull, where the crews and drilling equipment are located, being well above ice level. The vertical column would be equipped with a rotating sheath fitted with ice cutters so the vessel can remain stationary in advancing ice. This vessel would be dynamically positioned and self propelled. It is designed to cut through pressure ridges 55 feet thick and travel through multi-year ice from one drilling location to another at a speed of 4 knots. The designers, Sedco Inc. of Dallas and Sea-Log Corporation of Pasadena have done model testing and field testing of the Ice Cutter at Resolute Bay. Development of this drilling system is being financed by nine Canadian oil companies and it looks very promising. There are two main drawbacks to each of these proposed future drilling systems. First, they are very expensive, probably in the range of 100 million dollars. Secondly, they are still at the design stage. With the long lead time needed in shipyards today they cannot be made available for at least another 4 years.

CONCLUSION

A year-round offshore drilling system is needed for the Arctic Islands in order to develop the threshold gas reserves to justify the high capital cost of the Polar Gas pipeline (Fig. 2). It is the hope of Panarctic that the Ice Platform drilling technique will bridge the gap in discovering offshore gas fields over the next four years until the date that year-round offshore drilling does become feasible.
FIGURE 20—PROPOSED ICE-CUTTING SEMI-SUBMERSIBLE DRILLING VESSEL
SECTION 12

EDUCATION FOR OFFSHORE TECHNOLOGY

Sackinger, W. M.

*Education for Offshore Technology*

(University of Alaska, United States)

Gaither, W., T. Murray, T. Patten and P. Bruun
Discussion

Patten, T. D.

*The Evolution of an Offshore Engineering Program*

(Heriot-Watt University, Scotland)

Tryde, P., P. Bruun and L. Bengtsson
Discussion

Murray, T. E.

*Curriculum Development for Offshore Technology: Possible Role for Sea Grant*

National Oceanic and Atmospheric Administration, Washington D.C., United States)

Patten, T. and W. M. Sackinger
Discussion

Bruun, P.

*Educational Patterns in Port and Ocean Engineering in Scandinavian Countries*

(The Norwegian Institute of Technology, Trondheim, Norway)

Godfrey, W. R.

*The Thalatect: His Purpose and Capabilities*

(University of Notre Dame, Indiana, United States)

Sackinger, W. M., P. Bruun, T. E. Murray, and T. Patten
Discussion
Alaska possesses 6,640 miles of general coastline, and 830,000 square miles of continental shelf area (74% of the United States total). Most of this shelf can be categorized as sub-Arctic or Arctic in that the hazards of sea ice, as well as severe winds and waves, are often encountered. Two major industries exploiting resources offshore are fisheries and petroleum, with additional limited mineral extraction by dredging in selected shallow waters.

Education in fisheries technology is a formal two-year degree program at Kodiak Community College, and is also the subject of numerous short courses held at fishing communities throughout the state by the University of Alaska Cooperative Extension Service. Subjects include safety in operations, survival techniques, mechanical equipment operation and repair, communication and navigation equipment operation, fishing gear choices and operational methods. Several vital elements of the fishery operations are taken from existing services; for example, the design and construction of fishing boats is done by shipyards outside of Alaska, as is the design and construction of seafood processing plants. Weather prediction is the responsibility of the National Weather Service, while search and rescue is the mission of the Coast Guard. The subject of fish location is almost totally neglected, although the broader topic of fisheries biology is the focus of a new advanced educational program at the University of Alaska, Juneau campus, aimed at the Bachelor's and Master's level. Some research is being directed into the legal and economical aspects of non-profit fisheries cooperative organizations, an emerging business form for marketing and possible hatchery operation. The knowledge acquired in this research, sponsored by the Alaska Sea Grant program with both federal and state funding, will eventually be made available through the University of Alaska educational system.

The fast-growing offshore petroleum industry makes full use of the sophisticated technology developed and used for offshore operations throughout the world. If any generalization could be made, it is that experienced professional and technical personnel are transferred to Alaska in the initial phases of any such effort, and if a more permanent demand is apparent in any skill area, educational programs develop locally to fill the need. These are usually on-the-job training programs or union apprenticeship programs, with some formal classes superimposed upon heavy practical experience. Some specialties in which this is generally true include geophysical exploration, drilling operations, offshore platform services such as mud and mud logging, cementing, line logging, energy sources, water and waste systems, food service, mechanical maintenance.
and repair, equipment supply, communications, navigation, aircraft support, and diving services. The design and construction of equipment for offshore activity, such as platforms, semi-submersibles, drilling ships, and supply ships, is executed by specialized marine architectural firms and shipyards on the south and west coasts of the U.S. by engineers educated in the traditional disciplines of mechanical, electrical, civil, and metallurgical engineering and who have experience in the marine engineering business.

Engineers from both the petroleum industry and the marine construction industry take advantage of the two-week School of Offshore Operations, offered twice each year at Baytown, Texas, sponsored by the International Association of Drilling Contractors (IADC). This course covers a broad spectrum of topics at an introductory graduate level. Included are such subjects as oceanography, meteorology; soil mechanics; marine drilling units (submersibles, jack-ups, semi-submersibles, and ship shapes); buoyancy, stability, and mooring; planning for exploratory drilling; marine riser and motion compensation; exploratory drilling; tension measuring and recording devices; diving equipment and services; subsea completions. All of these subjects are treated briefly during the first week of the course. Subjects covered in the second week are platform foundations for offshore drilling; production planning; production safety controls; pipelines in the sea; workover operations offshore; cathodic protection; special coatings and paint; marine insurance; oil spills - containment and removal; special regulations; economics; and safety. It is clear that the breadth of the subject matter to be covered is so great that a full year - or a lifetime - could profitably be spent on the above subjects, but this is left for the student, on his own initiative, as he pursues the voluminous notes accompanying the course. Most of these topics apply equally well in all offshore areas of the world, but in Arctic waters the presence of severe storms, high winds and waves, sea ice, and subsea relict permafrost represent special conditions which also must be considered.

Such special Arctic environmental factors are treated as parts of University of Alaska graduates courses in Arctic engineering, ocean engineering, and environmental quality engineering which are offered in Anchorage, Fairbanks, and Juneau as demand warrants.

The design and construction of subsea pipelines, tanker loading and docking facilities, petroleum storage, general marine and air terminal facilities, petrochemical, refining, and LNG plants is invariably on a turnkey contract basis with an international firm experienced in that field. Time schedules during construction leave no time available for the formal education processes. During operation of such facilities, however, the local personnel do attempt to improve their skills by additional formal education. At Kenai Community College, a two-year program has been tailored to those students interested in production operations on platforms and in petrochemical, refining, and LNG plants ashore. For engineers evolving into management responsibilities, the Engineering Management Program offers a Master's degree which has been quite popular among petroleum industry engineers in the Anchorage area. The more advanced topics of reservoir control, land and leasing policies, marine insurance, marine ecological monitoring and oil spill contingency planning are usually taught in specialized schools within the industry itself.

Clearly, the expansion of activity in the Outer Continental Shelf areas of the United States will require the services of many highly-skilled individuals. Depending upon the results of exploration offshore, as much as half of the domestic production of the United States could eventually come from the Continental Shelf areas. As offshore discoveries are made, it is imperative for educational institutions to respond to the inevitable need for educated engineers and skilled technicians to cope with the broad spectrum of disciplines necessary in the offshore environment. As a first step, a Master's degree program could be offered, including the material of the School of Offshore Operations mentioned above. Several specific two-year programs in offshore drilling technology, offshore services, and offshore production could be developed. An assessment of industrial demand in future years for graduates of such programs should be done to
optimize the scale, timing, and geographic location of these educational offerings. By careful planning, the skilled personnel required for offshore development can be available when needed, and the wise and safe development of our offshore resources can be optimized.
DISCUSSION

W. Gaither: How much does Sea Grant contribute in the extension program for education for fishermen?

W. Sackinger: I do not recall an exact value, but I think it is approximately $230,000.00 per year including federal and state contributions; and this supports people who go out and offer courses in fishing technology in a large number of villages. In regular, structured marine programs, a total of $225,000.00 is budgeted in the current Sea Grant program.

T. Murray: Has Alaska been developing a senior college, or anything like it, to give additional training to graduates of the community college two-year experience?

W. Sackinger: That has been the procedure in the Anchorage area. The Anchorage area originally started with a community college two-year program, and then it added on later a senior college, a four-year program, and a graduate program. Historically, in Fairbanks, the four-year school existed here first, and it is only in the last two years that the community college has been formally organized and has gotten under way. So the four-year college in Fairbanks is now trying to coordinate with the community college so that graduates of the community college can go on, if they wish, in the four-year program. In Juneau, the community college is relatively new and is offering programs there. The senior college in Juneau also is growing. The other community colleges do send some of their graduates to a senior college, but it is relatively less often that this happens.

Anonymous: There was nothing in your abstract specifically devoted to the question of the legal, social, and economic side of Alaskan resource development. Do you feel any pressure for having what might be called marine resources programs that would possibly help with environmental impact assessments from the social acceptance point of view? Can you comment on that?

W. Sackinger: Yes, I do mention just briefly that some research is being directed into the legal and economic aspects of non-profit fisheries cooperative organizations, which is an emerging business form for marketing and hatchery operation. This is about as close as the University is getting to the question you ask; that is, of the social, economic, and legal environment for offshore operation. I think that the University considers it a research activity rather than an instructional activity. The idea of offering educational services for people who wish to go into the marine resource management and resource planning fields has not yet been carefully reviewed. Anything that you could mention along those lines would be very helpful in estimating the extent to which this field is growing and the kinds of skills people need to proceed in resource management. We might be able to develop an option in some existing programs that would assist people in that direction.

Anonymous: There are two conflicting points of view on this; or not conflicting, I guess, but they are divergent paths. One is to say that engineering and technology is the liberal education of our century now; the other is to take the tact that the philosophers and scientists take, which is that it is really necessary to civilize the technologists. That is often done from the point of view of a non-technical background but more of a liberal studies background. I do not have any clear answers for us, but I certainly know that there is a great deal of importance that will be associated with the legal and economic side of things in connection with the application of technology.

W. Sackinger: I think the idea of offering a sequence of technical courses for people who are primarily oriented along the social, economic, and political aspects or planning
aspects is one possible way - technical courses that give perspective but not necessarily design expertise. Then the alternate approach is to take people educated in the design side to the point where they could do design, but they may be interested more in the planning aspects. Then you could try to give them more exposure to the liberal arts side of the planning process.

T. Patten: I think if our experience in the U.K. was of any value it was that for a short period of time there was extreme - overheating would be the way you would describe it - on the planning side; and there was a period of time during which, if someone could have moved in quickly, there would have been a general benefit if there could have been refresher courses for legal practitioners on the changes in the planning laws. We seem to have gotten over such time expediencies because of the uncertainties and delays in resource development.

W. Sackinger: Well, it appears that there is a possible direction for further progress here; that is in providing assistance in developing expertise among the planners in both the technical and non-technical aspects of their activities. It sounds like it needs to be a quick-reaction educational approach.

Anonymous: When Guy Martin was talking the first day, he talked about those kinds of considerations, and I wonder if the State of Alaska has any instructive or organized approaches to evaluate the social impacts or the economic impacts of, say, offshore oil development in the fishing communities and things like this. I got the impression that they just did not know those answers.

W. Sackinger: Someone here perhaps can answer better than I, but the State of Alaska has a planning office, a part of the Office of the Governor. Some of the people on that staff are planners oriented along the lines of coastal zone management. Coastal zone management is just growing in this state, and there is a search for people who are talented in coastal planning and could serve in the state government. There is a great need for some kind of federal support for this kind of planning. Guy Martin has held for some time that for outer continental shelf planning, which is federal leased land, federal support for coastal planning should be available.

P. Bruun: In Norway we established a year ago what we call a coastal department, which actually combines different departments we had earlier. This department runs everything on the coast. There are courses now in Norway that try to put together all those so that everybody knows what the other is doing. We have not had too much concern about environment, but, as you know, we have Mr. Bugge who is the Director of the Environmental Department in Norway. He is here at this meeting, and they are concerned with all kinds of environmental problems, but they work closely together with this coastal department.

W. Sackinger: It seems that the time for planning always seems too short. When the day finally comes that someone realizes that planning is needed, it is already something that should have been done a year ago; so you need someone already trained and capable of going ahead very quickly with the planning process. The educational process has traditionally been a rather ponderous one. It is geared to operate over time scales on the order of one or several years. Perhaps the educational process might try to act on a shorter time frame, of the order of months or weeks, to bring specific material in front of coastal zone planners or managers.

P. Bruun: I have a comment which does not refer to anything you have said, but actually to the planning which is being done down in Florida and the Carolinas. They are now passing a great number of laws; tideland law is one of the most important ones. This gives specific advice on how to handle problems of the tidelands.
ABSTRACT

The recent development of offshore engineering activity in the United Kingdom stems from the discovery of major oil reserves in the UK Continental Shelf and its impact is partly due to the technological difficulties associated with the weather and the water depths. Some universities and technical institutes throughout the country have introduced new course material in relevant special areas; in most cases this initiative has arisen from within one department of a university and the interests range from geology through medicine, law and economics, to a specific branch of engineering.

Physically, Scotland is the base for UK offshore oil activity. Edinburgh is located behind the front line of oil service operations and vies with London for leadership in financial matters related to new ventures in North Sea oil. At Heriot-Watt University, engineering is a major activity and when the strong urge to contribute to the emerging activity first arose in the University it was decided to concentrate initially on engineering aspects. Relevant offshore involvement also now exists in the areas of science, economic and social studies and environmental studies.

Largely due to the recognition by individual faculty members of the potential merits of a concerted academic attack the situation has been reached where, in addition to the provision of minor options in Offshore Engineering, there exists a course which permits a student to graduate B.Sc. in Offshore Engineering while retaining a strong association with one of the existing Engineering Departments. Higher degrees in Offshore Engineering are obtained primarily through research. A course of study leading to the degree of Master of Engineering in Petroleum Engineering will commence in October 1975 and successful candidates will gain a detailed understanding of the three main upstream engineering activities, namely reservoir, drilling and production engineering. An outline is given of the composition of these courses.

The Institute of Offshore Engineering is an integral part of the University and in addition to becoming a source of engineering expertise and information the Institute aims to collaborate with industry in cooperative research/development projects. The Institute's main educational role has been in the post-experience field with short and very specific courses at professional level. Whenever reasonable and possible the Institute has assisted in establishing the academic courses already mentioned. Similarly, the Institute gathers strength for its research, consultancy and other activities through the collaboration of Faculty staff.
At this instant the needs of oil and gas inject the urgency and sustain the momentum but for the coming generation of graduates the future challenges offshore are likely to be for more extreme environments and could have completely different objectives.

INTRODUCTION

We are fortunate in having as Session Chairman, Professor Bill Sackinger, and I wish to take this opportunity in congratulating him for his part in mapping out the technical content of this POAC conference. His opening remarks were addressed to some aspects of offshore technology, particularly in the context of Alaska and in talking about education he also included training, as one should properly do. If I make little explicit reference to training, it is not because I regard it as unimportant but because there is inadequate time to give comprehensive coverage. On the other hand, I shall underline my view that at University level, education and research are contemporary and connected activities, particularly in the context of new offshore engineering developments, since a direct participation in research is necessary to produce the knowledge and experience required to provide the opportunity for education.

The principal aim of my presentation is to give a broad outline of the Offshore Engineering activities at Heriot-Watt University. Separately, our various activities may mirror those of other establishments, but the combination is rather rare, if not unique. The earliest identifiable developments occurred about 1969 and I shall try to confine myself to itemizing the various components of the current program.

This panel also has representatives of educational activities in the United Kingdom and in the Scandinavian countries. To be fair to relevant education in the rest of the world, not specifically represented on the panel, I feel obliged to mention Holland and the Technische Hogeschool of Delft and also France where engineering courses of some relevance take place at the Ecole Nationale Superieure de Techniques Avancees (ENSTA) and at the Institut Francais du Petrole. In Japan, at Kagoshima, Tikai and Tokyo Universities, Ocean Civil Engineering is an area of considerable interest. In Canada relevant engineering interests appear to exist in the University of British Columbia, Queens University, Nova Scotia Technical College and in Memorial University.

It appears to me that in most of the cases already mentioned the offshore engineering developments have arisen through recognizing a relevant strength within a specific department and capitalizing on it. This is certainly the overall situation in the United Kingdom. Notable examples exist at University College, London, Liverpool and Manchester Universities, University of Birmingham, Cranfield Institute of Technology, Robert Gordon's Institute of Technology and at Glasgow College of Technology. It is left to the reader's discretion to include in this list those various Universities having degree courses in Naval Architecture.

Physically, Scotland is the base for UK offshore oil activity. Currently the front line extends from Dundee to the Shetland Isles with major concentrations at Aberdeen and Peterhead. The highly-industrialized central belt has built up around Glasgow, now the base of the Offshore Supplies Office, the techno-commercial arm of the Department of Energy, and the future home of the British National Oil Corporation.

At the eastern end of the central belt, and on the Firth of Forth where new offshore platform fabrication and oil transportation facilities are to be found, lies Edinburgh which in addition to being the focus for the administration of Scotland is the centre of considerable financial support for North Sea ventures.

Heriot-Watt University, in Edinburgh was born the Edinburgh School of Arts in 1821 and is now establishing its fine facilities on the City boundary within sight of the origin and much of the remnants of the shale oil industry. At the University, engineering is a major
activity along with science, economic and social studies and also environmental studies. The decision by existing engineering departments to collaborate in providing a focus for offshore engineering activity was taken consciously and was based to some extent on the existence of a number of relevant research projects. In my opinion the three factors which ensured a relatively easy take-off for the venture were as follows:

1. Because of our relative infancy as a University interdepartmental boundaries were not barriers;
2. sufficient enthusiastic and talented staff were strategically located throughout the various departments; and
3. the need within the University of a viable eclectic study area such as offshore engineering with its many cross-disciplinary links.

Very quickly we observed the enthusiastic intellectual response among those senior undergraduate students involved in design, analysis or experiment, on aspects of an offshore engineering project.

A new urgency was injected when in 1971, further giant oil fields were confirmed in the UK sector of the northern North Sea. As a result a large market was created for short technical post-experience courses and seminars for engineers already in industry and we also recognized the longer term needs for qualified manpower with an understanding of the offshore environment and the associated forces.

OFFSHORE ENGINEERING PROGRAM

The present content of the offshore engineering educational package at Heriot-Watt University is summarized in Table 1; all of the items listed are under way, although (2) and (4) have not yet completed one full cycle. The remainder of this presentation will expand slightly on the individual headings tabulated.

TABLE 1. Offshore Engineering Program.

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<tr>
<td>1.</td>
<td>B.Sc. (with minor option in Offshore Engineering) in:</td>
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<td></td>
<td>Building</td>
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<td></td>
<td>Chemical Engineering</td>
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<td>Civil and Structural Engineering, and</td>
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<td>Mechanical Engineering</td>
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<td>2.</td>
<td>B.Sc. in Offshore Engineering</td>
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<td>3.</td>
<td>M.Sc. (by course and research) with emphasis on Offshore Engineering aspects</td>
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<td>4.</td>
<td>M. Eng. in Petroleum Engineering (by course)</td>
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<td>5.</td>
<td>Ph.D. (by research) in specific Offshore Engineering problem areas</td>
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<td>6.</td>
<td>Specialist Post-Experience Seminars and Courses</td>
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For the many people who believe that a good basic engineering education is necessary and sufficient the University would draw attention to the many good graduates in the traditional engineering areas, some of whom will have a background introduction to Offshore Engineering through a minor option. For example, a Civil Engineering graduate might have studied a substantial amount of ocean engineering and external hydraulics. For clarification it should be emphasized that in Scotland, Honors Degrees courses in Engineering take
four years and at Heriot-Watt University, the ordinary Degree course in Engineering also takes four years of full-time study. Engineering Bachelor Degrees are available in Building, Chemical Engineering, Civil and Structural Engineering, Electrical and Electronic Engineering and Mechanical Engineering.

(2) B.Sc. in Offshore Engineering

We have also initiated a course leading to the Bachelor Degree in Offshore Engineering. Understandably this was greeted with some criticism most of which we can counter with the claim that instead of being a more specialized engineering course it contains engineering material not normally available in the traditional courses and that it should be sufficiently interesting and exciting to draw to engineering studies, students who otherwise would have opted for non-engineering courses.

For any one student the course is structured around core material from one of the available and traditional engineering branches already mentioned, complemented by five full Offshore Engineering subjects up to Honors standard.

Ideally, entry to this new program has been by transfer of selected applicants at the end of the second year of undergraduate studies. The first few Bachelors of Offshore Engineering can be expected in 1976. We have also succeeded in negotiating the first hurdle of professional recognition for this course.

The titles of the specific Offshore Engineering subjects are listed in Table 2 along with a diagrammatic indication of the general structure of the studies. In the Fourth Year of the forthcoming session Technical Options will contain a design study for a concrete production platform complete with items of the associated production equipment. For the same year the Project will be a group project and will be based on the design and test of an instrumented buoy.

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<th>TABLE 2. B.Sc. in Offshore Engineering</th>
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### Subject material allocation on a percentage basis.

Parent Engineering Studies are based in one of the following Departments:
- Building
- Chemical and Process Engineering
- Civil Engineering
- Electrical and Electronic Engineering, and
- Mechanical Engineering

Offshore Engineering Subject Titles:
- **Third Year**: Environmental Design, Measurement and Control
- **Final Year**: Dynamics of Fluids and Particles, Management of Offshore Activities, Technical Options, Project

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(3) M.Sc. (by Course and Research)

In this context the emphasis of the program will be on Offshore Engineering aspects whether the student is based in an existing department or in the School of Offshore Engineering. (The possibility of offering a course in Offshore Engineering leading to a Master's degree was widely discussed but further action was deferred pending achievement of the other related academic objectives.)

For one student the research component of his work could be a contribution to research into the fluid loading of structures in the Department of Civil Engineering, into ship roll motion modelling in the Department of Mechanical Engineering or into underwater plasma studies in the Department of Chemical and Process Engineering.

(4) M.Eng. in Petroleum Engineering (by Course)

In 1974 the UK Government selected Heriot-Watt University and Imperial College, London, as the two centers for education in Petroleum Engineering. In the former case the emphasis will be more towards the production end with particular reference to offshore aspects. (To avoid any confusion it is important to add that Aberdeen University is the locus for a Master's degree course in Petroleum Geology.) The course is designed for students with an Honors degree in an engineering subject and 10 students have already been selected to commence the first course in October 1975. Professor Jim Brown and his supporting Petroleum Engineering staff have been appointed. The Government had provided recurrent funding as well as funding for the new building to be operational in 1976. Target student numbers are 20 to 30 but for the first course the number has been restricted to 10 for practical reasons.

The main subject headings are shown in Table 3 from which it may be deduced that the coverage is rather broad - nevertheless, we hope to achieve reasonable penetration in the intensive one year study period. Oil company support for this course has been through offers of scholarships and a few cash donations.

<table>
<thead>
<tr>
<th>TABLE 3. M. Eng. in Petroleum Engineering</th>
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<tbody>
<tr>
<td>Main subject headings:</td>
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<tr>
<td>Reservoir Engineering</td>
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<tr>
<td>Drilling Engineering</td>
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<tr>
<td>Production Engineering</td>
</tr>
<tr>
<td>Technical Supporting Subjects</td>
</tr>
<tr>
<td>Marine Technology</td>
</tr>
<tr>
<td>Petroleum Economics and Regulations</td>
</tr>
<tr>
<td>Project or Dissertation</td>
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</tbody>
</table>

The academic structure devised for the new degrees in Offshore Engineering and Petroleum Engineering has been the formation of the School of Offshore Engineering. As a result it is theoretically possible to ensure the collaboration of staff within existing departments with a minimum of organizational complications.

(5) Ph.D. (by Research)

As in the case of research studies leading to the Degree of M.Sc. (3), Ph.D. work can be associated either with a Department or with the School of Offshore Engineering. A suitable example is research into the de-entrainment of liquid droplets from a gas stream. In the marine context this is a major problem related to the design of improved ship gas turbine intake systems and over recent years an understanding of the performance...
characteristics of demisters has been gained in the Department of Mechanical Engineering. Contributions to this understanding have been made by graduate students and new and improved configurations have resulted.

(6) Specialist Post-Experience Seminars and Courses

The University, particularly in its engineering departments, has an orientation and willingness to work with industry. When in 1971 it was appreciated that many industrial companies with past University associations had a potential to become suppliers to the offshore engineering patch, early priority was given to mounting residential seminars on specific technical matters aimed principally at senior management and technical staff from industry. This activity has built up over three years and Table 4 lists courses and seminars in planning for November on.

<table>
<thead>
<tr>
<th>TABLE 4. Specialist Post-Experience Seminars and Courses</th>
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</thead>
<tbody>
<tr>
<td><strong>Session 1975 - 1976:</strong></td>
</tr>
<tr>
<td>Use of Explosives Underwater</td>
</tr>
<tr>
<td>Single Point Moorings</td>
</tr>
<tr>
<td>Subsea Equipment (Diagnostic Course)</td>
</tr>
<tr>
<td>Pipeline Design</td>
</tr>
<tr>
<td>Advanced Reservoir Simulation</td>
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</tbody>
</table>

INSTITUTE OF OFFSHORE ENGINEERING (IOE)

The above post-experience activities are the responsibility of the Institute of Offshore Engineering. Established in November 1972 through a generous grant from the Wolfson Foundation, the Institute has the declared aim to meet speedily and effectively the needs of the ancillary industries in oil exploitation by establishing a source of specialized knowledge in offshore engineering to the advantage of the participating companies and the primary industry. The principal functions envisaged for IOE are listed in Table 5.

<table>
<thead>
<tr>
<th>TABLE 5. Institute of Offshore Engineering</th>
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<tbody>
<tr>
<td>Main functions in the broad field of Offshore Engineering:</td>
</tr>
<tr>
<td>(1) To provide information through reference to available publications.</td>
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<tr>
<td>(2) To provide information through seminars, colloquia and conferences.</td>
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<tr>
<td>(3) To provide information through consultancy.</td>
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<tr>
<td>(4) To obtain information through collaborative research and development.</td>
</tr>
<tr>
<td>(5) To feed back advice to the University on possible academic developments.</td>
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</tbody>
</table>

In practice consultancy is usually provided through a team made up from IOE and academic staff. The aim is not to enter into direct competition with commercial consultants but to provide the kind of mixture of expertise not normally available. Having assisted in establishing the various degree courses already referred to, the Institute is giving priority to building up a program of collaborative research. Wherever possible, faculty staff combine with IOE staff to form the project team, with IOE retaining the management role and responsibility. For example, the Institute has the contract from Occidental of Great Britain for the baseline study of the marine environment in Scapa Flow and in the area of the ballast water outfall at Flotta in the Orkney Isles. In this instance the IOE Marine Science Unit works under the direction of the University's Senior Lecturer in Marine Biology.
An interesting example of collaborative research which is supported by the Directorate of Naval Oceanography and Meteorology, is in the charge of a Naval Officer, with sponsorship from the Naval Marine Wing of the National Gas Turbine Establishment, a Department of Director General Ships, and a commercial company. The purpose of the marine aerosol study is to investigate the salt content of the air near the sea surface as a function of meteorological factors. Weatherships in the Atlantic Ocean are the source of the aerosol samples which are analysed in the IOE laboratory for sodium content. Further assistance is provided by the Department of Meteorology of the University of Edinburgh and by the Departments of Computer Science and of Mechanical Engineering of Heriot-Watt University.

The ultimate of the IOE objectives is for some of the experience and technology derived through this kind of collaborative research to be fed back into education.

CLOSURE

Of necessity this has been a hurried and selective presentation. Hopefully, academic colleagues in the UK and elsewhere will appreciate that it has not been possible to give full coverage to their particular activity.

The conventional wind-up to this topic would be a statement of the needs for highly-qualified engineering and scientific manpower to deal with today's offshore problems, with the bigger problems still to be faced, of which many face us here in the Arctic. Only last month a UK Parliamentary Select Committee reported to the House of Commons, that they were not impressed by the priority given by the Government to the problems of higher education related to oil and gas exploration and development. It seems to me that industry and government should combine to try to give guidance on manpower needs. In its absence I sustain myself with an extrovert and optimistic belief that academic, industrial and social benefits will follow from grappling enthusiastically with those problems which are consistent with our range of skills and further stretch our knowledge and understanding.
DISCUSSION

Anonymous: I wonder if you could say a little bit about your research park. Do you lease land, sell land, set standards for the types of business that industries do that might locate in your research park. Could you elaborate?

T. Patten: We lease land. We control the kind of building that goes on; we have a say in the activities that go on. Because of the fact that it is a green belt zone, it can only be research; not manufacturing. We like to choose incomers who would benefit from being in close association with the University, and the financial fact of research is that there is no great incentive for them to come unless there is such a benefit. The latest, biggest incomer has been Syntex, which is an American pharmaceutical operation; they will set up a European research organization there.

Anonymous: I have a question concerning Angus. Has it been used to inspect pipelines in the North Sea?

T. Patten: It has not been used to inspect pipelines. It has been used to inspect a pipeline but not an oil pipeline.

Anonymous: Is it capable?

T. Patten: It is capable, but it is a limited piece of equipment. It is a sort of on/off control system; not one of the very fancy systems like Hydrotech.

Anonymous: In other words, it will not, it could not replace the conventional two-man inspection submarines that are being used in the North Sea.

T. Patten: Well, for some things it could. In a low-priority mission, there are many things it could do. Its latest task was for a marine biologist who wished to study sand dunes. After a week they decided that every marine biologist should have one.

Anonymous: How big a support package is required to operate it?

T. Patten: It could easily work from a 50-foot boat. Every module is man-handled.

Anonymous: Are you familiar with the containers that are used in the North Sea to transport equipment out to the rigs, vessels, and barges? Would the whole unit fit in one of those containers?

T. Patten: Easily.

W. Sackinger: You mentioned the chartered engineer recognition of your graduates. In the United States, the law of a state prescribes the board of examiners to determine whether an engineer should be registered and called a professional engineer. In Canada, the power is given over to an engineering society. How is it in the U.K.?

T. Patten: It is more like the Canadian system. I think in Canada the provinces have something to say about it too, don't they? There is some control and authority, too, from one state to another; but it is the professional society, either mechanical, electrical, or chemical, that evaluates programs for professional acceptability. Their findings are finally ratified by a body called the Council of Engineers. The situation is that the Institute of Mechanical Engineers have agreed to sponsor this degree course to go to the Council of Engineers. If they once accept it, and to the best of my knowledge they might well have done that by now, then it is acceptable in all of its forms.
W. Sackinger: So it is a mechanical registration or certification that is given?

T. Patten: No, after it is accepted by the Council, it will be accepted because of the civil part or the electrical part or the chemical part.

Anonymous: If it is of general interest, perhaps you could make a little clarification about the honors course and masters degree. Could you make a clear distinction?

T. Patten: They are separate in a way. It is normally assumed, I suppose, that the masters degree candidate or applicant should have had a first degree; and in the past, this would normally have been an honors degree. The honors degree student would normally have a dominant talent in analytical work, and that is often the way that the masters degree works as well.

P. Bruun: Gentlemen, as Dr. Sackinger said, Scandinavia is some kind of United Scandinavian States. Three of us are present here, representing Denmark, Sweden, and Norway. Mr. Tryde from Denmark is going to discuss offshore education in Denmark.

P. Tryde: As to the ocean engineering part of education, I would say that very little has been done in Denmark on the educational part, which is simply due to the fact that very little oil has been found in the Danish sector of the North Sea. That means we have not had any pressure on the industry to develop the Danish sector. Research has been done, and that again is not even nearly as much as has been done in Great Britain and in Norway. Unemployment of engineers is the situation that comes up in Denmark. The building industry is in trouble, which means that many engineers are out of a job; but they have specialized in the building industry and have to be re-educated. The Danish Association of Civil Engineers have taken up continued education courses in ocean engineering, and we have been very fortunate that Professor Patten in Edinburgh has provided some of these. I must say, very excellent courses. I think we are going to continue on that line; and that, of course, also goes with our Norwegian counterpart. So far, we are now trying to work together with the Norwegians and with Great Britain on these matters in education. We have been doing in Denmark quite a lot at the laboratory and at the Institute of Hydrodynamics and Hydraulic Engineering. We have been working very much with waves before all this started; and, of course, that has been continued and intensified on many of the basic research problems. The University in Denmark is mainly concerned with basic research. Very little applied research or engineering research is being done directly at the University. That is done by what we call the Danish Hydraulic Institute, and that is part of a group institute which is called the Danish Academy of Technical Sciences. At the Danish Hydraulic Institute, they have about sixty people—thirty scientific and thirty assistants—working all over the world now doing coastal engineering jobs. They are working in the near east, Kuwait, for example. They have a contract there for investigation of a proposed dam. I think that is about one million dollars. They have several projects in Africa. These institutions are initiated by the government, but the state only guarantees the beginning of the institute. It is a non-profit organization, which means all they earn will have to go into research. Of course, if any clients want something done, this material might be proprietary like in any other country and not be passed on. As to the qualifications of engineers, in Denmark the entire university education is based on the masters degree. We do not have any qualification system like you have. Once graduated from the university, an engineer can do any kind of work and have the responsibility for it. The number of engineers graduating in civil engineering is about 150 each year. We have a limitation on the number of students, 150, and there used to be about 300 openings; but now it is 145 and it is not fashionable to be an engineer in Denmark at present. Of course, the tendency now is, probably like what happened here, that the students will go up again because the environmental laws will work. Another thing is Greenland. Drilling concessions have been given on the west coast of Greenland in the offshore area for drilling, and the American and Canadian companies have been starting...
to make investigations this year. We hope that will feed back to the Danish institutions. I think that is about all I can say in this connection. I will be glad to answer any questions.

W. Sackinger: I have one question about your master's degree and the professional qualifications. Does the master's degree give an engineer the right to practice his specialty then and give him the legal liability for malpractice and so forth?

P. Tryde: That is right. If he is a consulting engineer, he will probably belong to an Association; but he does not necessarily have to. In other words, he is legally qualified for all state and community work.

Anonymous: Is the public sentiment tending to support or anticipate that Denmark will unilaterally adopt a 200-mile zone and control all of the resources in it offshore?

P. Tryde: I could not answer that question. I do not know, really. You realize that Greenland, which is really fifty times as large as Denmark in Europe, has less than 50,000 people. Still they certainly will have some influence on what is going on. The Danish government is very much concerned about what is going on there. There have been very strict rules as to the drilling and exploration operations outside of Greenland. For instance, you are not supposed to make any fixed kind of platform. You have to use floating types, and they have to be dynamically positioned.

Anonymous: The reason I mentioned it is the fact that if it does go through and if Danish engineers or administrators are expected to administer this really huge area, I would expect the manpower demands coming from the kind of courses you teach would have a severe strain on them today.

P. Tryde: That probably would be correct, but I also expect that, like in Norway, many Americans, Canadians, French, and people from other countries that have worked in these fields, will go there; and many of the first ones will probably be not from Denmark but from other places.

P. Bruun: I think maybe we could continue with Sweden now. Dr. Lars Bengtsson comes from the northernmost technical university in Sweden, the University of Lulea.

L. Bengtsson: Specialized education on the graduate level is proved by four technical universities: the Royal Institute of Technology in Stockholm, the Chalmers Institute of Technology in Gothenburg, and University of Lund, and the very new university which I represent, the University of Lulea. All four universities give compulsory courses on hydraulics and applied hydrology, but there is not very much in ocean engineering. However, at the University of Lulea we include stream hydrology and fluid mechanics in the course on applied hydrology. In courses on water construction, coastal engineering is included. One can take courses that I would call coastal engineering; that is, courses on pollution problems and disposal of sewage into coastal areas. There is only one more advanced course on coastal ocean engineering and it is provided by the University of Lund in the south of Sweden. There are courses on transfer processes in which the students are introduced to mathematical models on circulation and special dispersion models. In that course, exploitation of the sea is also included, especially utilization of sand and gravel, because the land resources are exhausted in Sweden now, and the environmental aspects are very much stressed there. At my university, the University of Lulea, ocean mining is included in a very large course on mining techniques. We also have special courses on stream hydrology and arctic environmental problems but nothing that has to do with ocean engineering. However, if a student wants to specialize in ocean or petroleum engineering, he can do so by taking courses at the University of Lulea for two years and then continue his studies at the University of Trondheim in

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Norway; then he will become a Master. At the post-graduate level there are no fixed courses because these courses are very individual, and a professor gives lectures for just two or three students. These classes are more discussions than lectures. After his master's degree, the student must continue his studies for four to five years to become a Doctor of Engineering. He must devote half of his time to original research, and during half his time he must take courses. Thank you.

T. Patten: Is there any sign of change? We are aware of quite strong moves by some of the Swedish manufacturing companies to provide equipment for the European continental shelf. Is this likely to have any impact on education?

L. Bengtsson: I imagine that there are courses now, which are called more advanced topics in engineering, where the students are taught about extracting sediments and gravel from the sea; that is a very new course. I think the trend is that there will be more and more courses on applied hydrology instead of more coastal engineering courses.

T. Patten: I was wondering whether there is a strong interest in Sweden in submarines and underwater systems development?

L. Bengtsson: I would have to check on that.

W. Sackinger: Are there a large number of students from Sweden in that field, and do they find jobs after they are trained?

L. Bengtsson: As for now, I cannot tell you, but a few years ago it was quite difficult to find jobs. Then the students were not very inclined to go into technical studies. However, the student places in mining engineering are filled every year.
CURRICULUM DEVELOPMENT FOR OFFSHORE TECHNOLOGY: POSSIBLE ROLE FOR SEA GRANT

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National Oceanic and Atmospheric Administration
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United States

ABSTRACT

The National Sea Grant Program is described, with particular emphasis on education projects. The program is ready to help prepare the trained manpower which offshore industries require. It is particularly ready to help in the development of curricula to carry out this training in better or more efficient ways. Any proposal for an education project which is submitted to the Office of Sea Grant should show (1) that there is a projected or actual shortage of trained manpower and (2) how the proposed education program is designed to alleviate this shortage.
THE MANPOWER SITUATION

Because the offshore industries use complex equipment in an extremely demanding environment, these industries depend essentially on trained manpower. Some people work on offshore platforms. Some operate seismic survey vessels. They fly helicopters to carry men and supplies to and from platforms and drilling ships. They monitor and control vessel traffic under crowded conditions. The list of technical tasks seems to be endless; and each job on the list requires a considerable amount of special training.

The men and women who fill these marine jobs do so at the end of a long educational process. The process begins with general purpose education and ends up with highly specific training. For the professional person who will later do professional work in an offshore industry, the early phases of his education are often concentrated on mathematics and the natural sciences. Gradually the person’s education becomes more and more sharply focused until it is completed by training in the use of a particular piece of equipment or in the piloting of a particular variety of ship or the like. This last phase is "training" in the strictest sense of the word.

I believe the nation's schools are the best places for people to acquire general purpose education. This is the part of education that schoolmen do best. The other end of the educational process -- specific training for this or that job -- is best done on-the-job or in training classes run by industry. I cannot imagine that the nation's schools will ever be able to do this last phase of a person's training; nor should they try.

All that American industry can legitimately demand of our high schools and colleges is that they provide men and women who have the developed mental skills and habits of study that equip them to progress into the last or training phase of their preparation for jobs. Some think there is a gap between the world of school and what we may call the work. And I have to admit that there sometimes is such a gap. This is both an unnecessary and an undesirable situation. And it is a situation that -- at least in the marine area -- the Sea Grant Program is helping to avoid. To see why, let me describe something of the program.

SEA GRANT EDUCATION AND TRAINING

The federal government provides support for education and training in three ways: (1) through direct loans and grants, (2) through research and development grants and contracts which involve the employment of students as research assistants and in other roles, and (3) through in-house educational and training activities with federal agencies.

Some of the agencies of the federal government which support education and training in the above ways are the Department of Health, Education and Welfare, the National Science Foundation, and the Department of Labor. But, since the passage of amendments to the Marine Resources and Engineering Development Act of 1966, the principal focus of responsibility for marine education and training has been in the Sea Grant Program which is part of the National Oceanic and Atmospheric Administration.

The Sea Grant Program has three main objectives: (1) research programs in various fields relating to the development of marine resources, (2) advisory services in marine resource development, and (3) education and training. Funding for education and training is directed toward producing the manpower necessary for marine resource development: marine technicians, marine scientists, ocean engineers, and other technologists. The Sea Grant Program provides funds to educational institutions to cover up to two-thirds of the cost of any project. Sea Grant is ready to involve industry as well as academic institutions in its education and training programs. Industry might provide fellowships and scholarships to institutions enjoying Sea Grant sponsorship. Or it might sponsor cooperative on-the-job educational programs between Sea Grant institutions and itself.
Some Sea Grant education and training activities are:

(1) technician training at two-year institutions of higher undergraduate and graduate education;
(2) marine curriculum development at four-year colleges and universities;
(3) senior college training programs such as the Marine Engineering Technology Program at Mississippi State University;
(4) other educational programs, often under the Marine Advisory Service.

One last note to this description of the program: As a responsible manager of the people's money, Sea Grant is only ready to support education and training programs when there is a present or predicted shortage of trained manpower. We have no interest in contributing to unemployment.

"CURRICULUM DEVELOPMENT FOR OFFSHORE TECHNOLOGY: POSSIBLE ROLE FOR SEA GRANT"

The Sea Grant Program stands ready to help prepare trained manpower for the offshore industries. It is ready to help especially in the development of curricula better designed to carry out this training. And, as I mentioned above, the Office of Sea Grant would first have to be reasonably certain that there is or will be a shortage of trained manpower.

Marine education and training programs might be carried out in any number of ways. The above list of on-going Sea Grant educational activities may suggest possibilities. Retraining programs might be started; these could be particularly useful for workers who have reached the effective limit of their technical skills and yet are capable of additional training. Sea Grant could act as a partner in starting industry-university cooperative programs involving a mixture of work and study.

The Office of Sea Grant will not take the lead in specifying the education and training needs of industry. Neither will the office take upon itself the burden of proving whether or not there is a shortage of trained manpower in offshore technology. These are jobs that leaders in the offshore industries themselves are in the best position to do. Moreover, industry and the schools working together are in the best position to design education and training programs and to arrange a mutually beneficial division of labor among themselves. Once this preliminary work is accomplished, the Office of Sea Grant will readily receive proposals for education and training projects in marine areas. Funds are limited for new education and training projects in FY 1976, to be sure. But they will always be limited. If we wait for a guarantee of available federal funds before starting, nothing will ever get done.

Let me close by repeating that any education and training proposal submitted to the Office of Sea Grant and directed toward meeting the manpower needs of the offshore industries should show the projected shortage of such manpower and how the proposed education program is designed to alleviate this shortage. If a proposal fulfills these requirements, I assure you it will receive careful consideration in my office.
DISCUSSION

Anonymous: These colleges receive a grant for teaching diving. I imagine that their curricula is quite well checked.

T. Murray: Yes, we do use outside reviewers. When a program starts, we do try to get outside reviews, the best people we can find.

Anonymous: I am in favor of divers being trained in anything at all. As for things like actual diving, even if they were trained in the school, a lot is learned through the apprenticeship, you know. They have to serve an apprenticeship even if they go through diving school, no matter how good the school is. One of the things you are not going to learn any place except in a book is diving physiology. That one thing should be emphasized more than anything else, because the only place you are going to learn that is to study it yourself and understand. The misunderstanding of diving physiology has caused more havoc in the diving field than probably anything else I can imagine. More than sharks and big fish, it is just a lack of knowledge of diving physiology. Do not depend on doctors to know it either.

T. Murray: I believe that is part of every program, at least to some extent; but it is the University of Michigan that is doing the most in this regard, and perhaps the University of Wisconsin. I am sure Michigan is doing a lot of this. Part of what they are doing is to educate doctors in the Great Lakes area so that when these cases come to the doctor, he has some idea of what he is facing.

Anonymous: Many a job cannot afford a doctor. You do not have to be a doctor or have a degree to understand it. Even I understand it. You have just got to take the trouble to learn it, and it has got to be learned out of a book; you have got to study it. A good instructor helps a lot.

T. Patten: I would not want to disagree with that. I have no qualification in diving. Did you really mean that you do not need a qualified doctor in diving operations? That is what I took from what you said. Do you or do you not need a qualified doctor?

Anonymous: You do not.

T. Patten: Do you have no limit to that statement?

Anonymous: You need a medical technician in a large operation like saturation diving. Some diving operations only have four divers on the job. If you put a doctor on the crew for four divers....

T. Patten: I agree with you that you do not need a doctor there for every operation, but I would have thought that within a call distance you would need to make sure that there are doctors, particularly in the kind of operations they are concerned with in the North Sea.

Anonymous: Could I give you a typical example? Take a man who has been around diving for years, who is very knowledgable, and who has worked as a diving consultant, who knows the work and knows the general principles of decompression and all that. He put a diver over, the diver went down to about 120 feet, and stayed there a couple of minutes; and according to all your tables, he needed no decompression. Well, he came back, and it was just a couple of minutes, he only went down to inspect something and then came back up. About a half hour later he started complaining about pains and one thing and another, but the expert figured it was not a decompression dive so there was no decompression required. Finally the diver was put in the chamber and given a half-way treatment. The diver was crippled for life. What he did not realize was that
this diver could somehow have had his breath interrupted just as the tender took a heavy pull on him, he could have ruptured his lungs and injected air directly into the bloodstream; then the bubble got into his spine and he was crippled. This is an example of people not having a knowledge of physiology.

T. Patten: My comment was only that we seem to be going through a situation in the U.K. where people are seeking large sums of money to train doctors so that there can be doctors not too far away.

Anonymous: A doctor's not going to be on the job, though?

T. Patten: No, but you still need doctors within distance.

Anonymous: Very definitely.

T. Patten: Sorry, I must have misunderstood.

W. Sackinger: Are the regulations that are applied in the U.K. and in the North Sea area for diving consistent with what we are doing in this country? Perhaps you might like to comment about the legal framework for diving.

Anonymous: Well, starting this year under the Mining Offshore Act in England, their safety regulations have come into effect and are operative. To date, the American government has not started. Everything is still in the mill, so to speak. Over in England they are in effect, and diving companies are abiding by them.

T. Patten: There are two sides, I think, over there: Those regulations which are there to safeguard the diver himself, with respect to his own physical fitness condition and registration and so on; and then the other side relates to the company's responsibility, such as having access to chambers and so on.

Anonymous: Yes, the British government has come out with a set of regulations, and the diving contractors have come out with something to protect themselves also.

T. Patten: That means that our diving schools would have to operate according to those regulations, and they probably would have to be checked out.

Anonymous: I also believe that starting some time this summer, they are to have inspectors come out on job sites.

W. Sackinger: It sounds like diving schools, then, really must be keenly aware of progress in the development of diving safety regulations in this country and the regulations that are already established overseas.

Anonymous: Yes, that is correct.

T. Patten: This year in the North Sea they are estimating that there will be approximately a thousand divers or so.

Anonymous: What is the number of fatalities.

T. Patten: A dozen or fifteen to date.

T. Patten: As a foreigner I am interested in how you checked the statements, by people who are looking for support, about the manpower situation. Do you have some feel for what the manpower needs are so that things can be checked?
T. Murray: You can look at the quality of the arguments the person puts forth, what slight evidence he may have; and then you check with other, hopefully knowledgable, people, using the peer review system. Reasonableness and the peer review system are really all you have to rely upon.
EDUCATIONAL PATTERNS IN PORT AND OCEAN ENGINEERING IN SCANDINAVIAN COUNTRIES

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Trondheim
Norway

EXTENDED ABSTRACT

Education in Port and Ocean Engineering in the Scandinavian Countries usually starts at the age of 20 to 21 when the student, at the age of 18 to 19 has passed two exams, the first one corresponding to final exams in the American Junior College and the second when he, after having been permitted to the Technical University, has studied basic disciplines: mathematics, physics, mechanics etc., for at least two years. Without these two exams and proper grades he will not be permitted to continue this education.

Educational modes differ somewhat in the Scandinavian countries. In Sweden the education is a joint program including Hydraulics and Hydrology, which is of great importance to that country, and Port Engineering. In Denmark and Norway the education is divided into entirely separate courses in Hydraulic Engineering and in Port Engineering. The education includes four semesters. In Denmark emphasis is put on the part of Port Engineering which is of particular interest to Denmark. This does not only include physical conditions in Denmark but aspects of overall international interest due to the very considerable Danish consultant engineering activities in foreign countries. Classical wave mechanics, littoral drift phenomena and all kinds of harbor structures and their location and operation are the main items. Due to the activities in Greenland, ice problems have recently drawn attention to the educational program. This education takes place during four semesters.

The Norwegian education is a combination of Port and Ocean Engineering (Table I), the latter being introduced when the exploration for oil and gas started in the Norwegian part of the North Sea in the late sixties.

The educational program is now run for five semesters of one half year, starting with a basic course in Hydro and Wave Mechanics. Due to the rapid expansion of activities in the North Sea and adjacent waters and in order to better serve the large Norwegian merchant marine, it is practical to divide the following part of the education in two lines which overlap in certain aspects of wave technology including wave statistics and forces by waves and currents on structures which are of equal importance to Port and Ocean Engineering. Certain special transportation aspects also overlap.

A very important part of the education in Port Engineering is what is termed "Harbor Transport Technology". This deals with all phases of harbor transport technology and analyses of operations of equal importance to port and shipping technology. This also includes wave analyses for operation at offshore installations.
The education in Ocean Engineering includes advanced wave hydrodynamics and statistical wave analyses which refers to forces on structures like platforms, tanks and pipelines as well as to offshore operations. With respect to the latter it may be noted that Ocean Engineering and Offshore Transport Engineering overlap. This is very useful for both.

It has been found practical to arrange combined lectures in Ocean Engineering for students majoring in naval architecture and in the civil engineering part of Ocean Engineering to avoid an illogical split-up in "floating" and "fixed" structures. The education also includes forces on the sea bottom, forces on structures by ice and environmental problems. Cooperation with soil mechanics is established.

In the fifth semester of this educational program the student carries out an independent thesis on a pertinent problem. Following his final exam he may - if his grades permit - continue a two year advanced education program leading to a degree which is quite similar to the American Ph. degree but carrying less weight on courses and emphasizing more on a particular research project.

To avoid tendencies to domestication it is customary to encourage the students to go to a foreign country e.g., the U.S., Canada or the U.K. to study for some weeks or for a few months. This may be done during summer vacations. The result has been that most thesis are now written in English and outside impulses receive proper attention.

ABSTRACT ONLY AVAILABLE
TABLE 1. Education in port and ocean engineering in Norway

<table>
<thead>
<tr>
<th>Years</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>General basic education in mathematical-physics &quot;gymnasium&quot;-line. Age 16 to 19</td>
</tr>
<tr>
<td>3-5</td>
<td>Basic education at the technical university, mathematics, physics, mechanics, statics, age 19 to 21</td>
</tr>
<tr>
<td>0.5</td>
<td>Hydromechanics, wave and current mechanics, forces on structure, tidal hydraulics, sed. transport. Age 21 to 21.5</td>
</tr>
<tr>
<td>5-5.5</td>
<td>Harbor Engineering, Harbor Transport, Harbor hydraulics and navigation, Cargo handling systems, Sediment transport and dredging, (Coordination with courses in soil mechanics and general transportation) Age 21.5 to 22.5*</td>
</tr>
<tr>
<td>5.5-6</td>
<td>Marine Technology, Wave and current mechanics and statistics, Forces on and stability of fixed and floating structures including mooring, Short and long term operation analyses, Bottom stability, Forces by ice, Model laws, (Coordination with Naval Architecture) Age 21.5 to 22.5*</td>
</tr>
<tr>
<td>6.5-7</td>
<td>Harbor Engineering, Special Course, Advances topics of harbor engineering and transport emphasizing in operational and transportation aspects, (Coordination with soil mechanics and transportation) Age 22.5 to 23 (23.5)</td>
</tr>
<tr>
<td>7-7.5</td>
<td>Marine Technology, Special Course, Advances topics of wave and current mechanics and statistics, calculation of forces on fixed and floating structures, operation analyses, Underwater technology, (Coordination with Naval Architecture) Age 22.5 to 23 (23.5)</td>
</tr>
<tr>
<td>23</td>
<td>Thesis**</td>
</tr>
</tbody>
</table>

Note: *Students are encouraged to seek employment in industry during summer months
**Most thesis are written in English

A Dr. of Engineering degree may be obtained about two years later if the student is considered qualified. It includes courses and thesis.
The thalatect, or marine designer, has emerged in response to the increased demands on the oceans for energy, accommodation, mineral resources, food, transportation and recreation. His function as coordinator of marine personnel, technologies, and concepts distinguishes his profession from the other marine disciplines. His worth is particularized by his training, expertise and natural personal attributes. In this paper his field, and therefore his purpose, will be defined for the first time.

Design is a finite context, as are the tolerances and resources of the seas finite. Yet we have expanded into the seas as if they were not finite. It is the function of the thalatect to be versed in this context and to be able to interpret and direct man's expansion into the seas mindful of the disciplines the seas impose.

Thalatecture is any systematic design created in response to the disciplines of the seas, rapide, shallows and littoral regions and mindful of the context and scales in, on, or associated with the seas (with design meaning to plan the form and construction).

Thalatecture is then, a multi-purpose, spatially omnidirectional concept. Since the seas are themselves dynamic transportational mediums, thalatecture provides that point of union between static structure and transport systems of each element; land, air, and water.
INTRODUCTION

The thalatect, or marine designer must emerge in response to the increased demands on the oceans for energy, accommodation, mineral resources, food, transport and recreation. His function as coordinator of marine personnel, technologies and concepts distinguishes his responsibilities from the other marine sciences.

His worth is particularized by his training, expertise and personal attributes. In this paper his profession, and therefore his purpose, will be defined for the first time. Just as architecture existed long before a person was called an architect, so too have we been mining, exploring, and building in the seas and littoral regions for quite some time: it has been in a piecemeal manner, without determining either the broad limits of the sea's capacities and disciplines, or the long range demands and effects of man. Design, however, implies a systematic approach; a marshalling of resources, talents, materials and economies. Design is a finite context just as the resources and the tolerances of the seas are finite. It is the function of the thalatect to be aware of the finite context he deals with in order to design for man's advancement into the seas. In defining the disciplines of the seas, one must first distinguish between which are the disciplines of the seas and which are the disciplines of man's laws, technologies and economies. The first are the independent variables and the second the dependent variables. Man must adapt to the disciplines of the seas for the seas will not adapt to man.

DEFINITION

Thalatexture is any systematic design created in response to the disciplines of the seas, rapids, shallows, and littoral regions and mindful of the context and scales in, on, or associated with the seas (with design here meaning to plan the form and construction).

This definition is derived from two Greek words: 
thalassic, meaning of, or pertaining to the seas, growing in, found in or living in the seas, that is marine; and tectonics, the science or art of assembling, shaping or ornamenting materials in construction.

Thalatexture is then a multi-purpose, spatially omnidirectional concept. Thalatexture begins as a complexity, since the seas themselves are a dynamic transportation medium. Thalatexture provides that point of union between static structure and the transport systems of each element; land, water, and air.

Therefore, the thalatect is a systematic designer in, on, or associated with the seas, creating in response to the disciplines of the seas. The thalatect fills a design and planning role with reference to the seas.

The use of the term "ocean engineering" over the last ten years has come about due to the perceived need to link and encompass many fields that are being drawn into closer contact due to the increased advancement into the seas. This coordination needs direction and a director -- the Thalatect.

ROLE

The role of the thalatect may, for analysis, be divided into two parts at this time. The first is the thalatect as a planner. Thalatects are needed immediately for all phases of marine planning from the planning of international development to the coordination of development for local coastal zones with associated offshore activity. To assemble the wealth of data and research accomplished up to the present on marine environmental problems and their relation to resource demands, and to determine where gaps exist in this information (which is needed to formulate rational plans for advancement into, and development of the seas), thalatects are needed. In the process of weighing the legal, economic technical, social and environmental variables, the role of the coordinator.
for successful synthesis is paramount. (One area for needed work being the quantification of "external variables" such as social and environmental costs and benefits.)

On land the most successful regions and cities have been those designed for future growth and modification; those having been able to adapt to modern technologies. Until now the oceans have been subject to individual uses by man with the different support facilities meeting onshore in increasingly crowded coastal zones. This crowding is what in many cases, is propelling man offshore in search of new sites and it is essential that these offshore areas be designed with foresight to avoid similar problems that exist on land. We can no longer develop with only our own generation in mind since the rigors of marine development entail great expense and will impact long into the future.

The second job of thalatects is to supervise the execution of final design at all levels, mindful of the reasons and interrelationships of not only the technologists, but also the goals of the long range planning.

COORDINATION

As the oceans develop, an economic philosophy must develop with them; that of long term use and investment. Since substantial initial investment is involved, and hence large commitment, it is in everyone's interest that the offshore areas be designed effectively. The present scramble for additional energy should not obscure the long range effects of extremely expensive stopgap investment. In addition, the legal framework and control of the oceans must be examined closely by marine policy lawyers in conjunction with thalatects knowledgeable in sensible long term management of the seas and able to perceive the direction of future development.

The thalatect functions then, as the coordinator of the technical and theoretical advances in marine design.

QUALIFICATIONS

To provide qualified people for these positions, individuals must be located and trained to address the new concerns. The qualifications of the thalatect are two: first, the possession of personal attributes fostering the ability for complex decision-making; and second, the necessary education and training.

His personal competence is derived from the following attributes: the possession of the ability to perceive, interpret and harmonize the disciplines of man and the seas. This should not be done in a static manner, but with an eye toward whether the seas imply answers to planning problems, and whether a present "problem" may change, grow, multiply or even disappear with time.

Second, the thalatect possesses the coordinating ability and training to dovetail people, technologies, and concepts. Further, these coordination powers are supplemented by maximization abilities to avoid strain on the people, technologies and concepts. Since planning and execution of ocean development demands the talents of many professions, the thalatect must coordinate the efforts of these specialists and incorporate their efforts into logical, long term plans and construction.

Thirdly, the thalatect possesses the ability to withdraw from the particulars of his work long enough to reaffirm the direction of the work with the long term goals. He also possesses the ability to apply new ideas to established practice.

We often train individuals for a specialty without determining if those individuals are capable of anticipating and projecting their work into the future. When we speak of
the oceans, we are almost always dealing with the future; whether it be the tolerance for a yet deeper dive, an even larger floating station or the multiple uses of a present single-use offshore area. It is fundamental then that the thalatect be vitally aware that his actions dictate the life of tomorrow and not only today.

TRAINING

The training of the thalatect must be based on the common fundamentals of design and especially marine design. These fundamentals include: a study in generic form, especially marine and littoral forms as the seas' disciplines of hydrodynamics call for a stricter adherence to sympathetic form than do the conditions on land. In conjunction with this, the study of volumes, positive and negative space in man, the seas, and other living organisms is required as well as other physiological studies in life support systems, both natural and man-made. States of homeostasis with reference to biofeedback, biorhythm, neodic cycles and body tolerances, the tolerances of systems both organic and inorganic, especially with respect to changes over time, need examination for man's future in the sea. Education in the basic laws of biology, geology, physiology, natural form, mechanics, and the properties of organisms and materials should be the basis for design education whether it be for land, air, or the seas.

The thalatect, having cultivated the natural "feel" for the marine environment, and being aware of the long range goals of man with his relation to the oceans, can then specialize in thalatectonics, systems analysis, or port design in arctic conditions, better equipped to perform his job. He can then coordinate the work with his associates in the context of how design, systems, and organization naturally must comply with the properties and disciplines of the marine environment.

With this broad theoretical knowledge, as well as personal expertise in particular aspects of marine design and construction, the thalatect would be in the position to be aware of numerous developments in other fields and could look into new approaches to marine design.

For example, with the construction of superports we can learn a lesson from the past; namely, that often harbors were planned and constructed without sufficient thought of future needs or technologies. It could be argued that older ports have been in use for centuries and therefore have served their purpose well, but it must be remembered that today technology advances at a much faster rate and design evolutions that took over one hundred years can now be leap-frogged in five or ten years. In such light, it seems folly to design a superport for only large surface transport means. It should also be able to adapt to future transport means, such as dracones or airships.

EDUCATION

At present, our universities could be said to be training persons in the field of thalatectonics, that is, the science of constructing in response to the hydrodynamic demands of the seas. Surely, this training is fundamental to answering the needs of the work there is to be done. Programs exist throughout the world to train people for such practice, yet nowhere is there a training ground for thalatects. A few universities do offer some guidance in the field of marine design, but the training of the person in comprehensive planning and design of the seas does not yet exist and it is needed.

We must no longer merely train people to react to the seas, that is, only to design to overcome the seas. We are now designing for a population of increasing size that is daily involved with the seas and therefore subsistence design is no longer adequate.

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Godfrey 4
This calls for a revision of the educational system as it relates to ocean engineering. A program for the education of thalatects is needed. This instruction would be a departure from the traditional courses in oceanography, marine biology, fluid mechanics, etc. Courses on those subjects would have to be reevaluated and then taught in the light of what they have to offer the thalactect, to assist him in his decision-making. Thalatects must know and understand the principles of all the marine related sciences in order to be able to develop logical plans and to coordinate specific projects as well as be able to determine from whom to seek expert advice on marine decisions with which they are confronted.

This approach raises many new questions about educational and professional intercommunications as well as questions of how to improve the methods of communication and information dissemination itself. This is part of another phase in the development of thalatecture, namely the development of the concept after its identification. This is being followed up and included in a volume entitled: Thalatecture: Strategic Design for the Marine Environment. The first step, though, in any development is the stating of the task.

One fact is clear, however, as the United Nations' Geneva and Caracas Conventions have proven; and which the nonexistence of comprehension concerning long-range development plans underline -- We need people trained in marine decision making -- Thalatects.
DISCUSSION

Anonymous: I think it is an interesting idea he put forth, and it is one that puts a little bit different focus on education. I think there are trends towards this now in that we recognize the need to have people in any given science familiar with what adjacent sciences are doing. I take the architecture or planning analogy and I carry it a little further, and I find there has been a tendency to overgeneralize in the training of those people, though, often to the detriment of their effectiveness. Architects, I think, have in many cases gone to the point of being artists with building but not always understanding the principles that make the building stand. In other words, for example, taking things which originally were engineering necessities and putting them in as adornments not related to their original purpose, or not understanding their purpose. I think the sea is even a more difficult environment in which to operate, where non-essentials are put on for reasons that are not related to their being there; and I think this would be a particularly difficult thing to overcome. If I carry this to the planners, I find that if I watch planners directly, I find them to be a particularly ineffective group, as a group, in that they are usually staff to governments or to corporate decision makers; and they do not seem to have the impact on the overall scheme of things on land that you would think they should. Now, if we could analyze some of the deficiencies that I think exist in architecture and the deficiencies which exist in planning, and come up with a curriculum that would bring your thalatect into being in a sensible way, it might be a very good move.

W. Godfrey: That is a very good point. Part of the reason that I have been looking at the design and planning of 70% of the world's territory which has yet not been really designed and planned to any great extent, or subject to legislation, is because of the way it has been done ineffectively on land. Now I would like to promote discussion on the topics of education that are needed so that people have the right criteria. Two extremes are given by those who say, "I want to make the sea beautiful", and those who say, "Look, let me build what I want". An education should teach just how far man can push the sea, whether it be in the extraction of fish population or whatever, and then to start design in those realistic contexts.

Anonymous: The problem that we are facing in education is just developing curriculum. One does not reject these things that are needed, but at the same time there are only a limited number of credit hours available. We feel that as a tremendous constraint. More basics are needed; be sure that they have the very fundamentals and that they know basic physics. In order to do that, it uses quite a bit of the curriculum and the time available. Students have not been trained in the lower years in the basics, and in order to build them up from that level, there is a tremendous conflict between time and need.

W. Godfrey: I think what they are doing at Heriot-Watt University is in the right direction. They specialize in subjects at which they excel, in which they have people and facilities. Furthermore, a problem with "liberal education" is that the university administration insists on students taking five courses in English and three in something else, which really do not apply to your program at all. Someone should coordinate those theoretically disparate activities to have some bearing or some applicability to the career direction in which a person is interested, and in which he is going to practice.

Anonymous: Most of the students that we get in our course come from a hundred different disciplines, you might say; and they certainly have to have some basic training in the fundamental courses. We have made it a policy at our university not to offer any financial assistantship the first semester or perhaps even the first year, making them take the courses that they need. If we do give them money by means
of an assistantship, then they are going to avoid taking the courses. They do not do well in either course work or in their research.

W. Godfrey: Right. That is one thing I mentioned.

W. Sackinger: It appears to me that there is an analogy between architecture as it has evolved over the past several decades and its focus on structures on land, its planning for structures and even cities on land, and the engineering side that actually is responsible for making these buildings. If we move to consider the sea, we have the very exact educational disciplines we have talked about in engineering, such as the marine engineer and the naval architect. Now I am just curious as to what kind of discipline or person has played the role of the architect in the marine environment for the last several decades or has it been just a vacant role?

W. Godfrey: I think it has been a vacant role. That is the conclusion I came to initially.

W. Sackinger: Someone has been responsible for the system engineering of some of the things associated with the sea, like the construction of a large platform or a large terminal. Perhaps what you are implying is that there are even larger systems associated with the sea that are in the future, and we need more of a system plan approach for them?

Godfrey: Yes. I mentioned the two possible roles of a thalatect, the first being as a regional planner and developer. I think we are a little bit out of phase with our steps because technologically we do not have the time to delay projects; needs have to be met. At the same time we must start training people to look at the problem in this light. We should ask the questions: What is out there; what could be done with it; and then how to design best for it over the long term. That is the kind of people I do not think we have.

P. Bruun: At the University we are certainly limited by time in how much we can do. One thing we shall teach them is the basic aspects of the problems, because they are not going to learn this in a marine school. It is very important to get them trained as well as possible in the basic fields. Now, it is important that these young fellows know what the whole thing is all about, as well. We give them regular seminars where we try to tell them about the practical aspects; tell them how this is being used. Then we call in people from industry and let them conduct seminars. Once a week or once every second week we have a man from industry who tells about pipe-laying barges, about how to build concrete platforms, how to float them out, a number of things. Students like very much to listen to that.

T. Murray: That is a very large task for the thalatect. Have you considered the team approach towards this sort of thing, or do you despair of that being a workable solution?

W. Godfrey: As I was saying, a thalatect's role is principally as a coordinator of expertise. You could not have one person who knows everything about structures, everything about marine populations. That is an impossibility.

T. Patten: I have a very personal opinion. The thing that I find very difficult is that with architects there is this strong tendency to relate objects to position. I do not think we have found the way, with one or two notable exceptions, to educate architects, so if you introduce an order of magnitude in another dimension as the sea does, I am a bit lost. Having said that, it seems to me there are many problems that have to be solved in the environment where the other man might not be a planner but rather an economist or an environmentalist. You are almost going again for another
kind of rigid approach when it seems as though your argument was based on that idea of the difficulty or the danger of education is that it is already rigid.

W. Godfrey: I do not see how someone whose education and training is premised on the fact that he has to coordinate and learn from others would be a rigid thinker or designer.

T. Patten: It is so because of the education part; and, of course, obviously this man must have practical experience. He cannot be produced just by education; he must be trained and developed. But in order to get all the things that it seems are needed, his education will be rigid. There are only so many hours in the day.

W. Godfrey: Rigid in physical sciences, in hard scientific principles of physics and mathematics, yes. Rigid in the laws of science, surely. That is the biggest, strongest base that he works from. But then his job is not to specialize in one of those particular disciplines or the application of that discipline; but to coordinate the work of those people, with those who are legislating the uses of the seas. At present, the technological people have no control over this.

T. Patten: It seems to me then, you are differentiating between the chap who is a very talented creator, a design-oriented individual is identified only with great difficulty in educational fields; and the other kind of chap who is a very good manager, gets on very well with people, and can put together a very complicated picture. You do not then have to talk about whether he knows anything about littoral forms or marine forms. He can somehow piece these very difficult problems together.

W. Godfrey: Right. And there is no reason why the person could not be talented in both respects. Every once in a while you meet someone like that, too.
SECTION 13

JAPANESE MEMBERS OF THE 6TH UNITED STATES - JAPAN PANEL
ON MARINE FACILITIES

PAPERS PRESENTED AT POAC-75

Ando, Noritaka
Some Aspects of Collision Problems
(Ship Research Institute, Japan)

Hayashi, Satoshi, Osami Horii and Shigeru Ueda
Investigation of Crude Oil Tanker Berths in Japan
(Port and Harbor Research Institute, Japan)

Ito, Tatsuo, Isao Ueno and Sadao Ando
Semi-Submersible Work Vessels for Offshore Operations in Rough Seas
(Ship Research Institute, Japan)

Kawakami, Yoshihisa, Yoshimi Goda and Tsutomu Kihara
Some Recent Studies on Floating Marine Facilities at Port and Harbor
Research Institute
(Port and Harbor Research Institute, Japan)

Kunitomi, Akira
Pipe Lay Barge with Derrick KOKAN PIONEER I
(Shipbuilding Technical Department, Nippon Kokan K. K., Japan)

Miki, Sansei, Nobuo Ohkubo and Shinichiro Sano
Jacket and Pile Supported Offshore Airport
(Kawasaki Heavy Industries, Ltd., Kobe, Japan)

Mutoh, I.
Modoc Oil Skimmer (MIPOS)
(Mitsui Ocean Development and Engineering Company, Ltd., Japan)

Mutoh, I.
Underwater Observation Vehicle (Eye Robot)
(Mitsui Ocean Development and Engineering Company, Ltd., Japan)

Noguchi, Misao and Minoru Ishida
Research on Integrated Offshore Treatment System of Town Wastes and
Effective Utilization of its Byproduct
(Ishikawajima Harima Heavy Industries Company Ltd., Japan)
Samethima, Taisuke
Governmental Administrations on Preservation of Ocean Environment in Japan
(Bureau of Ports and Harbors, Japan)

Takarabe, John T.
Application of Dielectric Radar Reflector (Lensref)
(Tokyo Keiki Company, Ltd., Japan)

Tamehiro, Masayuki
Construction, Towage and Installation of the Floating City "Aquapolis"
(Mitsubishi Heavy Industries, Ltd., Japan)

Ueno, Isao
On the Mechanical Properties and Results of Soak Test in Sea Water for Anti-Corrosive Wire Ropes
(Ship Research Institute, Japan)
INTRODUCTION

There is strong public concern about environmental damage resulting from oil spills in collisions. However, any form of control and regulation of marine traffic patterns on coast or inland seas will not be adequate to prevent future collisions. One of the most important factors in connection with collision is the "crash-stop" ability of such structures as offshore oil storage tanks.

Most research on collision is mainly concerned with protection of reactor against collisions in nuclear-powered ships. The protection structure is adopted for all nuclear ships and LNG carriers, in which there is used a double-hull construction with a considerable distance between outer hull and inner hull. The distance is determined mainly by Minorsky's method which relates the absorbed energy to the so-called "resistance-factor" of structural elements from analysis of data on actual ship accidents. Minorsky made the assumption that the members having little distance in the direction of penetration such as the shell plate of a struck ship, were regarded as small and unconsidered in making an estimate regarding the ability of a structure to absorb energy. The protection structures designed by using his method tend to become deep, double hull ones. Though this method is incomplete, it may be practically applicable to the nuclear ships.

It is important, however, to conduct research on the possibility of designing economical offshore oil tanks or ships to safely withstand the impacts. In the research there must be given a rational breakdown of absorbed energy into structural elements so that a designer can easily determine how colliding energy is being absorbed. Some model testing has been undertaken recently to determine the structural behavior of a double-hull construction in collisions. The tests are statically carried out with the double-hull models simulating side structures of offshore oil tanks.

The test results suggest that there will be two design concepts. The first is, there has to be introduced some means of reducing the speed of a striking vessel around the offshore oil tanks and that her remaining energy is forced to be absorbed by the action of membrane force developed in the outer hull. The second is, that there is adopted a double-hull structure with a considerable distance between inner and outer hull and that the entire energy of a striking vessel is forced to be absorbed after the outer hull plate is broken. At any rate, there can be an optimum "collision protection" system. Research concerned with this field is now vigorously in progress.
Figures 1 and 2 show one example of Japanese future planning of an offshore oil tank system. Twelve tanks lie at anchor and each one is connected by a pipeline. The structure is similar to the middle body of a huge tanker, except that double hull structure is adopted as shown in Figure 2.

Figure 3 shows the state of a middle section of tank collided by a tanker. Horizontal girders are placed at the upper part of tank where a tanker is supposed to collide.

Figure 4 represents Minorsky's method. It shows a linear relation between absorbed energy in collision and "resistance factor". There is a zone where this relation is not definite.

Figure 5 shows the test setup adopted by us.

Figures 6 through 9 show some examples of test results. These represent the relation between load (Figs. 6 and 8) or absorbed energy (Figs. 7 and 8) and penetration of bow. We can see how the curves are varied with the thickness of the outer hull plate (Fig. 6), and with thickness of girder plates (Fig. 8). The first hump is caused by the action of membrane tension in the outer hull plate. A large portion of energy is found to be absorbed in the outer hull plate (Fig. 7), while considerable energy seems to be absorbed after the outer hull plate is broken (Fig. 9).
Figure 1. Future planning of offshore oil tank system
Figure 2. Future planning of 1,000,000 kl offshore oil tank
A striking ship

Full load

Ballast

Section of tank

Figure 3. Condition of collision

Figure 4. Minorsky’s Method
Figure 5. Test setup
Figure 6. Load curves (1)

- \( t_e \): thickness of outer plate
- \( t_i \): inner
- \( t_h \): horizontal
- \( t_v \): vertical
Figure 7. Energy curves (1)
Figure 8. Load curves (II)
Figure 9. Energy curves (II)
INVESTIGATION OF CRUDE OIL TANKER BERTHS IN JAPAN

Satoshi Hayashi, Osami Horii and Shigeru Ueda

Structures Division
Port and Harbor Research Institute
Ministry of Transport
Japan

INTRODUCTION

The investigation of oil tanker berths in Japan was carried out in 1974. The purpose of this project was to survey structural design, operational conditions, and means for safety of deep sea terminals, as well as maneuvering, berthing, and mooring of big tankers in ports. The berths which accommodate crude oil tankers greater than 100,000 D.W.T. became the objects of this investigation. The questionnaire was handed over to the berth owners and collected after a month. In addition, berths for tankers greater than 200,000 D.W.T. were investigated in detail at the site by the survey staff.

On the basis of this survey, the actual status of structural design, operational conditions, safety measures, and maintenance of deep sea terminals is reported.

For further details, the reader should refer to Technical Note No. 201 of Port and Harbor Research Institute.

SCALE AND STRUCTURAL TYPE

Scale

Figure 1 shows the number of deep sea terminals constructed in the three periods as a function of tanker size. In the first period, 1953 to 1962, berths for tankers smaller than 100,000 D.W.T. were mainly constructed. In the second period, 1963 to 1967, the number of berths constructed for tankers of 100,000 to 150,000 D.W.T. class was increased. In the third period, 1968 to 1972, it was noticed that a large number of berths for 150,000 to 200,000 D.W.T. tankers were built. However, it should be noted that the construction of berths for tankers below 100,000 D.W.T. was still continued in this five-year period.

This tendency may have come from the fact that new berths were constructed due to the end of service time of the old ones and due to the dispersion of oil refineries. However, when the joint operation of a large scale C.T.S. is established in the future, crude oil produced in the Middle-East and elsewhere, will be transported to C.T.S. by a supertanker of the 500,000 D.W.T. class and will be distributed to oil refineries by pipelines or small tankers of the 50,000 to 100,000 D.W.T. class.

Consequently, future construction will require the berths for tankers of the 100,000 D.W.T. class.
Figure 1. Numbers of berths constructed for crude oil tankers larger than 100,000 D.W.T. before June, 1974
Depth

The relations between tanker size and depth of berth, and between tanker size and depth of fairway are shown in Figures 2 and 3, respectively. The curve (1) in the figures is the following regression relation between tanker size D.W.T. and full draft $d'f$, which Katayama, et al. derived from other statistical data. (see Technical Note No. 101 of Port and Harbor Research Institute)

$$\log d'f = -0.180 + 0.266 \log \text{D.W.T.}$$  \hspace{1cm} (1)

Other curves (2) and (3) give the draft increased by 10 and 20 percent, respectively, from the draft calculated by the above-mentioned equation. It is seen that equation (1) generally gives a value of draft smaller than that obtained from the present survey. For tankers less than 200,000 D.W.T., the curve (2) almost coincides with results of the present survey, but for tankers over 300,000 D.W.T., it is in the lower range than the present data.

It may be concluded that the water depth of at least 1.2 $d'f$ is required for tankers below 200,000 D.W.T. and that the water depth more than 1.2 $d'f$ is required for tankers over 300,000 D.W.T., in consideration of a keel clearance.

Structural type

The dolphin type berths sum up to 18 and the buoy type berths to 6 regarding berths for tankers over 200,000 D.W.T. Regarding berths for tankers of the 100,000 D.W.T. class, the former berths sum up to 14 and the latter to 11.

A great number of dolphins are pile structures using steel pipe piles or H-shaped steel piles. In particular, the dolphins for the tankers over 200,000 D.W.T. were constructed with steel pipe piles except for the following two cases; one was of jacket type and the other was of a steel cellular type.

As to buoy berths, three types were used; i.e., IMODCO buoy, shell type single buoy mooring (S.B.M.) and multi-buoys mooring (M.B.M.). The S.B.M. was mainly installed for berths of the 200,000 D.W.T. class and the IMODCO for berths of the 100,000 D.W.T. class, respectively.

From 1968 to 1970, a buoy type berth was frequently employed for the berths of the 200,000 D.W.T. class. The buoy type has the advantages of low construction cost and a short period of execution. On the other hand, it has the disadvantages of high maintenance cost and the necessity of wide water area for mooring. Most of the berths constructed after 1971 were of the dolphin type except for one case. This trend is due to the advantages of high workability, high handling efficiency and relatively narrow water area for mooring as well as the recent progress of construction techniques. Jacket type dolphins were also employed recently for an offshore berth in the outer deep sea with severe natural conditions. The dolphins of this type were prefabricated on land to be constructed at sea in a considerably short period.

However, the dolphin type may be hopeful, but not perfect as an offshore berth for a supertanker of the 500,000 D.W.T. class because of the large force imposed on the dolphins and mooring lines by the wave-induced motion of a moored tanker in the rough outer sea.

In order to cope with this problem, it may be necessary to shelter a deep sea terminal with some breakwaters from rough waves or to make some improvement on the conventional mooring methods.
Figure 2. Relation between water depth of berth and D.W.T. of crude oil tanker

(1) \[ \log d_f = -0.189 + 0.266 \log \text{DWT} \]

(2) increased by 10% from \( d_f \) of Eq. (1)

(3) increased by 20% from \( d_f \) of Eq. (1)

- Full draft of tanker
- Water depth of dolphin berth
- Water depth of buoy berth
Figure 3. Relation between water depth of fairway and D.W.T. of crude oil tanker
OPERATIONAL CONDITIONS

Introduction

The operational conditions of a deep sea terminal are influenced with natural conditions such as meteorological (especially wind) and marine (especially waves and currents) conditions. Therefore, the operational conditions are defined as critical wind speed, wave height and current speed below which a deep sea terminal can be operated.

They are obtained in each of the following four cases:
1) conditions for pilot's boarding a tanker,
2) conditions for tanker's berthing and mooring,
3) conditions for unloading operation, and
4) conditions for tanker's undocking.

The distribution of critical workable conditions is shown in Figures 4, 5 and 6.

Conditions for Pilot's Boarding a Tanker

Wind speed There is little difference in critical wind speed between berths of the 100,000 D.W.T. class and those of the 200,000 D.W.T. class.

As a whole, 23 berths (59%) permit pilot's boarding at the wind speed below 15 m/sec and 8 berths below 12 m/sec.

Wave height Twenty-five berths (75.8%) permit pilot's boarding at the wave height below 1.5 m and others permit it either below 1.0 m or below 2.0 m each counting 4 berths (12.1%).

Current speed It is difficult to judge the standard condition because of the limited number of answers. But, as a result, the current speed below 1.0 kt is set at 10 berths (45.5%) as the critical workable value.

Conditions for berthing and mooring

Wind speed The wind speed below 15 m/sec is adopted at 8 berths (53.3%) of buoy berths, 18 berths (62.1%) of dolphin berths and 26 berths (59.1%) of all. The second majority is the wind speed below 12 m/sec which is adopted at 9 berths (20.5%) of all.

Wave height The height below 1.5 m is set at 8 berths (61.5%) of buoy berths, 15 berths (62.5%) of dolphin berths and 23 berths (27%) of all, while the wave height below 1.0 m is set at 10 berths.

Condition for unloading operation

Wind speed The workable wind speed below 15 m/sec is set at 27 berths (71.1%).

Wave height The workable wave height below 1.5 m is adopted at the majority of berths.

This value is adopted at 8 berths (53.3%) of 15 buoy berths, 17 berths (77.3%) of 22 dolphin berths, and 25 berths (67.6%) of all.

Furthermore, the workable wave height at dolphin berths is generally slightly higher than that at buoy berths.
Figure 4. Distribution of critical workable conditions (200,000 D.W.T. class)
Figure 5. Distribution of critical workable conditions (1000,000 D.W.T. class)
Figure 6. Distribution of critical workable conditions (total)
Conditions for undocking

Wind speed  Six (42.9%) of buoy berths 11 (55%) of dolphin berths and 17 berths (51.5%) of all permit the wind speed below 20 m/sec. The berths below 15 m/sec are a little over 20%.

Wave height  Fourteen of 20 berths permit the wave height below 2.0 m.

Summary of operational conditions

Besides the above mentioned statistical method, four case studies were carried out; two buoy berths for the 100,000 D.W.T. tanker on the side of the Japan Sea and two dolphin berths for the 200,000 D.W.T. tanker on the side of the Pacific Ocean. Wind and wave records at these four deep sea terminals for the period of their operations were investigated.

Combining the above-mentioned statistical results with these four case studies, operational conditions are obtained as shown in Table 1.

DESIGN AND CONSTRUCTION

Design condition

We investigated the design conditions, which include a maximum tanker size, design water depth, wave height, wave period, design stillwater level, tidal current, wind speed, wind pressure, soil conditions, seismic coefficient, berthing velocity, effective berthing energy, berthing impact, mooring force, crown height and allowable stresses. At a few deep sea terminals wave forces were considered as the design condition. The steel piles are generally driven long, because of such soft clay layer as considerably thick silty layer. The berthing energy is calculated based on the Design Standards of Ports and Harbor Structures in Japan, precluding only one berth.

Fender

The main fenders used are of a constant reaction type such as H type, cellular type, V type and super-arch type. Whole fenders have loading plates so as to distribute berthing impact uniformly to the side plate of a vessel. The impact to the side plate of a vessel is less than 30 ton/m².

Mooring equipment

The capacity of mooring equipment is decided on the basis of the breaking strength of mooring rope equipped with tankers and the number of ropes bounded at bitts or hooks. The standard diameter and breaking strength of mooring ropes are 70 m/m and 80 tons, respectively, for the 200,000 D.W.T. tanker. Some tankers, however, install the mooring ropes with a diameter of more than 70 m/m. Mooring equipment is generally designed as breaking strength of 100 ton per one rope. A quick release hook is equipped with a few berths. It seems that a quick release hook will be installed for berths constructed in the future.

Construction method

There are so many problems to be solved in construction works including driving of longer-sized piles and fabrications of structural members under severe weather and marine conditions. Hence, the structural type must be chosen in full considerations of constructional methods.

The pile driving barge is generally used to drive piles. In recent years, the floating crane loading a large-sized pile driver and a SEP has been occasionally employed. In
jacket-type dolphins, a steel framed jacket fabricated on land is transported by sea and fixed at the planned position with piles driven along the guide ring of a jacket.

SAFETY MEASURES AND MAINTENANCE

Equipment for prevention of fire and accidents

The equipment for prevention of fire and accidents are; firefighting arrangements, oil spill treatment equipment, marks, horns, gas checkers, large-type speakers, telephones, alarm signals, and a docking sonar.

The installed fire-fighting arrangements usually include foam monitor nozzles, water hydrants, foam hydrants, water curtain-wall nozzles, fire boats, and fire engines. Oil skimmers, chemical powder and absorbent are generally arranged to treat oil spills.

Most deep sea terminals are well equipped with various types of fire-fighting provisions. It is expected that a fire can break out, not only at a deep sea terminal but also at an oil tanker. It is also impossible to reduce the force of a fire after it has started and furthermore, there is danger of an explosion. Therefore, fire-fighting at the initial stage is quite important to prevent a serious accident. Then, it is necessary for a deep sea terminal to arrange complete fire-fighting provisions including automatic ones. Some buoy berths are constructed at the inner part of the bay which is generally congested with various types of ships. In this case, the safety facilities must be fully arranged to prevent a secondary accident.

The oil fence is also used for preventing oil from diffusing on the sea. The revised Law on Protection of Marine Contamination requires more oil fences than those presently arranged. The types of oil fence are as follows: an oil fence permanently arranged at some area on the sea (defining the permanent type in this paper); an oil fence which is buoyed on the sea and submerges under the sea (defining the rise-and-fall type in this paper); and an oil fence which is able to be transported to any place (defining the transport type in this paper). At the dolphin berth, the rise-and-fall type or both the rise-and-fall and permanent type are installed. The transport type oil fence at the buoy berth installed around floating hoses and the king-stone valve is generally used for loading ballast.

Some deep sea terminals prohibit starting oil handling and loading ballast during night time in order to prevent oil spilling. Most deep sea terminals for 200,000 D.W.T. class tankers have oil skimmers although the belt type, the suction type and the rotary type oil fences have been used. The following new types, which provide higher efficiency, have been developed: a clean sweep by Lockheed Company, Ltd.; a rotary belt type and a centrifugal separator by Bridgestone Rubber Company, Ltd.; a rotary type and shuttering board separator by J.B.F. Company, Ltd.; and, an inclined board type by Mitsui Marine Development Company, Ltd.

Corrosion protection

Deep sea terminals are exposed to marine environment, and as steel is the chief material used in construction of berths, adequate protective measures against corrosion must be adopted. Steel piles of pier-type dolphins are designed with need to limit corrosion of steel. The main method usually adopted is the cathodic protection, as that by galvanic anodes and power-impressed method. Some piles are covered or painted with coal tar, epoxy resin and cement concrete. Another method is to design steel members in anticipation of thickness allowance against corrosion.

Buoys and anchors for buoy berths are mostly painted with anti-corrosive paint.
Main methods for oil pipelines are cement concrete lining, painting and cathodic protection.

Steel members of facilities built above deep sea terminals are plated with zinc or are painted.

TABLE 1. Guidance of operational conditions

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Wind Speed $V_w$ m/sec</th>
<th>Wave height $H_w$ m</th>
<th>Current speed $V_c$ kt</th>
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</thead>
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<tr>
<td>Conditions for pilot's boarding</td>
<td>15</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Conditions for berthing and mooring</td>
<td>15</td>
<td>1.5</td>
<td>-</td>
</tr>
<tr>
<td>Conditions for unloading</td>
<td>15</td>
<td>1.5</td>
<td>-</td>
</tr>
<tr>
<td>Condition for undocking</td>
<td>20</td>
<td>2.0</td>
<td>-</td>
</tr>
</tbody>
</table>
SEMI-SUBMERSIBLE WORK VESSELS FOR OFFSHORE OPERATIONS IN ROUGH SEAS

Tatsuo Ito, Isao Ueno, and Sadao Ando
Ship Research Institute
Japan

INTRODUCTION

When offshore drilling and dredging began water was very shallow, but drilling depths rose rapidly as new oilfields were exploited. Working platforms and vessels show a marked development designed to meet the needs for greater depths, in succession to fixed platforms or jack-up barges in shallow water. The important factor which bears most heavily on all offshore operations is the effect of ocean or sea conditions on performance efficiency.

The semi-submersible work vessel concept will represent a major improvement over conventional designs. The inherent stability and large deck area of a semi-submersible work vessel has the possibility of making it an ideal vehicle for servicing offshore operations in severe ocean environments. The semi-submersible, in general, has a much longer natural period of vertical motion than the conventional surface vessel, which will have beneficial influence on the vessel motions in rough seas. An investigation of the semi-submersible work vessel has therefore been taken up and pursued at the Ship Research Institute.

This paper shows some of the preliminary results, in particular, the effectiveness of the semi-submersible concept in reducing the motions of work vessels in rough seas.

Model experiments

As a preliminary, effects of length to width ratio and water-plane area of work vessels upon the vessel motions in regular waves were systematically examined. The models of work vessels have a submerged hull of rectangular cross section with vertical circular columns. The main particulars are shown in Table 1. A model experiment on the work vessel of conventional design was also carried out for comparison. Table 2 shows the main particulars of the model.

The experiments were conducted in regular waves at No. 3 Ship Model Basin of the Ship Research Institute. Test conditions are shown in Table 3. The vessel motions in head seas are shown in Figure 1 and those in beam seas are shown in Figure 2.

Prediction of the vessel motions in actual seas

Prediction of the full scale motions of the work vessels in actual rough seas will necessitate wave information in the concerning sea area. Fortunately, there are a few publications available. The 2nd District Port Construction Bureau of Japan has presented wave data in the coastal zone of Japan (1961), while those in the adjacent sea of Japan,
TABLE 1. Principal particulars of model ship  
(scale ratio = 1/87.3)

**SUBMERGED BODY**

<table>
<thead>
<tr>
<th>Symbolic character</th>
<th>H10</th>
<th>H20</th>
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</thead>
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<tr>
<td>Length (m)</td>
<td>0.6928</td>
<td>0.9798</td>
</tr>
<tr>
<td>Breadth (m)</td>
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<td>0.4899</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>0.1200</td>
<td>0.1200</td>
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<tr>
<td>Plane area (m²)</td>
<td>0.4800</td>
<td>0.4800</td>
</tr>
<tr>
<td>Displacement (m³)</td>
<td>0.0576</td>
<td>0.0576</td>
</tr>
<tr>
<td>Length - Breadth ratio</td>
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<td>2.000</td>
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<tr>
<td>Breadth - Depth ratio</td>
<td>5.773</td>
<td>4.083</td>
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**COLUMN**

<table>
<thead>
<tr>
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<th>C1</th>
<th>C2</th>
<th>C3</th>
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</thead>
<tbody>
<tr>
<td>Diameter (m)</td>
<td>0.400</td>
<td>0.230</td>
<td>0.200</td>
</tr>
<tr>
<td>Height (m)</td>
<td>0.200</td>
<td>0.200</td>
<td>0.200</td>
</tr>
<tr>
<td>Sectional area (m²)</td>
<td>0.1257</td>
<td>0.0415</td>
<td>0.0314</td>
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### TABLE 2. Conventional type working vessel model (symbolic character: Conven.)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Length L (m)</td>
<td>2.200</td>
</tr>
<tr>
<td>Breadth B (m)</td>
<td>1.000</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>0.200</td>
</tr>
<tr>
<td>Draft d (m)</td>
<td>0.100</td>
</tr>
<tr>
<td>Displacement (kg)</td>
<td>208.030</td>
</tr>
</tbody>
</table>

aft and fore shapes: beveled bottom  
scale ratio = 1/87.3

### TABLE 3. Tested condition in regular waves

<table>
<thead>
<tr>
<th>Symbolic Character</th>
<th>Submerged Body</th>
<th>Column</th>
<th>Number of Columns</th>
<th>Draft (m)</th>
<th>$A_w/A_i$</th>
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</thead>
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<tr>
<td>H10-4 B</td>
<td>H10</td>
<td>C2</td>
<td>4</td>
<td>0.224</td>
<td>0.346</td>
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<tr>
<td>H10-4 F</td>
<td>H10</td>
<td>C2</td>
<td>4</td>
<td>0.240</td>
<td>0.346</td>
</tr>
<tr>
<td>H10-5 B</td>
<td>H10</td>
<td>C2</td>
<td>5</td>
<td>0.216</td>
<td>0.433</td>
</tr>
<tr>
<td>H10-5 F</td>
<td>H10</td>
<td>C2</td>
<td>5</td>
<td>0.242</td>
<td>0.433</td>
</tr>
<tr>
<td>H20-2 B</td>
<td>H20</td>
<td>C1</td>
<td>2</td>
<td>0.200</td>
<td>0.524</td>
</tr>
<tr>
<td>H20-2 F</td>
<td>H20</td>
<td>C1</td>
<td>2</td>
<td>0.240</td>
<td>0.524</td>
</tr>
<tr>
<td>H20-6 F</td>
<td>H20</td>
<td>C3</td>
<td>6</td>
<td>0.240</td>
<td>0.393</td>
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<tr>
<td>H20-8 B</td>
<td>H20</td>
<td>C3</td>
<td>8</td>
<td>0.199</td>
<td>0.524</td>
</tr>
<tr>
<td>H20-8 F</td>
<td>H20</td>
<td>C3</td>
<td>8</td>
<td>0.240</td>
<td>0.524</td>
</tr>
</tbody>
</table>

$A_i$: plane area of submerged body  
$A_w$: waterplane area.
Figure 1. Ship motions in regular waves.
Figure 2. Ship motions in regular waves.
as indicated in Figure 3, were published by Yamanouchi and Ogawa (1970). For the North Atlantic Ocean, Walden’s report (1964) is available as well. Figures 4 to 6 afford some instances of those wave data in winter and through the year.

It is supposed that apparent significant wave height and mean wave period could be determined by the wave with cumulative frequency of 70 to 80 percent occurrence through the year. I.S.S.C. standard wave spectrum will then be applicable. The wave spectrum is expressed as follows;

$$\left(\frac{f(\omega)}{H_w}\right)^2 = 0.11 \omega_1^{-1} \left(\frac{\omega}{\omega_1}\right)^{-5} \exp \left[-0.44\left(\frac{\omega}{\omega_1}\right)^{-4}\right]$$

where \(\omega_1 = 2\pi/T_w\), significant wave height \(H_w\) is 3.0 m and mean wave period \(T_w\) is 7.5 sec in this case.

Hence, spectrum density and standard deviation can be obtained by making use of the wave spectrum and frequency response of the vessel motions determined by the model experiments in regular waves. The mean values (1/1), significant values (1/3) and expectation (1/1,000) are given in Table 4.

CONCLUSIONS

Prototype of the work vessel has thus been found favorable for offshore operation in rough seas, which satisfies the conditions that roll and pitch amplitudes will fall below 1°, and that heave and surge amplitudes are to be less than 0.5 m at 1/1,000 expectation. The model experiments have proved the basic advantages of its novel design concepts.

However, there is certainly need of further extensive investigations. Systematic model experiments, in particular, are being conducted to clarify the effects of variations of bow shape and stern configurations of submerged body with various vertical columns of rectangular, ellipsoidal and oblong cross sections.

REFERENCES

"Study on wave properties in the ports and harbors of Japan" 1961. 2nd District Port Construction Bureau of the Ministry of Transport of Japan, March.


Figure 3. Concerning sea zones in the adjacent sea of Japan.

Figure 4. Frequency at the Niigata harbor in Japan (2nd District Port Construction Bureau of the Ministry of Transport of Japan, 1961).
Figure 5. Frequency in the adjacent sea of Japan (Yamanouchi and Ogawa, 1970).

Figure 6. Frequency in the North Atlantic (Walden, 1964).
TABLE 4. Estimation of ship motions in ISSC wave spectrum ($T_w = 7.5$ sec, $H_w = 3.0$ m)

### Head seas

<table>
<thead>
<tr>
<th>Condition</th>
<th>variance (R)</th>
<th>pitching amp. (deg)</th>
<th>heaving amp. (m)</th>
<th>surging amp. (m)</th>
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</thead>
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<tr>
<td></td>
<td>pitch heave surge</td>
<td>1/1 1/3 1/1000</td>
<td>1/1 1/3 1/1000</td>
<td>1/1 1/3 1/1000</td>
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<tr>
<td>H10-4 B</td>
<td>1.11</td>
<td>0.848</td>
<td>0.714</td>
<td>1.39 2.22 4.30</td>
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<tr>
<td>H10-4 F</td>
<td>0.994</td>
<td>0.808</td>
<td>0.795</td>
<td>1.24 1.99 3.85</td>
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<tr>
<td>H10-5 B</td>
<td>1.218</td>
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<td>0.200</td>
<td>1.52 2.44 4.71</td>
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<tr>
<td>H10-5 F</td>
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<td>1.316</td>
<td>0.726</td>
<td>1.14 1.83 3.54</td>
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<tr>
<td>H20-2 B</td>
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<td>0.404</td>
<td>0.182</td>
<td>0.49 0.78 1.51</td>
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<tr>
<td>H20-2 F</td>
<td>0.386</td>
<td>0.304</td>
<td>0.199</td>
<td>0.48 0.77 1.49</td>
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<tr>
<td>H20-6 F</td>
<td>0.596</td>
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<td>0.271</td>
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<tr>
<td>H20-8 B</td>
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<td>0.274</td>
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<tr>
<td>H20-8 F</td>
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<td>0.209</td>
<td>0.208</td>
<td>0.42 0.67 1.30</td>
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<td>Conven.</td>
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<td>0.219</td>
<td>0.442</td>
<td>1.14 1.82 3.52</td>
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### Beam seas

<table>
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<th>Condition</th>
<th>variance (R)</th>
<th>rolling amp. (deg)</th>
<th>heaving amp. (m)</th>
<th>swaying amp. (m)</th>
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<tr>
<td></td>
<td>roll heave sway</td>
<td>1/1 1/3 1/1000</td>
<td>1/1 1/3 1/1000</td>
<td>1/1 1/3 1/1000</td>
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<tr>
<td>H20-2 B</td>
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<td>0.831</td>
<td>0.345</td>
<td>2.09 3.34 6.48</td>
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<tr>
<td>H20-2 F</td>
<td>0.782</td>
<td>0.421</td>
<td>0.656</td>
<td>0.98 1.24 3.02</td>
</tr>
<tr>
<td>H20-6 F</td>
<td>0.419</td>
<td>0.326</td>
<td>0.254</td>
<td>0.52 0.84 1.62</td>
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<tr>
<td>H20-8 B</td>
<td>0.462</td>
<td>0.410</td>
<td>0.347</td>
<td>0.58 0.92 1.79</td>
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<tr>
<td>Conven.</td>
<td>2.696</td>
<td>0.628</td>
<td>0.386</td>
<td>3.37 5.39 10.44</td>
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SECTION 1
ANALYSIS OF THE MOTIONS OF FLOATING BODIES IN WAVES

1. INTRODUCTORY NOTE

The problem of the motions of a ship in waves is one of most complicated subjects of hydrodynamics because a ship has six degrees of freedom of motions. Complications are further increased when a ship is moored at dolphins through nonlinear mooring lines and fenders. The situation is the same for any offshore structure of a floating type, which is considered as the most promising facility for ocean exploitation projects.

According to Professor Tasai of Kyushu University (1971), the approach to the problem of floating structures in waves may be classified as follows:

1) Find the solution of velocity potential for each floating body.
2) Solve the equations of motion with hydrodynamic forces and coefficients of damping being estimated by theory and experiments.
3) Estimate the motions by means of scale model tests in a wave tank.

Though the development of potential theory is most desirable, it is feasible only for a floating body of simple geometry. At present, the solutions of velocity potential are available for a two-dimensional floating body of rectangular section and a floating circular cylinder in an upright position, both including the cases with linear mooring springs. Both theories have been developed by Professor Ijima of Kyushu University (Ijima et al., 1972) under the assumption of small amplitudes of waves and motions of floating bodies. The solutions are exact, but they are cumbersome to calculate because of the infinite series involved. Approximate solutions have been proposed by Dr. Ito, Former Head of the Hydraulic Engineering Division of the Port and Harbor Research Institute (Ito and Chiba, 1972; Ito and Kihara, 1972). He showed that the infinite series can be omitted without serious decrease in accuracy and advocated the advantage of the approximate solutions over the exact ones: e.g., savings in computational work, clarity of physical meaning of each term in the solutions, versatility to various mooring conditions.

The second approach of solving the equations of motion is often employed in the study of ship motions in waves. The equations of motion of a floating body in waves can be expressed as follows (Tasai, 1971):

\[ M_{ij} = F_{Rj} + F_{Ej} + F_{vj} + P_{j} + R_{j}, \]  

(1.1)
where, the subscript $j$ refers to the components in the $j$-direction, and,

\[
\begin{align*}
M & : \text{mass of a floating body} \\
\xi_j & : \text{displacement of a floating body} \\
F_{Rj} & : \text{radiation force, which is caused by waves} \\
& \quad \text{generated by the motion of a floating body} \\
& \quad \text{in still water} \\
F_{Ej} & : \text{exciting force, which is exerted by incident} \\
& \quad \text{waves upon a stationary body} \\
F_{Vj} & : \text{viscous resistance of fluid against the motion} \\
& \quad \text{of floating body} \\
P_j & : \text{hydrostatic restoring force due to the displacement} \\
& \quad \text{of a floating body} \\
R_j & : \text{restraining force due to mooring lines and others.}
\end{align*}
\]

When the motion is rotational, the mass is replaced by the mass moment of inertia and the forces by their moments. Some of the motions such as rolling and swaying are always coupled and must be solved simultaneously. Other motions too are often coupled, depending upon the characteristics of restraining forces.

The radiation force in Eq. (1.1) has a component proportional to and in phase with the acceleration of a body and another component proportional to and in phase with the velocity. Hence, if the motion concerned is not coupled with the motions in other directions, it is written as:

\[
F_{Rj} = -m_j \ddot{\xi}_j - N_j \dot{\xi}_j, \tag{1.2}
\]

where,

\[
\begin{align*}
m_j & : \text{added mass for the motion of a floating body} \\
N_j & : \text{coefficient of wave-making damping.}
\end{align*}
\]

The exciting force is sometimes treated as the sum of the Froude-Kryloff force and the diffraction wave force. The former is proportional to the acceleration of orbital motion of water particles, while the latter has the components proportional to both the acceleration and velocity. Hence it is written as:

\[
F_{Ej} = A_j \ddot{\bar{n}}_j + B_j \dot{\bar{n}}_j, \tag{1.3}
\]

where,

\[
\begin{align*}
A_j & : \text{virtual mass of fluid representing wave effect upon} \\
& \quad \text{a stationary body} \\
B_j & : \text{proportionality factor of wave force in phase with} \\
& \quad \text{orbital velocity} \\
\bar{n}_j, \dot{\bar{n}}_j & : \text{mean orbital acceleration and velocity of water particles.}
\end{align*}
\]
The viscous resistance is essentially the drag force due to the velocity difference between the floating body and water particles, i.e.,

$$ F_{v,j} = C_j \rho \frac{\vec{v}_j - \vec{V}_j}{|\vec{v}_j - \vec{V}_j|}, \quad (1.4) $$

where,

- $C_j$ : proportionality factor of drag force $= \frac{1}{2} \rho C_D S_j$
- $C_D$ : drag coefficient
- $S_j$ : projected area of a floating body
- $\rho$ : density of water.

The hydrostatic restoring forces are proportional to the displacement, i.e.,

$$ P_j = -D_j \ddot{z}_j, \quad (1.5) $$

where,

- $D_j$ : proportionality factor of restoring force.

For heaving motion, $D_j = \rho g A_w$ where $A_w$ is the waterline area of a floating body, $D_j = 0$ for the motions of swaying, surging, and yawing.

By substituting Eqs. (1.2) to (1.5) into Eq. (1.1), the latter is rewritten for the uncoupled components of motion as follows:

$$ (M+m)\ddot{z}_j + N_j \ddot{\xi}_j - C_j |\vec{v}_j - \vec{V}_j| (\vec{v}_j - \vec{V}_j) + D_j \ddot{z}_j = A_j \ddot{\xi}_j + B_j \ddot{\xi}_j + R_j. \quad (1.6) $$

In the analysis of ship motions, the added mass and other hydrodynamic factors are estimated by the technique called the strip theory. According to this technique, a ship's hull is subdivided into stripwise sections and the hydrodynamic forces on these sections are estimated by two-dimensional theory. Results are summed up to yield the total forces. Potential theory of the first approach can also be utilized to estimate the above hydrodynamic factors. Especially, Dr. Ito's approach uses the approximate solutions of velocity potential for the estimation of hydrodynamic forces and then formulates the equations of motion similar to Eq. (1.6).

Difficulty arises in the process of solving Eq. (1.6) from three sources. The first is uncertainty of drag coefficient included in the factor of $C_j$: overestimation of $C_D$ leads to the underestimation of motions at resonance. The second is the nonlinearity of viscous forces. A common technique is to linearize them with equivalent drag coefficient and to resort to successive iteration to solve the interaction between $\xi_j$ and $z_j$. The third is the nonlinearity of restraining forces. This problem will be solved by numerical integration of the equations of motion.

The third approach is well practiced at a number of ship testing tanks and hydraulic laboratories in the world. Though some difficulty is met in modelling the characteristics of restraining forces and interpreting the scale effects, it can be overcome by experience. The Port and Harbor Research Institute has conducted a scale model test of 500,000 DWT tanker's mooring at offshore dolphins, which was introduced at the fourth meeting.
The following two sections introduced the studies on the motions of floating bodies in waves being carried out at the Port and Harbor Research Institute. The studies are intended to investigate the usefulness of Dr. Ito's approach and to promote our understanding of the nature of the problem.

2. MOTIONS OF A TWO-DIMENSIONAL RECTANGULAR BODY IN WAVES

Theory

A two-dimensional floating body of rectangular section shown in Figure 1 is considered. The displacements of and rotation about the center of gravity of the rectangular body are denoted as:

\[
\begin{align*}
\xi &= e^{i\omega t} \\
\eta &= \eta e^{i\omega t} + z_0 \\
\theta &= \theta e^{i\omega t}.
\end{align*}
\]

The incident waves and the waves transmitted behind the rectangular body are expressed as:

\[
\begin{align*}
\eta &= a \exp[i(\sigma t - k(x + i))] \\
\eta_T &= a_T \exp[i(\sigma t - k(x - i))].
\end{align*}
\]

Note that the amplitudes, \(\xi\), \(\eta\), \(\theta\), and \(a_T\) are complex variables, containing phase terms. The exact solutions of \(\xi\), \(\eta\), \(\theta\), and \(a_T\) have been given by Professor Ijima (Ijima et al., 1972) and will be compared with experimental results. But the description of the theory is deleted here because of too many mathematical expressions involved.

According to Dr. Ito's theory, the simultaneous equations for the above complex amplitudes are derived as follows:

Swaying motion;

\[
\xi = -\frac{g t_H}{\omega^2} (a - a_T - \frac{ikf_B}{n} \eta) - \frac{R_H}{2\rho \omega d^2},
\]

Fig. 1. Sketch of floating rectangular body.
Heaving motion;
\[ \zeta = -\frac{\omega_v^2}{\sigma^2 - \omega_v^2} f_B (a - \frac{ikf_B}{n} \zeta) - \frac{\omega_v^2}{\sigma^2 - \omega_v^2} \frac{R_v}{2g\delta}, \] (2.4)

Rolling motion;
\[ \theta = \frac{\omega_R^2}{\sigma^2 - \omega_R^2} \frac{f_H}{GM} (a - a_T - \frac{ikf_B}{n} \zeta) - \frac{\omega_R^2}{\sigma^2 - \omega_R^2} \frac{R_M}{2g\delta\delta GM}, \] (2.5)

Continuity equation;
\[ \frac{ikdf}{n} \zeta - \frac{ikdf}{n} \theta + (1 - iy_B) (a - a_T - \frac{ikf_B}{n} \zeta) = a, \] (2.6)

where,
\[ f_B = \frac{\sinh k(h-d)}{k(h-d) \cosh kh} \]
\[ f_H = \frac{\sinh kh - \sinh k(h-d)}{kd \cosh kh} \]
\[ f_M = f_{MH} + \frac{\delta}{3d} f_B \]
\[ f_{MH} = \frac{1}{k^2 d \cosh kh} \left\{ -kz \sinh kh - \cosh kh + k(d+z_o) \times \sinh k(h-d) + \cosh k(h-d) \right\}, \]
\[ n = \frac{1}{2} \left( 1 + \frac{2kh}{\sinh 2kh} \right), \] (2.8)
\[ \omega_v^2 = \frac{2g\delta}{M+H_1} \]
\[ \omega_R^2 = \frac{2g\delta d \overline{xGM}}{I+I_1}, \] (2.9)

\( k \): wave number = \( 2\pi/L \)
\( \sigma \): angular wave frequency = \( 2\pi/T \)
\( M \): mass of rectangular body = \( 2\rho d^2 \)
\( \overline{xGM} \): metacentric height = \( \frac{x^2}{3d} - \frac{d}{2} - z_o \)

\[ M_1 = \frac{2\rho\delta}{3(h-d)} \left\{ \delta^2 + (h-d)^2 \right\} \]
\[ I_1 = \frac{2\rho\delta^3}{h-d} \left\{ \frac{x^2}{45} + \frac{(h-d)^2}{9} \right\}. \] (2.10)

The restraining forces \( R_H \), \( R_V \), and \( R_M \) are multi-variable functions of \( \xi \), \( \zeta \), and \( \theta \) in general. As a first approximation, they may be assumed to form a linear system.
Equations (2.3) to (2.6) when rewritten in the real and imaginary parts constitute 8 linear simultaneous equations which yield the solutions for the amplitudes and phases of $\xi$, $\zeta$, $\theta$, and $a_T$.

Model Experiment

An experiment was carried out for a freely-floating body of rectangular section in waves. The rectangular body tested was made with steel plates and provided with balance weights in its inside. Its dimensions are,

- Width: $2l = 130$ cm
- Draft: $d = 50.5$ cm
- Height of the center of gravity: $z_o = -19.0$ cm
- Mass moment of inertia: $I = 14.63$ kg·cm·sec$^2$/cm.

The experiment was carried out in a wave flume of 39.6 m in length, 0.48 m in width, and 0.90 m in height. The water depth was kept at $h = 0.65$ m. At this water depth, the rectangular body had the following natural periods of oscillation:

- Heaving period: $T_H = 2.36$ sec
- Rolling period: $T_R = 2.32$ sec.

The wave period was varied from $T = 1.6$ to 3.3 sec with the increment of 0.1 sec, while the wave height was kept around $H = 2$ cm. The motions of rectangular body were recorded on 8 mm motion picture films, which were later projected on a screen for reading.

Figure 2 shows the records of the horizontal movement of rectangular body reconstructed from 8 mm film. Most of runs exhibit strong drifting of the body in the direction of wave propagation.

![Fig. 2. Horizontal motion of the center of gravity observed in experiment.](image-url)
Comparison of Experiment with Theory

The amplitudes of the motions of the rectangular body observed in the experiment are compared with theoretical ones in Figure 3(a) to (c). The amplitudes are normalized by the incident wave amplitude, $a$. The exact solution is shown with dashed lines, whereas the approximate solution is shown with full lines.

In case of swaying motions shown in Figure 3a, experimental data agree with both solutions in the range of short wave period of $T \leq 2.1$ sec, but they do not show a sharp change in sway amplitude predicted by theory around $T = 2.3$ to 2.4 sec. The difference seems to be originated from the assumption of small amplitudes of oscillation, especially of rolling angle, employed in the derivation of theory.

In case of heaving motions shown in Figure 3b, experimental data fairly well agree with the theory. Between the exact and the approximate solutions, no superiority is observed with respect to the degree of agreement with experimental data.

In case of rolling motions shown in Figure 3c, the approximate solution provides slightly better agreement with experimental data than the exact solution. The difference around the resonant period seems owning to the inadequacy of small amplitude assumption and the neglect of viscous resistance force.

Computation of Drift Phenomenon by Means of Numerical Solution

The drift of the rectangular body illustrated in Figure 2 is caused by the residual force of horizontal wave pressures. If the water surface elevations at the front and back of rectangular body are denoted by $\eta_A$ and $\eta_B$, the instantaneous net wave force in horizontal direction is expressed as:

$$\Delta P = \frac{\rho g}{k \cosh kh} \left[ \eta_A \{ \sinh k(h+\eta_A) - \sinh k(h-d+\zeta+i\theta) \} - \eta_B \{ \sinh k(h+\eta_B) - \sinh k(h-d+\zeta+i\theta) \} \right].$$

When the long wave approximation is applicable,* the above equation is approximated by

$$\Delta P = \rho g \left( \eta_A^2 - \eta_B^2 \right) - \eta_A \eta_B (\zeta - d) + (\eta_A + \eta_B) k \dot{\theta}. \quad (2.11)$$

Equation (2.11) is quadratic except for the term of $(\eta_A - \eta_B)d$. Therefore, each term yields a residual force when integrated over one wave period and contributes to the drift of a rectangular body.

In order to take into account such quadratic forces and nonlinear restraining forces for the analysis of the motions of a rectangular body, a numerical method has been developed. The equations of motion are rewritten into the form of different equations. The hydrodynamic forces are estimated with the approximate solution of velocity potential. In computation, numerical integration technique is partially employed. Application of the numerical method to the case of freely-floating body is illustrated in Figure 4. As shown by the line denoted by "swaying", the rectangular body with the test dimensions drifts towards the bay side while showing swaying oscillation. Examination of the phase relation between the water surface elevation and the heaving and rolling motions in the light of Eq. (2.11) indicates that both the motions are contributing to the drift of the rectangular body.

*The deepwater wave approximation also yields the same expression as Eq. (2.11) but without the term of $(\eta_A - \eta_B)d$. 

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Fig. 3a. Swaying amplitudes by theory and experiment

Fig. 3b. Heaving amplitudes by theory and experiment

Fig. 3c. Rolling amplitudes by theory and experiment
Fig. 4. Horizontal motion of the center of gravity

Fig. 5. Trace of the motion of the center of gravity
Another example of the effectiveness of the numerical method is shown in Figure 5, where the trace of the center of gravity of the rectangular body in waves is shown. The numerical method succeeds in predicting the drifting pattern observed in the experiment.

3. MOTIONS OF A VERTICAL CIRCULAR CYLINDER IN WAVES

Theory

A freely-floating circular cylinder in an upright position is considered. The diameter of the cylinder is $2a$, the draft is $d$, and the height of the center of gravity in still water is at $z = z'$. As in the case of rectangular body shown in Figure 1, a circular cylinder performs the motions of heaving, swaying, and rolling; they may be described in Eq. (2.1). A difference from a two-dimensional rectangular body is the equality of the amplitude of transmitted waves at infinity with that of incident waves.

Though the exact solution obtained by Professor Ijima will be compared with experimental data later, a summary of the approximate solution by Dr. Ito is presented here. The exciting forces and moment are given as follows:

Horizontal force, exciting the swaying motion:

$$F_{Ex} = \frac{4\rho g d a f H}{k h H^*} e^{i\omega t}. \quad (3.1)$$

Uplift force, exciting the heaving motion:

$$F_{Ez} = \frac{2\rho g a f B}{k t H_o(2, (k t)} e^{i\omega t}. \quad (3.2)$$

Moment of wave force about the y-axis passing through the center of gravity, exciting the rolling motion:

$$M_{Ey} = \frac{4\rho g a f' H}{k h H^*} e^{i\omega t}. \quad (3.3)$$

where,

$$H^* = H_1^{(2)'}(k t) - y_B H_1^{(2)}(k t)$$

$$y_B = \frac{2 \rho g}{\sigma^2 k} \frac{k (h-d)}{n} f_B^2 \frac{2}{f_B}$$

$$f'_M = f_{MH} + \frac{3}{4d} f_B.'$$

The parameters $f_B$ and $f_{MH}$ are given by Eq. (2.7), and $H_1^{(2)}$ denotes the Hankel function of second kind, which generally takes a complex value.

The linear equations for the complex amplitudes of swaying, heaving, and rolling motions are derived as follows:

$$-\omega^2 M \xi = \frac{\rho g d f H}{H^*} \left\{ \frac{41}{k t} a - \frac{\pi k d}{n} H_1^{(2)}(k t) (f_H \xi - f_M \xi) + R_H^* \right\} \quad (3.5)$$

\[\text{1254} \quad \text{Kawakami/Goda/Kihara 10}\]
\[-(c^2 - \omega_{VC}^2)(M_o + M_c)\zeta = \frac{\pi g \xi^2}{H_o^2} \left\{ \frac{f_B}{(k\xi)} \left( 2 \frac{k\xi}{\pi k\xi} a - \frac{k\xi f_B}{2a} \right) \right\} x_{H_o^{(2)}(k\xi)\zeta} + R_V, \tag{3.6}\]

\[-(c^2 - \omega_{RC}^2)(1 + I_c)\theta = -\frac{\rho g \xi^2 f_M}{H^2} \left\{ \frac{4i}{k\xi} a - \frac{i k d}{n H_1^{(2)}(k\xi)} \right\} x(f_H\xi - f_M k\theta) + R_M, \tag{3.7}\]

where,

\[
\begin{align*}
\omega_{VC}^2 &= \frac{\pi g \xi^2}{M_o + M_c} \\
\omega_{RC}^2 &= \frac{\pi g \xi^2}{I_o + I_c} \\
M_c &= \frac{\pi g \xi^2}{h - d} \left\{ \frac{\xi^2}{8} + \frac{(h - d)^2}{3} \right\} \\
I_c &= \frac{\pi g \xi^4}{12(h - d)} \left\{ \frac{\xi^2}{8} + \frac{(h - d)^2}{3} \right\} \\
M_o &\text{: mass of circular cylinder} \\
I_o &\text{: mass moment of inertia about the center of gravity.}
\end{align*}\tag{3.8}
\]

The angular frequencies given by Eq. (3.8) are close to the natural angular frequency of heaving and rolling, but they are not the same because there exists a small quantity of added mass and moment which depend on the frequency of oscillation. In case of a two-dimensional rectangular body, the approximate solution does not yield the frequency-dependent added mass and inertia.

Equations (3.5) to (3.7) when rewritten in the real and imaginary parts yield 6 linear simultaneous equations for the solutions of the amplitudes and phases of \(\xi, \zeta, \text{ and } \theta\).

Model Experiment

A series of experiments were carried out for freely-floating circular cylinders in upright positions in waves. Five cylinders were made with acrylate cylinders and tested. The heights of the center of gravity were so adjusted to be positioned at the mid-draft by placing circular steel plates inside the cylinders. The dimensions and natural periods of test cylinders are listed in Table 1. The natural periods are those measured in still water with the depth of \(h = 40 \text{ cm}\).

The experiments were conducted in a wave flume of 40.0 m in length, and 1.50 m in both width and height. The water depth was kept at \(h = 40 \text{ cm}\) throughout the experiments. The wave period was varied from \(T = 0.69\) to 1.79 sec. Wave heights were controlled at two levels of about \(H = 1\) and 2 cm for the measurement of the motions of cylinders, while the wave heights of about \(H = 4\) cm were employed for the measurement of wave forces.

The motions of the center of gravity of each cylinder were measured with a special TV camera device called "X-Y analyzer", which yields two voltage outputs proportional to the

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TABLE 1. Dimensions of Circular Cylinders Employed in Experiments.

<table>
<thead>
<tr>
<th>Cylinder No.</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter, (2t) (cm)</td>
<td>20.0</td>
<td>20.0</td>
<td>20.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>Draft, (d) (cm)</td>
<td>30.0</td>
<td>20.0</td>
<td>10.0</td>
<td>30.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Height of center of gravity, (z_0) (cm)</td>
<td>-15.0</td>
<td>-10.0</td>
<td>-5.0</td>
<td>-15.0</td>
<td>-10.0</td>
</tr>
<tr>
<td>Mass moment of inertia, (I_o) (kg\cdot cm\cdot sec(^2))</td>
<td>0.738</td>
<td>0.392</td>
<td>0.173</td>
<td>0.223</td>
<td>0.105</td>
</tr>
<tr>
<td>Heaving period, (T_V) (sec)</td>
<td>1.23</td>
<td>1.05</td>
<td>0.78</td>
<td>1.17</td>
<td>0.95</td>
</tr>
<tr>
<td>Rolling period, (T_R) (sec)</td>
<td>2.30</td>
<td>1.80</td>
<td>1.02</td>
<td>7.71</td>
<td>5.06</td>
</tr>
</tbody>
</table>

coordinates in \(x\) and \(y\) of the brightest spot on the screen of a Braun tube. The exciting forces were measured with a six-component dynamic balance.

Exciting Forces

Examples of the measured exciting forces are presented in Figures 6 and 7, where normalized amplitudes of \(F_{Ex}\) and \(F_{Ey}\) are plotted against the relative water depth. The exciting force in the \(z\)-direction or the uplift force uniformly decreases as the relative water depth increases. The approximate theory predicts smaller values of \(F_{Ey}\) than the exact theory; the latter shows better agreement with experimental data than the former. Though the deviation of the approximate theory from the exact one is large for cylinder Nos. 3 and 5 (shown in Figure 6) the deviation is small for other cylinders. In general the deviation decreases as the draft increases or as the diameter increases.

As for the exciting force in the \(x\)-direction or the horizontal wave force, experimental data fall lower than theory in the range of deepwater waves. The cause of the difference has not been examined. The tendency of underestimation of exciting force by the approximate theory is the same as in the case of \(F_{Ey}\).

The moment of exciting force about the \(y\)-axis was also measured. The experimental results were almost in agreement with the exact theory although data show some scatter as in the cases of \(F_{Ex}\) and \(F_{Ey}\). The performance of the approximate theory is the same as before.

Swaying and Heaving Motions

Examples of the amplitude of swaying motions are shown in Figure 8. According to theory, floating cylinders are expected to show abrupt change in swaying amplitude across the resonant point of rolling. The resonant point is calculated as \(\sigma^2h/g = 1.55\) and 0.06 for the cylinders Nos. 3 and 5, respectively, on the basis of the natural periods of rolling listed in Table 1. Experimental data, however, do not indicate such singularity but gradually vary across the resonant point of rolling. The situation is the same with the swaying motion of a rectangular body shown in Figure 3(a). Discrepancy probably originates from the inadequacy of small amplitude assumption and the neglect of viscous damping effect in theory.

Examples of the amplitude of heaving motions are shown in Figure 9. As expected, the heaving amplitude becomes very large at the resonant point, which is calculated as \(\sigma^2h/g = 1.55\) and 0.06 for the cylinders Nos. 3 and 5, respectively, on the basis of the natural periods of rolling listed in Table 1. Experimental data, however, do not indicate such singularity but gradually vary across the resonant point of rolling. The situation is the same with the swaying motion of a rectangular body shown in Figure 3(a). Discrepancy probably originates from the inadequacy of small amplitude assumption and the neglect of viscous damping effect in theory.
Fig. 6. Amplitude of exciting force in the z-direction
Fig. 7. Amplitude of exciting force in the x-direction
Fig. 8. Amplitude of swaying motions
Fig. 9. Amplitude of heaving motions
= 2.65 and 1.78 for the cylinders Nos. 3 and 5, respectively. The approximate theory is unsuccessful in predicting the resonant points for the cylinders Nos. 3 and 5, but the deviation becomes small as the draft increases. In fact, the approximate theory gives almost identical results with those by the exact theory for the cylinders Nos. 1 and 4 with the draft of 30 cm. An interesting point in heaving motions at resonance is the fact that the relative heave amplitude decreases as the wave amplitude increases. This is considered due to the viscous effect: i.e., the influence of the term of $-C_j |\hat{n}_j - \xi_j| (\hat{n}_j - \xi_j)$ in Eq. (1.6) which is neglected in potential theory.

Drift Phenomenon

As in the case of a rectangular body, a freely-floating circular cylinder is easily put on drift by waves. Figure 10 shows the measured drift velocity $V_d$ in terms of the maximum orbital velocity of water particles, $u_{\text{max}}$, where

$$u_{\text{max}} = \frac{gka}{c} = \frac{\sigma a}{\tanh kh}$$

(3.10)

The relative drift velocity $V_d/u_{\text{max}}$ becomes very large at the resonant point of heaving, approaching the value of 0.9 for the cylinder No. 3. This clearly indicates the coupling effect of water surface elevation and heaving motion being in reverse phases (refer to Eq. 2.11). Amplification of the drift at the resonant point of rolling is rather weak with some shift in peak frequency. This may be due to the small diameter of cylinders tested.

Another point of interest is that the relative drift velocity near resonance seems unaffected by the absolute magnitude of wave heights in the range of experiment, although the drift force is proportional to the square of wave amplitude. An additional test carried out for a wider range of wave heights, however, indicates that the drift velocity away from the point of heaving resonance is almost proportional to $a^2$.

4. CONCLUDING REMARKS

Theory of floating body with the solution of velocity potential is very effective in predicting the motions of floating body in waves, as demonstrated in the preceding sections. The approximate solution developed by Dr. Ito is useful especially when the draft of floating body is large. For the problem of tanker's mooring at offshore dolphins, the approximate solutions should provide a powerful means of analysis, because the bed clearance is usually small and the tanker's hull can be approximated with rectangular sections. The numerical method based on the approximate solution will be fully utilized to deal with nonlinear mooring problems.

Three problems remain to be solved in the near future. The first is the incorporation of viscous term in the equations of motion. Appropriate estimation of drag coefficient is one task and the development of numerical technique to deal with the quadratic term is another task. The second problem is the modification of two-dimensional analysis for three-dimensional effects, including wave approach from oblique angle. Theory of wave interaction with a vertical cylinder of elliptical shape may provide an answer to the three-dimensional effects. The third problem is the wave irregularity effects. There are two aspects of the problem. One is the directional spreading of wave energy, characterized by directional wave spectra. This will be dealt with as an application of three-dimensional effects. The other is the effect of drift force on mooring forces. Since the drift force is quadratic with wave amplitudes, linear spectral calculation cannot be employed if the effect of drift force is significant. A technique of equivalent linear drift force is required to be developed, or numerical calculation with simulation of irregular waves will become necessary.
Fig. 10. Drifting velocity of vertical circular cylinder
SECTION 2

PERFORMANCE AND ANCHORING FORCE OF A TIGHTLY-ANCHORED FLOATING BREAKWATER OF CIRCULAR SECTION

Principles and Advantages

A floating breakwater of circular section is immersed into water by tight anchoring in such a manner that the anchor chains will never be slacked, even at the time of wave troughs (Fig. 11).

The breakwater has the following dynamic advantages: 1) In general, a floating breakwater which is tightly anchored by two lines of chains may experience rotational motion about the intersection point of anchoring lines, when it is subjected to wave actions. But if the floating breakwater has a circular section and the intersection point coincides with the center of the circle, there is no turning force because all wave forces which act on the surface of the circle are concentrated to the center of the circular section. Even if the breakwater rotates, it generates little waves because the section is circular, and 2) if anchor chains are kept on the stretch all the time, there will be no impact tensions which may act on the anchoring chains suddenly stretched from the shock state.

RESULTS OF HYDRAULIC MODEL TESTS

Tensions of anchoring chains and the wave transmission coefficient of breakwater were measured by hydraulic model tests for cases shown in Figure 12. The ranges of wave heights were from about 3 to 17 cm, and the ranges of wave periods were from about 1.57 sec to 3.35 sec, except for in Case 3, where the range was from 1.61 sec to 2.68 sec. The tensions and transmitted wave heights are linearly proportional to the incident wave heights, as shown in Figures 13 and 14.

APPLICABILITY OF APPROXIMATE THEORY

Since the oscillation of a circular section of floating breakwater under the tight anchoring condition is negligibly small, the breakwater may be represented with a semi-submerged, fixed circular cylinder. However, the theory of a semi-submerged, fixed cylinder in waves is very complicated (Tasai, 1971).
Fig. 13a. Amplitude of tension of anchoring chains against incident wave height.

Fig. 14. Transmitted wave height against incident wave height.

Fig. 13b. Definition of tension forces and wave heights

Fig. 15. Comparison of experimental tension amplitudes with approximate theory.

Hence, an approximate theory for a fixed rectangular beam (Ito and Chiba, 1972) is employed for the breakwater model to compare with experiments. As shown in Figure 15, theoretical values and test results of $P_F$ (amplitude of tension of front-side anchoring chain) are nearly the same, and theoretical values of $P_B$ (amplitude of tension of back-side anchoring chain) are slightly larger than the test results. The theoretical values of wave transmission coefficient are larger than the test results as shown in Figure 16.

On the basis of the above comparison, approximate values of $P_F$, $P_B$, and $K_T$ can be estimated by the approximate theory of fixed rectangular beam. ($K_T$ is the wave transmission coefficient.)
EFFECT OF WAVE BREAKING

In the case of wave breaking, the test models of breakwater did not move, and did not induce a large wave force. The tensions of anchoring chains are nearly the same or rather smaller than the theoretical values.

IMPACT TENSION UPON ANCHOR CHAINS AT SLACK STATE

In a moment when a slacked anchor chain is suddenly stretched, the impact tension is exerted. A comparison of the tension of initially stretched chains and that of slacked chains is shown in Figure 17. The upper figure is the record of tensions for the Case 1 of Figure 12.

![Graph](image)

Fig. 16. Comparison of experimental wave transmission coefficient with approximate theory

![Graph](image)

Fig. 17. Records of tensions for the cases of initially stretched and initially slacked conditions of anchoring chains.

The lower figure is the record of tensions when the water level of the Case-1 was lowered by 10 cm, so as to cause the anchor chains to slack at wave troughs. Tensions of initially stretched chains vary smoothly like sine curves, but the tensions of initially slacked chains rise suddenly like impact.

MODIFICATION OF SECTIONAL SHAPE OF BREAKWATER

Modification of sectional shape was tried as sketched in Figure 18 and tested for the performance.

![Graph](image)

Fig. 18. Sketches of original and modified shapes of circular sections.
1) Bottom-cut model: In order to reduce the wave uplift force, a quarter of bottom of outer cylinder was cut. It was observed that $P_F$ was reduced by nearly 10 percent, but the wave transmission coefficient increased by nearly 4 percent.

2) The circular section with cuts at bottom and top: When the submergence is increased to 16 cm, the tensions of anchor chains were reduced, without reducing the efficiency of breakwater.

ON THE PRACTICAL APPLICATION OF THE RESULT

For the tightly-anchored floating breakwater of circular section with its center being located at the still water level, the tension amplitudes of anchoring chains and the wave transmission coefficient have been calculated by the approximate theory and are shown in Figures 19 to 21. Figure 22 shows the amplitudes of tensions of anchoring chains for various tide levels, in the case of $D$ being equal to $h$. From these figures, the maximum tensions of anchoring chains of the breakwaters shown in Model-1 of Figure 23 are calculated as 30.4 t/m for $P_{F_{max}}$ and 24.3 t/m for $P_{B_{max}}$. On the other hand the maximum tensions of Model-2 of Figure 23 are given as 21.0 t/m for $P_{F_{max}}$ and 17.5 t/m for $P_{B_{max}}$ by hydraulic model test.

If the cohesion of sea bed subsoil is equal to 3.5 t/m$^2$, and if we use anchoring piles of steel pipes which have the diameter of 1.0 m and the depth of penetration of 30 m, the spacing of anchoring piles for the Model-1 breakwater will have to be smaller than 5 m.
for the front-side anchoring chain and 6.4 m for the back-side anchoring chain, whereas, the spacing of anchoring piles for the Model-2 breakwater will have to be smaller than 7.4 m for the front-side anchoring chain and 8.9 m for the back-side anchoring chain.

It will be concluded that in order to put this type of floating breakwater to practical use, further research and large scale tests in the field will be required.

Fig. 20. Amplitude of tension of back-side anchoring chain calculated by approximate theory

Fig. 21. Wave transmission coefficient calculated by approximate theory
Fig. 22. Amplitudes of tensions of anchoring chains for different tide levels

Fig. 23. Models of floating breakwater for calculation of anchoring forces
REFERENCES


ACKNOWLEDGMENTS


INTRODUCTION

Nippon Kokan has laid submarine pipelines up to 48-in outside diameter mainly for use in connection with sea berths with the use of two lay barges, Anzenmaru and No. 2. Anzenmaru, constructed and owned by NKK.

In view of the increasing demand for submarine pipelines, the company constructed Kokan Pioneer I (among the world's largest submarine pipe laying barges) for use as a high-efficiency pipe laying barge as well as a derrick barge. The barge is capable of laying maximum outside diameter 56-in pipe (with concrete coating outside diameter 70 in). It was completed in February 1974 at NKK's Shimizu Shipyard.

This paper is a technical report on the particulars of Kokan Pioneer I (Fig. 1), and includes a summary of the submarine pipe laying construction work completed in June 1975 at the oil/gas field at Aga which is located in the Japan Sea some 11 km off the coast of Niigata Prefecture.

The project, in which NKK participated, is Japan's first full-scale development of continental shelf resources. The submarine pipe-laying construction work called for joining pipes and laying them on the sea bottom over the distance from the drilling platform installed off the coast to a land-based operation control terminal which is under construction. The drilling platform was erected in September 1974.

OUTLINE OF KOKAN PIONEER I

Outline

The barge is a non-self-propelled steel deck pontoon-type with forecastle. The main deck has a plant for laying submarine pipelines and the aft section is fitted with a revolving crane used for handling heavy equipment. Below the main deck are living quarters, engine room, anchor winches, etc.

The main hull is divided by two longitudinal watertight bulkheads and seven transverse watertight bulkheads. In way of the engine room and hydrophore/air conditioning machinery room, is a double bottom.

The greater part of the main deck is covered with wooden planks and used as a pipe depository. The aft section has facilities for storing gas used in welding and cutting operations. A crawler crane is used for handling pipe and miscellaneous materials.
Figure 1. Overall view of Kokan Pioneer I
The pipe launching way, located on the starboard side, has pipe laying facilities and equipment such as line-up station, tensioners, welding machines, x-ray unit, roller units, coating station, concrete coating station, pipe handling davits, control house, etc.

Living accommodations for 220 persons are provided amidship. The ceilings, bulkheads and furniture in the public rooms (and other rooms) are made of steel.

Ten anchor winches are installed for and aft.

Power is supplied by four generators, driven by diesel engines, and installed in the engine room. The engine room is equipped with four main generators, two auxiliary generators, and two auxiliary boilers.

The main specifications are as follows:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length overall</td>
<td>about 139.0 m</td>
</tr>
<tr>
<td>Length between perpendiculars</td>
<td>131.5 m</td>
</tr>
<tr>
<td>Breadth molded</td>
<td>30.0 m</td>
</tr>
<tr>
<td>Depth molded</td>
<td>9.0 m</td>
</tr>
<tr>
<td>Designed loaded draft molded</td>
<td>4.5 m</td>
</tr>
<tr>
<td>Diesel oil tank capacity</td>
<td>1300 m³</td>
</tr>
<tr>
<td>Fresh water tank capacity</td>
<td>1000 m³</td>
</tr>
<tr>
<td>Ballast water tank capacity</td>
<td>9200 m³</td>
</tr>
<tr>
<td>Windlass</td>
<td>15 x 9 m/min</td>
</tr>
<tr>
<td>Generator capacity</td>
<td>1500 kva, four (4) units</td>
</tr>
<tr>
<td>Complement</td>
<td>200 kva, two (2) units</td>
</tr>
<tr>
<td>(during pipe laying operation)</td>
<td>220</td>
</tr>
<tr>
<td>(during navigation)</td>
<td>30</td>
</tr>
<tr>
<td>Classification</td>
<td>A.B.S. A-1 Pipe Lay Barge</td>
</tr>
<tr>
<td>Regulation</td>
<td>Japanese Government Safety Rules of Shipping</td>
</tr>
</tbody>
</table>

General Outfitting

Living quarters

Accommodation space is as shown in the General Arrangement (Fig. 2). The engine room is arranged in the bow. The living quarters are provided with refrigerators, galley and dining saloon. Ample space is provided below the main deck. The ceilings are made of steel for sound insulation. The public and other rooms have steel screens and unlined bulkheads for facilitating maintenance.

Two air conditioning units (central cooling/heating) are installed in each of the following: living rooms (consisting of eight 2-man cabins and 51 4-man cabins), dining room, recreation space, barge operation control room, radio/telephone room and x-ray room.

When the barge is being towed (on international voyage) or during off-duty, the working crew numbers about ten. In this case, the occupied quarters are air conditioned by pack- age type units located in the ship's office.

Machinery

The pipe handling system, including the 10 anchor winches, is powered by electric or hydroelectric motors. Generator capacity includes room for installation of additional pipe handling equipment.
Figure 2. General arrangement of Kokan Pioneer I
Although the barge is non-self-propelled, extra considerations were taken in the design and construction because even a minor accident involving machinery or equipment may cause stoppage of the pipe laying.

A central control room is located in the upper part of the engine room. The control board is fitted with supervision and calculation devices, such as a remote starter for each generator, lubricating oil, fresh water and sea water, compressed air lines, and remote control knobs of AGB for generator and electric power supply.

Main generator engines

- **No:** 4
- **Type:** four-cycle, single-acting, supercharged diesel
- **Rated output:** 1730 PS x 720 rpm

Auxiliary generator engines

- **No:** 2
- **Type:** four-cycle, single-acting, supercharged diesel
- **Rated output:** 250 PS x 1200 rpm

Crane generator engine

- **No:** 1
- **Type:** four-cycle, single-acting, supercharged diesel
- **Rated output:** 720 PS x 1200 rpm

Main air compressors

- **No:** 2
- **Type:** vertical piston, two-stage, one cylinder type, motor-driven, fresh water cooled, with cooling water pump
- **Capacity:** 32 m³/hr at 25 kg/cm² delivery pressure

Emergency air compressor

- **No:** 1
- **Type:** vertical piston, two stage, one cylinder type, diesel-engine driven, air cooled
- **Capacity:** 4 m³/hr at 25 kg/cm² delivery pressure

Pipe laying air compressors

- **No:** 4
- **Type:** vertical piston, two-stage, four cylinder, V-type, motor direct driven, fresh water-cooled with cooling water pump
- **Capacity:** 39 m³/hr at 12.5 kg/cm² delivery pressure

Auxiliary boilers

- **No:** 2
- **Type:** oil burning, water mono-tube boiler (Takuma - Calyton Model RHO-125)
- **Max. evapor.:** 1500 kg/hr x 7 kg/cm²

Evaporators

- **No:** 2
- **Type:** MECO Model No. PEE 330M vapor compression type
- **Capacity:** 30 m³ per day at seawater temperature of 18°C
Fire and ballast pump

No: 1
Type: motor-driven, vertical, centrifugal with priming pump
Capacity: 250/100 m³/hr at 25/55 m total head

General service pump

No: 1
Type: motor-driven, vertical, centrifugal with priming pump
Capacity: 250/100 m³/hr at 25/55 m total head

Ballast and anchor winch cooling seawater pump

No: 1
Type: motor-driven, vertical, centrifugal
Capacity: 500 m³/hr at 20 m total head

Electric Parts

Great care was taken from the design stage to meet the requirements for the large capacity pipe laying equipment and facilities which include various measures for central remote control, precise measuring of barge position, wire communication between various stations and barge control room, lighting at work site, vibration, etc.

Generators

Main generators
No: 4
1500 kva x AC450V x 3φ x 60 Hz x 720 rpm

Auxiliary generators
No: 2
200 kva x AC450V x 3φ x 60 Hz x 1200 rpm

Crane generator
No: 1
625 kva x AC450V x 3φ x 60 Hz x 1200 rpm

Welding machine motor generator
No: 1
400 kva x 202/220 V x 3φ x 60 Hz x 1200 rpm

Interior communication and navigating equipment

50-line automatic exchange telephone system 1 set
Common battery switchboard telephone system (1:10) 1 set
Interphones 2 sets
Loud hailer (bull horn used for issuing instructions and public address system) 1 set
Barge movement "ready" panel 1 set
Quartz crystal clock 1 set
Fog signaling system 1 set
Radar JRC radar model JMA-150 1 unit
Gyro compass (Hokushin model D-1) 1 unit
Echo depth sounder 1 unit
(Kaijo Denki model PS-10-E)
Wind velocity and direction indicator 1 set
Ship position finding apparatus 1 set
Wireless radio equipment

Main SS transmitter  1 set
Main wave receiver  1 set
Ship telephone  1 set
International harbor and bay wireless telephone  1 set
Teletype receiver for weather reports  1 set
Weather map monitoring equipment  1 set
Facsimile recorder  1 unit

Pipe Handling System

The pipes, with anticorrosion and concrete coatings, are delivered by pipe supply barges. They are unloaded directly onto the longitudinal endo-conveyors or onto the main deck storage area with the use of a large crawler crane on the barge. The pipes are conveyed to the bow, transferred onto the transverse conveyor and, after being cut to adequate edge configuration with the use of a beveling machine, sent to the line-up station at the fore end of the launching way.

At the line-up station, each pipe length is supported and rotated by spin lift rollers to position the welding seam on a true line. Then the pipe is supported by moveable (up and down) longitudinal endrollers for proper lining-up for welding of the first layer.

Operation of the conveyors and line-up station, powered by hydraulic action, is controlled remotely from the transfer control panel and line-up control panel.

Each stage of welding is carried out at the five pipe stations of the launching way. The welds are inspected by x-ray equipment and the pipe then passes through the tensioner. The weld area is coated with anticorrosion and concrete coating. On the launching way the pipe is supported by rollers capable of adjusting height, which are installed at 12-m intervals. The pipeline is lowered onto the ocean floor, passing over the stinger.

Principal Particulars of Equipment and Devices

Crawler crane (Manitowoc 4100 W) (Abt. 699 T-M)  1 unit
Longitudinal endo-conveyors  3 units
Transverse conveyor and line-up station  1 set
Beveling machine (NKK-make)  1 set
Welding machine (M-G type)  19 units
X-ray inspection equipment  1 set
Automatic development equipment  1 set
Tensioner  2 units
Tension winch (AED-285 190 t x 15 m/min)  11 units
Support roller  1 set
Stinger (NKK-make)  1 set

Davit System

Six 60-short-ton hydraulically-driven davits are installed on the side of the main deck along the launching way. The one unit on the end of the line is capable of both revolving and luffing; the other five luffing only.
The davits are used for connecting submarine pipeline, and the repair and raising of submerged pipe to near the sea surface, as in the case of installation of riser pipe.

The six davits are remote-controlled from the operation room and can be operated in single or in multiple units. In addition, the tension of each davit can be adjusted by remote control, so that when hoisting and lowering pipeline, the required overall attitude can be maintained in such a manner to prevent placing undue stress on the pipeline.

Large Capacity Revolving Derrick

Lay barges and other work ships having large capacity derricks are very convenient for use in constructing marine structures and submarine pipe laying work. *Kokan Pioneer I* has a jib-type revolving crane having a rating of 513 tons.

Principal capacities and functions of the crane are as follows:

<table>
<thead>
<tr>
<th>Main Capacities and Functions</th>
<th>Rating load capacity</th>
<th>Maximum working volume</th>
<th>Total head</th>
<th>Velocity (m/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main hoisting</td>
<td>480 (t)</td>
<td>18.5 (m)</td>
<td>60.0 (m)</td>
<td>1.0-1.54</td>
</tr>
<tr>
<td>Auxiliary hoisting</td>
<td>110</td>
<td>45.0</td>
<td>75.0</td>
<td>3.0-4.31</td>
</tr>
<tr>
<td>Whip</td>
<td>27</td>
<td>77.5</td>
<td>83.0</td>
<td>10.0-12.67</td>
</tr>
<tr>
<td>Assistant hoist</td>
<td>50 x 2</td>
<td>71.0</td>
<td>75.0</td>
<td></td>
</tr>
<tr>
<td>Luffing</td>
<td></td>
<td></td>
<td></td>
<td>high speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.28 - 3.4 m/min</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>low speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.2 - 1.8 m/min</td>
</tr>
<tr>
<td>Revolving</td>
<td></td>
<td></td>
<td></td>
<td>0.1 rpm</td>
</tr>
</tbody>
</table>

The crane is equipped with a diesel generator which can also be used to provide electric power to the mother ship. Also, the crane is designed for use of large capacity driving equipment.

Boiler

The barge has marine boilers for supplying power for pile driving.

**SUBMARINE PIPE LAYING WORK AT AGA**

Introduction

The Aga Field is estimated to have reserves of about 8,500,000 tons in terms of crude oil. A total of 14 oil wells are to be drilled.

Production is to be started in December 1975. Natural gas production will be some 1,700,000 m³ per day and crude oil, about 320 kl a day.
NKK was awarded the order for pipe laying by the Japan Marine Oil Resources Company. The project was carried out during May and June 1975 covering a distance of about 11 kl using Kokan Pioneer I (Figs. 3 and 4).

At present, the barge is owned by NKK-Brown Root Overseas S.A., a joint venture firm established by NKK and Brown and Root, Incorporated.

The pipeline consists of a 14-in diameter pipe and a 3-in pipe. The 14-in pipe will be used to transport natural gas and crude oil from the platform. The 3-in pipeline will be used for transporting diethlene glycol to the drilling platform from the land-based refinery and operation control terminal. The pipes were joined on the barge and laid. The maximum sea depth of the pipeline route is 80 m in the vicinity of the platform, the deepest of any submarine pipeline in Japan.

Construction Specifications

The specification of the pipelines are shown in Tables 1 through 3.

### TABLE 1. Steel pipe

<table>
<thead>
<tr>
<th>Dia</th>
<th>14 in</th>
<th>3 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>O.D.</td>
<td>355.6 mm</td>
<td>89.1 mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>11.1 mm</td>
<td>5.5 mm</td>
</tr>
<tr>
<td>Length (per length)</td>
<td>12 m</td>
<td>12 m</td>
</tr>
<tr>
<td>Pipe material</td>
<td>API 5LX-X52</td>
<td>API 5LX-X52</td>
</tr>
<tr>
<td>Coating</td>
<td>Ordinary asphalt and vinyl cloth 2 layers (4 mm and thicker)</td>
<td>NEW-PLP (polyurethane) (1.9 mm and thicker)</td>
</tr>
<tr>
<td>Joints</td>
<td>asphalt and vinyl cloth</td>
<td>corrosion resistant tape-sheet 2 layers (Lapco tape) and protective layer (1 mm rubber sheeting)</td>
</tr>
<tr>
<td></td>
<td>asphalt board</td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 2. Concrete coating

<table>
<thead>
<tr>
<th>Water Depth</th>
<th>Concrete Thickness</th>
<th>Concrete Gravity</th>
<th>Submerged Gravity (empty)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 32 m</td>
<td>75 mm</td>
<td>3.0</td>
<td>1.923</td>
</tr>
<tr>
<td>32 - 55 m</td>
<td>64 mm</td>
<td>3.0</td>
<td>1.831</td>
</tr>
<tr>
<td>55 - 80 m</td>
<td>42 mm</td>
<td>2.2</td>
<td>1.344</td>
</tr>
</tbody>
</table>

Note: a) To insure stability in the sea, the 14-in pipe was coated with concrete with the use of molds.  
b) The strength after four weeks was more than 250 kg/cm².  
c) The 3-in pipe had no concrete coating.
Figure 3. Location of platform

Figure 4. Band-based control terminal
TABLE 3. Design factors for load condition of pipeline

<table>
<thead>
<tr>
<th>Water depth</th>
<th>0 - 80 m</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth covering</td>
<td>2.5 m</td>
<td>5,400-m from shore</td>
</tr>
<tr>
<td>Gravity of soil in water</td>
<td>1.0 t/m³</td>
<td></td>
</tr>
<tr>
<td>Tide current</td>
<td>0.5 kt</td>
<td></td>
</tr>
<tr>
<td>Wave height</td>
<td>13.1 m</td>
<td></td>
</tr>
<tr>
<td>Inner pressure</td>
<td>3 in 30 kg</td>
<td></td>
</tr>
<tr>
<td></td>
<td>14 in 70 kg</td>
<td></td>
</tr>
</tbody>
</table>

Weather, atmospheric conditions, etc., offshore Aga are shown in Tables 4 through 7.

TABLE 4. Weather

<table>
<thead>
<tr>
<th></th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ave. pressure</td>
<td>17.2</td>
<td>18.0</td>
<td>17.8</td>
<td>15.8</td>
<td>12.7</td>
<td>09.4</td>
<td>09.4</td>
<td>10.1</td>
<td>13.1</td>
<td>18.0</td>
<td>20.0</td>
<td>18.1</td>
</tr>
<tr>
<td>(+1000)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ave. temp.</td>
<td>1.7</td>
<td>1.8</td>
<td>4.8</td>
<td>10.2</td>
<td>15.3</td>
<td>19.9</td>
<td>24.1</td>
<td>25.8</td>
<td>21.4</td>
<td>15.5</td>
<td>9.8</td>
<td>4.7</td>
</tr>
<tr>
<td>Ave. day max.</td>
<td>4.5</td>
<td>4.9</td>
<td>8.3</td>
<td>15.0</td>
<td>19.9</td>
<td>23.8</td>
<td>27.9</td>
<td>30.2</td>
<td>25.5</td>
<td>19.4</td>
<td>13.5</td>
<td>2.7</td>
</tr>
<tr>
<td>Ave. day min.</td>
<td>-0.4</td>
<td>-1.0</td>
<td>-1.5</td>
<td>1.5</td>
<td>6.1</td>
<td>11.5</td>
<td>16.7</td>
<td>21.2</td>
<td>22.5</td>
<td>18.1</td>
<td>12.2</td>
<td>6.5</td>
</tr>
<tr>
<td>Ave. hum. (%)</td>
<td>77</td>
<td>76</td>
<td>72</td>
<td>71</td>
<td>75</td>
<td>79</td>
<td>82</td>
<td>80</td>
<td>79</td>
<td>77</td>
<td>76</td>
<td>77</td>
</tr>
<tr>
<td>Rain (mm)</td>
<td>194</td>
<td>125</td>
<td>121</td>
<td>104</td>
<td>95</td>
<td>127</td>
<td>193</td>
<td>107</td>
<td>177</td>
<td>165</td>
<td>171</td>
<td>264</td>
</tr>
<tr>
<td>Ave. cloud.</td>
<td>8.8</td>
<td>8.5</td>
<td>7.8</td>
<td>6.7</td>
<td>6.8</td>
<td>7.2</td>
<td>7.2</td>
<td>5.9</td>
<td>7.3</td>
<td>6.9</td>
<td>7.5</td>
<td>8.4</td>
</tr>
<tr>
<td>Clear</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td>7</td>
<td>7</td>
<td>3</td>
<td>4</td>
<td>8</td>
<td>3</td>
<td>5</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Fine</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>5</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Cloudy</td>
<td>13</td>
<td>14</td>
<td>16</td>
<td>13</td>
<td>16</td>
<td>17</td>
<td>17</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>12</td>
<td>11</td>
</tr>
<tr>
<td>Rain</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>6</td>
<td>5</td>
<td>7</td>
<td>6</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Snow</td>
<td>10</td>
<td>7</td>
<td>3</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>Others</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>-</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

TABLE 5. Wind velocity

<table>
<thead>
<tr>
<th>Period (year)</th>
<th>10 years</th>
<th>20 years</th>
<th>30 years</th>
<th>50 years</th>
<th>100 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected max.</td>
<td>wind vel. (m/s)</td>
<td>37.7</td>
<td>40.8</td>
<td>42.7</td>
<td>43.8</td>
</tr>
</tbody>
</table>
### TABLE 6. Waves

<table>
<thead>
<tr>
<th>Period (year)</th>
<th>10 years</th>
<th>20 years</th>
<th>30 years</th>
<th>50 years</th>
<th>100 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave ht. (m)</td>
<td>8.5</td>
<td>9.5</td>
<td>10.1</td>
<td>10.7</td>
<td>11.5</td>
</tr>
<tr>
<td>Period (sec.)</td>
<td>12.6</td>
<td>13.5</td>
<td>13.9</td>
<td>14.4</td>
<td>15.0</td>
</tr>
<tr>
<td>Significant swell ht. (m)</td>
<td>6.6</td>
<td>7.7</td>
<td>8.2</td>
<td>8.8</td>
<td>9.6</td>
</tr>
<tr>
<td>Swell period (sec.)</td>
<td>14.0</td>
<td>15.0</td>
<td>15.6</td>
<td>16.3</td>
<td>17.0</td>
</tr>
<tr>
<td>Max. wave ht. (m)</td>
<td>17.7</td>
<td>19.4</td>
<td>20.4</td>
<td>21.4</td>
<td>22.8</td>
</tr>
<tr>
<td>Upper limit of max. wave ht. (m)</td>
<td>18.5</td>
<td>20.3</td>
<td>21.3</td>
<td>22.2</td>
<td>23.6</td>
</tr>
</tbody>
</table>

### TABLE 7. Tides

<table>
<thead>
<tr>
<th></th>
<th>Niigata East Port</th>
<th>Niigata West Port</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. high tide</td>
<td>HHWL</td>
<td>0.65</td>
</tr>
<tr>
<td>Average high tide</td>
<td>HWOST</td>
<td>0.46</td>
</tr>
<tr>
<td>Average major tide</td>
<td>MHWL</td>
<td>0.43</td>
</tr>
<tr>
<td>Mean sea level</td>
<td>MSL</td>
<td>0.33</td>
</tr>
<tr>
<td>Mean low tide</td>
<td>MLWL</td>
<td>0.19</td>
</tr>
<tr>
<td>Average ebb tide</td>
<td>LWOST</td>
<td>0.17</td>
</tr>
<tr>
<td>Basic level</td>
<td>DL</td>
<td>0.16</td>
</tr>
<tr>
<td>Tokyo Bay mean tide</td>
<td>TP</td>
<td>0.00</td>
</tr>
<tr>
<td>Min. low tide</td>
<td>LLWL</td>
<td>-0.01</td>
</tr>
</tbody>
</table>

**Ocean Bed quality**

- Water depth 0 - 32 m: sand
- Water depth below 32 m: clay

**Welding**

Semiautomatic and manual welding
Semiautomatic welding using shielded CO\(^2\) - Ar gas method.
X-ray inspection - After welding, x-ray inspection to JIS-Z 3104 was conducted on full circumference of all pipe joints (X-raying adopted double-wall single-image technique for 14-in pipe; for 3-in pipe double-wall, double-image technique). The standard for criterion applied exceeded Grade II, JIS-Z 3104.

Inspection of field joint coating - Inspection for pin holes was conducted with the use of a Holiday Detector having a rating of 8,000 V.

Pressure test - After laying submarine pipe, and pipe on land, hydrostatic pressure testing was conducted. Following this, by passing pollypig through the pipe, the inside of the pipeline was cleaned. In addition, air pressure testing is to be conducted when back-filling is completed.

The following were the test specifications:

<table>
<thead>
<tr>
<th></th>
<th>Hydrostatic Pressure Test</th>
<th>Airtightness Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 in</td>
<td>105 kg/cm²</td>
<td>77 kg/cm²</td>
</tr>
<tr>
<td>3 in</td>
<td>53 kg/cm²</td>
<td>39 kg/cm²</td>
</tr>
</tbody>
</table>

Note: The pressure holding time was 24 hr for both tests.

Civil engineering and construction (Carried out by Nippon Steel Corporation)

Prior to laying the pipeline, over a distance of about 600 m from the shoreline, a 40-m-wide trench was dug in the ocean bed with the use of a grab dredge. From the shoreline, after laying the pipeline over a distance of 5,400 m to the platform site, water jet dredging was conducted. The pipe was then buried with an earth covering of more than 2.5 m.

At the shore area, the pipeline was submerged in a manner to ensure a moderate inclination between the height of the earth covering of 2.5 m at the sea bottom and 1.2 m of the land area. During construction, with use of sheet piling, an embankment was erected over a distance of about 85 m at the shoreline.

Measures for corrosion resistance

For preventing corrosion of the outer surface of the pipe, cathodic protection was applied using electric power from an outside source.

Work stations

<table>
<thead>
<tr>
<th>14-in pipeline</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1 station</td>
<td>alignment and first layer welding</td>
</tr>
<tr>
<td>No. 2 station</td>
<td>second layer welding</td>
</tr>
<tr>
<td>No. 3 station</td>
<td>third layer welding</td>
</tr>
<tr>
<td>No. 4 station</td>
<td>reserve station</td>
</tr>
<tr>
<td>No. 5 station</td>
<td>x-ray inspection</td>
</tr>
<tr>
<td>No. 6 station</td>
<td>corrosion resistant coating</td>
</tr>
<tr>
<td>No. 7 station</td>
<td>concrete coating</td>
</tr>
<tr>
<td>No. 8 station</td>
<td>securing 14-in x 3-in pipe together with steel bands</td>
</tr>
</tbody>
</table>

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3-in pipeline

A station alignment and welding *
B station x-ray inspection *
C station corrosion resistant coating *

Pipe laying fleet

1. Laybarge Kokan Pioneer I 1
2. Anchor tugs (see anchor tugs specifications below) 2
3. Supply barges (1,000 t and 3,000 t) 2
4. Ferry and tug boats 2
5. Submersible work craft 1
6. Anchor barge 1
7. Chucker boat 1

The following are the specifications of the anchor tugs:

No 3 Shinsan Maru (Fig. 5)

Length (o.a.) 35.4 m
Length (p.p.) 31.0 m
Breadth (M) 9.5 m
Depth (M) 4.1 m
Draft (full load) 3.5 m
Displacement 770 m
Main engine Daihatsu 8DSM-26

Type: 4-cycle, single-acting, supercharged diesel engine 2 sets
Output: 1,600 PS x 2

Propeller (IHI D.P. - 40B)

Type: Duck propeller, 4-bladed, copran, solid type
Speed: Max. 13 knots
Towing force: Bollard tension (max) 40.0 T
Classification: Greater Coast Service (not international)
Rule: Japanese Government Safety Rules of Shipping

Towing winch:

Type: Hydraulic x 1
Capacity: 40/8T x 8.5/35 m/min, Brake 80T

Hydraulic pump unit: 1 set hydraulic pump unit driven by electric motor for windlass and winches, installed in engine room.

* The 3-in pipe was laid concurrent with the work of laying the 14-in pipe. Prior to laying the 14-in pipeline, A, B, and C stations were temporarily constructed on the main deck.
Figure 5. No 3 Shinzan Maru
Fujin Maru (Fig. 6)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (o.a.)</td>
<td>38.8 m</td>
</tr>
<tr>
<td>Length (p.p.)</td>
<td>35.0 m</td>
</tr>
<tr>
<td>Breadth</td>
<td>9.2 m</td>
</tr>
<tr>
<td>Depth</td>
<td>4.2 m</td>
</tr>
<tr>
<td>Draft</td>
<td>3.3 m</td>
</tr>
<tr>
<td>GT</td>
<td>299.74 T</td>
</tr>
<tr>
<td>Main engine</td>
<td>Niigata E. Co. 8MG 25 Bx</td>
</tr>
</tbody>
</table>

Type: 4-cycle, single-acting, inter-cooler and supercharged 2 sets

Output: 18,000 PS x 2 (max. 3,900 CHP)

Propulsion: fixed propellers 2 sets

Kort Nozzle Rudder

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bow thruster:</td>
<td></td>
</tr>
<tr>
<td>Type:</td>
<td>TF-20</td>
</tr>
<tr>
<td>Thrust:</td>
<td>2.2 T</td>
</tr>
<tr>
<td>Speed:</td>
<td>(max.) 13.5 knots</td>
</tr>
<tr>
<td>Navigation range:</td>
<td>(abt.) 8,600 miles</td>
</tr>
<tr>
<td>Bollard pull:</td>
<td>45 T</td>
</tr>
</tbody>
</table>

Anchor and towing winch:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type:</td>
<td>HTT-40, 40 T/8T x 8.5/35 m/min</td>
</tr>
<tr>
<td>Prime mover:</td>
<td>diesel engine</td>
</tr>
<tr>
<td>Output:</td>
<td>185 PS x 1,200 rpm</td>
</tr>
</tbody>
</table>

Pipe Laying Work

Preliminary survey

Prior to the pipe laying work, the following matters were surveyed:

1. basic point  2. depth  3. shoreline  4. ocean bed
5. bottom soil  6. boring  7. tide current  8. water specific gravity
9. magnetometry 10. weather conditions 11. ocean weather
12. sea transportation and routes, port, bay and vicinity

Installation of riser pipe

In the area of the production platform, the sea depth is more than 80 m. Therefore, the conventional riser pipe installation method is difficult. The conventional method is to lay pipeline from the shore, and hoist it in front of the platform and then connect it with the riser pipe. The joined pipe is then lowered and secured to the platform.

In the Aga pipe laying project a guide rail was first installed on the jacket of the platform. In addition, guide rollers were attached to the guide rail. Then the pipeline was joined and extended in a horizontal attitude from the barge. The heel portion was joined with the guide roller. As each length of the riser pipe was added, the riser pipe was gradually lowered onto the ocean bed.

However, on the barge, pipe was connected in accordance with the number of riser pipes already installed, as the barge was advanced.
Figure 6. Fujin Maru
To ensure a safe bending curve of the pipeline from the barge to the heel of the riser pipe, the barge was moved to provide the precalculated pipeline curve.

The following shows the process of the work (Figs. 7 through 10):

1. Installation of guide rail (Fig. 6)
   a) Hoisting the guide rail with use of revolving crane and crawler crane on the barge
   b) Shifting of guide rail to crane on platform (Fig. 9)
   c) Attaching the guide rail to clamps and tightening

   Note: Originally the guide rail of the riser pipe should be installed and integrated with the platform structure while the platform is being constructed. However, in the Aga on-site construction work, as the progress of the work was behind schedule, only the supports of the guide rail were installed when constructing the platform.

2. Riser pipe installation (Fig. 10)
   a) Installation of guide roller on guide rail
   b) Hoisting heel portion of riser pipe and fitting it to the launching way
   c) Joining heel portion and pipeline and extending pipeline on stinger from barge toward the platform, the heel being hoisted by the revolving crane
   d) Shift hoisting of riser heel to platform crane
   e) Linking riser heel to guide roller
   f) Lowering riser pipe by connecting it with added riser pipe, and joining the pipeline with forward movement of barge
   g) Securing riser pipe to platform

3. Ordinary pipe laying work

   After installation of the riser pipe, over the offshore distance of some 600 m from the shoreline, conventional submarine pipe laying work was carried out.

   During the pipe laying, tension on pipe and buoyancy of stingers was adjusted in accordance with changes of water depth and specific gravity of pipe which varied according to thickness of concrete coating.

   During movement of the barge, the operation of each anchor winch was controlled with the use of a TV monitor installed at each anchor winch and No. 1 station, and use of the ship position finding apparatus (YM-100: jointly developed by Fuyo Ocean Development Company and Yamatake-Honeywell Company, Ltd.).

   After each approximately 500-m advance of the barge, it was re-anchored. At relocations of great depth and where the sea bottom consisted of clay soil, the anchor was buried thereby allowing high anchoring efficiency but resulting in difficulty of hoisting.

   At relatively shallow locations, the efficiency of the anchor was not very good because of the sandy nature of the sea bottom.

   The combined use of the barge and anchor boats was not efficient at first. However, with the progress of the work this was improved.

   In the conventional laying part of the project, blind plates were welded to the pipeline end. A tension cable was connected to the blind plate and proper tension applied by use of a tension winch. Then the tension of the tensioners was released. The pipe was fitted to a hoisting cable and marker buoy for the purpose of tie-in. And the pipe end was gradually lowered to the ocean bed, the tension cable extended and the barge moved.
Figure 7. Installation of guide rails
Figure 8. Platform and guide rails
Outline of structure of riser guide rail

Installation of riser guide rail

Figure 9. Shifting of guide rail to crane on platform
The barge carried about half of the total required pipe when it was towed from Niigata East Port. At sea, the remaining pipe was loaded from a 3,000-ton barge.

Pipe laying at shore area

Laying by submarine towing - After sinking the pipeline to the ocean bed, the anchor was hoisted. Then four segments of the stinger were shortened to one. At the same time, the barge was turned 180° and positioned at a work site some 380 m offshore. In anchoring the barge, two anchor cables were extended to the shore and connected with two dead-man anchors installed 80 m inland (Fig. 11).

Another cable was linked with the submarine pipe towing cable by way of the turn roller installed about 150 m on the inland side.

After the pipe tension was adjusted, the joined pipe was pulled by towing cable from the barge and laid at the sea bottom pointing in the direction of the turn roller. The towing was continued up to the position of pipe at the land, and the submarine pipeline was connected with the land pipeline.

Following this, conventional pipe laying was employed for a distance of about 200 m, up to the position where the offshore pipeline was already submerged, and lowered onto the ocean bed. Lowering the pipeline to the sea bottom was carried out by the same method as before. The pipe at the shore area was laid with proper overlapping for tie-in with the pipe of the offshore section.

Tie-in (Fig. 12) - The barge was positioned at the location of the tie-in while the necessary preparations for tie-in were completed. Of the six davits on the barge, three units were used for hoisting the offshore end section of the pipeline and the other three were used for hoisting the pipeline end of the coastal section. Hoisting pipe was executed in several stages while measuring the distance between the sea bottom and hoisted pipe.

After hoisting the pipelines to the final height, both ends were prepared for joining. Then they were aligned, welded and coated after confirmation of the weld by x-ray inspection. Upon completion of joining, the lines were lowered to the sea bottom in several stages.

CONCLUSIONS

Kokan Pioneer 1 was used for pipe laying in the Aga Development Project, the first full-scale pipeline construction project for developing Japan's continental shelves.

There was some initial apprehension concerning such a large-scale construction project, but after starting the work, such anxiety was found unnecessary and no major incidents occurred.

The major factors for the success were considered to be the following:

1. Favorable weather
2. Effective use of monitor television and equipment for measuring barge position
4. Very little trouble with equipment and devices. Proper arrangements for preparatory equipment and facilities.
5. Adequate preinspection of stingers, tensioners, etc. to prevent accidents.

In such construction work, even a very small accident might cause stoppage of the entire construction. Therefore, at the stage of completing plans, complete preparations and arrangements were required. The project afforded rich experience as well as valuable data.

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Kunitomi 23
Figure 11. Method of pipe laying in shore area
Figure 12. Sketch of tie-in
INTRODUCTION

There are several reported projects of offshore airport construction throughout the world. They may be categorized into the following four concepts based on the type of structure involved:

(a) Dike and polder concept  
(b) Land fill concept  
(c) Pile concept  
(d) Floating concept

From the technical point of view, concepts (a) and (b) are the most possible solutions and the construction cost is also less than the other concepts. However, these two concepts have some shortcomings, such as environmental destruction during removal and transportation of soil and water pollution by the land filling. Change of current due to the existence of new islands also raises problems after the completion of an airport. Further, in the land fill method, uneven settlement of ground might cause other future problems.

Although concepts (c) and (d) have some technical problems to be solved and the construction cost may be comparatively high, they have many advantages concerning environmental integrity.

The pile concept was adopted for runway extension at La Guardia Airport in New York in which platforms were supported by vertical piles as was the case in most off-shore airport construction projects. This method of application in the areas where water depth is large (and horizontal forces due to wave, wind, current and earthquake are big) has limitations. Their installation may be difficult due to the large size and tremendous number of piles required in these areas.

Proposed herein is a method to alleviate these shortcomings, using a combination of a jacket and piles to be a foundation unit.

PROPOSAL OF JACKET AND PILE CONCEPT

Structural types in pile concept are shown in Figure 1.

"Vertical Pile Method" has little resistance against horizontal load, and large sized piles are required. The number of piles closely driven may reduce the bearing resistance as a
groups of piles. In addition, it may be difficult to maintain the accuracy in driving the piles at the offshore site.

"Batter Pile Method" may be preferable in resisting horizontal forces, but it is difficult to drive big and long piles in a slanted position.

"Bracing Method" is the one in which vertical piles are connected by bracing members so that smaller piles can be applied than with the vertical pile method. However, there is difficulty with the construction since the bracing members must be installed in the water.

"Jacket and Pile Method" is proposed herein in which jackets are prefabricated in a shop, towed on the sea to the site, set at the prescribed position, fixed on the sea bed by piles driven through steel tubular columns of the jacket; thus, completing a foundation unit. After a number of these units are installed, their upper levels are connected to each other by beams. Over these units, a reinforced concrete slab is to be placed which is to be a deck of the airport. The jacket is to be a tubular space frame structure with sufficient strength against vertical and horizontal loads. It could be further reinforced by bracing members so as to compose a trussed structure when required. Since jackets are prefabricated in a shop, high accuracy can be expected, which eases the pile driving because the jacket works as a template.

AN OFFSHORE AIRPORT BY JACKET AND PILE METHOD

Figure 2 shows an artist's concept of an offshore airport by the jacket and pile method. The design conditions of this example are presumed as follows:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water depth</td>
<td>20 m</td>
</tr>
<tr>
<td>Wave height (max)</td>
<td>4.5 m</td>
</tr>
<tr>
<td>Wind speed (max)</td>
<td>50 m/s</td>
</tr>
<tr>
<td>Earthquake acceleration</td>
<td>0.25 g</td>
</tr>
<tr>
<td>Sea bed: Soft mud (up to - 50 m)</td>
<td></td>
</tr>
<tr>
<td>Sand (below - 50 m)</td>
<td></td>
</tr>
<tr>
<td>Runway</td>
<td>2 x 4,000 m</td>
</tr>
<tr>
<td>Take-off load (max)</td>
<td>750 t</td>
</tr>
</tbody>
</table>

A cross section of the runway area is shown in Figure 3. The modular size of the jacket is 30 m x 30 m and 27 m high. A jacket consists of tubular frame structure with 16 columns with 16 m spacing for the runway area where the big live load is to be subjected.

The tops of jackets are mutually connected by trussed girders with grillage of beams, on which a prefabricated deck plate is installed and concrete is placed after arrangement of reinforcing bars.

A construction procedure is shown in Figure 4. Application of a self-elevating platform (SEP) to installation of jackets and piles will be useful from the point of view of safety and accuracy of the construction work.

The problems to be considered in practical application of the jacket and pile method for an offshore airport construction are: (a) large capacity required for pile driver, (b) noise due to pile driving, and (c) corrosion to be protected by painting, lining and/or cathodic protection applying experiences in the present offshore structures.

CONCLUSION

The jacket construction method has already been applied in many fields such as offshore structures of oil production platforms, sea berthes, etc. The present proposal is an application of it to offshore airport construction, and its feasibility has already been assured by the present technology.
In case the integral application of jacket and pile method is difficult to an airport because of its construction cost, it would be possible to apply this method to the runway area only and the other parts could be constructed by other methods such as the land fill method.

The jacket and pile method will be applicable to offshore construction other than airport such as offshore atomic power plants, offshore cities, etc. The authors would like to acknowledge Kajima Corporation, Tokyo for their cordial cooperation in this study.

Figure 1. Pile concept
Figure 2. Artist's concept of offshore airport

Figure 3. Cross section of runway
Figure 4. Construction procedure
MODEC OIL SKIMMER (MIPOS)

I. Mutoh
Managing Director
Mitsui Ocean Development and Engineering Company, Ltd.
Japan

INTRODUCTION

In cooperation with Japan Ship's Machinery Development Association, MODEC has developed a practical and effective oil skimmer called MIPOS (MODEC Inclined Plane Oil Skimmer).

PRINCIPLE OF MIPOS

As shown in Figure 1, when MIPOS moves forward at 2 to 4 knots, the floating oil goes down along the inclined plane and goes up into the oil collecting well through a baffle plate at the well bottom. A false bottom plate is provided to increase the oil collecting efficiency.

CAPACITY OF OIL RECOVERY

The oil recovering capacity of running oil skimmer is shown as

\[ Q = 1.8 \times B \times t \times V \times C_1 \times C_2 \times C_3 \]

where,
- \( Q \): Capacity of oil recovery (m³/h)
- \( B \): Sweeping breadth (m)
- \( t \): Thickness of oil film (mm)
- \( V \): Running speed at oil skimming (knots)
- \( C_1 \): Efficiency of oil recovery
- \( C_2 \): Distribution rate of oil spill on the water
- \( C_3 \): Rate of actual oil skimming operation

Efficiency of oil recovery \( C_1 \) of MIPOS-10, for example, is shown in Figure 2, which has been obtained by a model test in an experimental tank. \( C_2 \) and \( C_3 \) depend on the floating conditions of spilled oil on the water and the operating condition of the oil skimmer. \( V-C_1 \) curve shows the efficiency at calm sea condition, and it will be reduced by 10 to 15 percent in waves.

As seen in Figure 2, maximum efficiency \( C_1 \) is found at about 2.4 knots. However, the maximum recovering capacity can be obtained at about 3 knots speed as shown by \( V-Q \) curve.

BUILDING RECORD AND OPERATION RESULTS OF MIPOS

The building record of MIPOS attaching their principal particulars is shown in Table 1.
TAMAMIDORI

Just after we delivered TAMAMIDORI, a calamity of a large oil spill occurred at Mizushima in the Inland Sea, on 18 December 1974. As the TAMAMIDORI was located near the oil spill center it went into clean-up operations (Fig. 2). From 21 December 1974 to 8 January 1975 it worked a total of 16 days and recovered the oil of 38.5 kl (about 10,000 gallons). Maximum recovered oil was 4 kl/day and mean recovered oil was 2.4 kl/day. After this actual oil skimming operation, the principle of MIPOS was quite convincing and satisfactory as per expectation by the model experiment. However, the oil spill of C-heavy oil was very viscous in cold temperature, and as TAMAMIDORI had been designed to clean-up small amounts of oil leakage in the shipyard basin, only a conventional hand pump was provided for oil transfer which was no use for such viscous oil and a dipper must be used instead of a pump (Fig. 4). As its oil storage tank capacity was also small, only 2.8 kl, the tank was full within a few hours. Finally, mean operating time was 2.7 hours for oil skimming, 2 hours for transferring the oil on land and 3.3 hours for sea going which was a very bad working rate (Fig. 5).

Pump Test

As it was recognized that a conventional pump was of no use for viscous oil, we tested a Mohno pump using the actual recovered Mizushima's spilled oil. By this test a Mohno pump was found to be very good for such viscous oil, even including various debris (Figs. 6 and 7).

MIPOS-S (Fig. 8)

During the oil spill calamity, we devised a small MIPOS (MIPOS-S) to reply to the urgent needs for an efficient and handy oil skimming system. MIPOS-S has no engine but is fitted with a simple debris collecting net. As it is small and light, it can be easily loaded onto a truck and transported to anywhere on land. After reaching shore, it was dropped on the water and bound to both sides of small coastal tankers or other available small mother boats on which small storage tanks and the Mohno pump are loaded.

The mother boat runs in 2 to 3 knots, embracing the MIPOS-S on both sides, then the sweeping breadth becomes so wide, and as the mother boat has plenty oil storage capacity, the rate of actual working time will be increased almost full day.

We built two MIPOS-S within 4 days and planned to go into actual operation, unfortunately, we missed the chance by bad sea conditions. However, it operated on the very thin oil film and could recover some oil and also it was proved that its maneuverability was not so bad.

EC-l (MIPOS-17) (Fig. 9)

The oil recovering capability of EC-l was tested on the water of Singapore in March, 1975 before its delivery. 110 l (30 gal) of red dyed linseed oil was spread on the water and recovered oil quantity was measured. At 2.4 knots speed, recovered oil ratio was 95.8 percent and at 3.8 knots the ratio was 41 percent, being attested by members of the Singapore Government.

EC-l is equipped with a debris collector. The oil skimming system is shown in Figure 10.

FEATURES OF MIPOS

(1) No moving part in the oil collecting system - no damage or trouble and no maintenance.
(2) Highly efficient - 95 percent floating oil can be recovered.
(3) Effective even in waves.
Any kind, viscosity and thickness of oil can be recovered.
Wide range of recoverable speed (2 to 4 knots).
Mohno pump can transfer any kind of oil including debris.
Various size and systems can be designed conforming to the oil skimming conditions.
(MIPOS-S, combination with oil boom, oil barge.)

Figure 1. Principle of MIPOS

Figure 2. Efficiency of oil recovery for MIPOS
### TABLE 1. Building Record of MIPOS.

<table>
<thead>
<tr>
<th>Item/Type</th>
<th>MIPOS-10</th>
<th>MIPOS-10</th>
<th>MIPOS-17</th>
<th>MIPOS-S6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Name</strong></td>
<td>MIDORI</td>
<td>TAMAMIDORI</td>
<td>EC-1</td>
<td>-</td>
</tr>
<tr>
<td><strong>Owner</strong></td>
<td>MSE MSE</td>
<td>Chiba Shipyard Tamano Shipyard</td>
<td>Maracca Strait Council (Singapore PSA)</td>
<td>MODEC</td>
</tr>
<tr>
<td><strong>L x B x D (m)</strong></td>
<td>9.0 x 3.5 x 1.3</td>
<td>9.0 x 3.5 x 1.3</td>
<td>17.0 x 6.0 x 2.5</td>
<td>6.0 x 2.5 x 1.5</td>
</tr>
<tr>
<td><strong>Oil recovering capacity (kl/h)</strong></td>
<td>29</td>
<td>29</td>
<td>30</td>
<td>87 (as unit 24)</td>
</tr>
<tr>
<td><strong>Oil storage tank capacity (kl)</strong></td>
<td>2.8</td>
<td>2.8</td>
<td>25</td>
<td>Depend on mother boat</td>
</tr>
<tr>
<td><strong>Gross tonnage</strong></td>
<td>9</td>
<td>9</td>
<td>52</td>
<td>-</td>
</tr>
<tr>
<td><strong>Main engine</strong></td>
<td>26 PS x 1</td>
<td>39 PS x 1</td>
<td>115 PS x 2</td>
<td>None</td>
</tr>
<tr>
<td><strong>Cruising speed (knots)</strong></td>
<td>4</td>
<td>5</td>
<td>7.3</td>
<td>Depend on mother boat</td>
</tr>
</tbody>
</table>

Note: *1 = Oil recovering capacity is calculated as skimming speed \( V=3 \) knots, oil thickness \( t=2 \) mm, \( C_1=0.9, C_2=C_3=1.0 \).

*2 = Calculated as mother boat breadth is 7 m, total sweeping breadth is 12 m.

*3 = Outboard engine can be fitted on the unit.
Figure 3. Actual oil skimming by TAMAMIDORI (MIPOD-10)

Figure 4. Collected oil in the coll. well

Figure 5. Recovered oil
Figure 6. Suction test of Mohno pump

Figure 7. Suction test of Mohno pump

Figure 8. MIPOS-S

Figure 1308
Figure 9. EC-1

Figure 10. Total system of oil skimming
UNDERWATER OBSERVATION VEHICLE (EYE ROBOT)

I. Mutoh
Managing Director
Mitsui Ocean Development
and Engineering Company, Ltd.
Japan

INTRODUCTION

An underwater observation vehicle has been newly developed by Mitsui Ocean Development and Engineering Company in cooperation with NAC (Japanese Optical System Enterprise) and Japan Ship's Machinery Development Association. The development started in April, 1973 and was finished in July, 1975. The main purpose of the development was to make a prototype of an unmanned underwater observation vehicle in which a color TV is installed to acquire the underwater scene more easily, efficiently, and safely than the conventional system.

DESIGN REQUIREMENTS

Vehicle

<table>
<thead>
<tr>
<th>Type</th>
<th>Unmanned, tethered remotely controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation Depth</td>
<td>Max 100 m</td>
</tr>
<tr>
<td>Maneuvering radius</td>
<td>100 m at 100 m depth</td>
</tr>
<tr>
<td>Maneuvering mode</td>
<td>Fore and aft, turn, ascent and descent</td>
</tr>
</tbody>
</table>

Optical system

<table>
<thead>
<tr>
<th>TV camera</th>
<th>Color TV camera</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual range</td>
<td>Horizontal ±90°</td>
</tr>
<tr>
<td></td>
<td>Vertical ±40°</td>
</tr>
</tbody>
</table>

Environment

| Current                    | Max 2 knots                            |

TOTAL SYSTEM

The operating configuration of this system and the total system are shown in Figures 1 and 2.

The vehicle itself has slight positive buoyancy in the water, and suitable float buoys are fitted on the tether cable.

CONSTRUCTION OF VEHICLE (Fig. 3)

<table>
<thead>
<tr>
<th>Length overall</th>
<th>2.55 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>1.89 m</td>
</tr>
<tr>
<td>Height</td>
<td>1.50 m</td>
</tr>
<tr>
<td>Weight</td>
<td>900 kg</td>
</tr>
</tbody>
</table>
Figure 1. Operating configuration
Figure 2. Total system block diagram
Figure 3. Observation vehicle
The inner pressure hull consists of a welded steel sphere (6 mm thick, 1,050 mm diameter) and a transparent acrylic plastic hemisphere (24 mm thick, 700 mm diameter). The outer hull is shaped in a stream-lined body and has Y type tail wings to reduce the resistance in the water and to keep the stable motion. The forward part is made of transparent acrylic plastic, and the other part is made of FRP. The guard rail on which illumination lights are fitted, legs for landing and lifting eye plate etc. are provided (Fig. 4).

**MOTION CONTROL SYSTEM**

All motions of the vehicle are controlled by two thrusters fitted on both sides of the vehicle. The thruster has a reversible propeller in Kort nozzle, and is driven by a proportionally controlled oil-filled A.C. motor. The thruster can be tilted ±90° in a vertical plane. Revolution and tilting motions of the thruster are controlled manually and indicated on an operation console.

Thruster: 2 sets  
Motor: A.C. 440 V x 60 Hz x 3 Ph x 6 Pole  
Rated output: 1.5 kw x 60 kg thrust  
RPM: 200 - 1,100  
Propeller: 3 blades x 250 mm diameter  

**Tilting unit:**  
Tilting range ±90°  
Tilting speed 6 deg/sec

**OPTICAL SYSTEM**

A color TV camera is installed in the pressure hull. Pan and tilt, zoom, focus and iris of the camera can be controlled remotely (Fig. 5). Pan and tilt of the camera is accomplished by a specially designed mirror system fitted in front of the lens, instead of moving the camera itself. Using these special pan and tilt devices, power can be saved and also the influence of the camera's motion on vehicle's motion can be minimized. Turning center of the mirror is located at the center of acrylic hemisphere.

Two color monitors and V.T.R. are built in the operation console. Eight to five hundred W underwater lights are fitted on the guard rail.

**NAVIGATION AND COMMUNICATION SYSTEM**

During the operation, the following items are indicated on operation console so as to make the operation sure and safe (Fig. 6).  
1. Vehicle's depth in the water  
2. Vehicle's direction (by magnet compass)  
3. Vehicle's speed against water (by speed sensor)  
4. Vehicle's horizontal position relative to support vessel (by acoustic transponder system)  
5. Vehicle's trim angle  
6. Thruster's revolution of propeller  
7. Thruster's tilt angle  
8. Alarm for water leakage in the pressure hull

**CABLE AND ITS HANDLING SYSTEM**

The tethered cable consists of two coaxial cables for TV, two shielded pair cables for communication, 16 power leads for thrusters, thruster tilting unit, U/W lights and others, and armoured with stainless steel strand jacket.
Figure 4. General arrangement
Figure 5. TV camera

Figure 6. Operator console

Figure 7. Cable and winch
To reduce the resistance of the cable in the water, plastic tapes are fastened to the cable. The cable is handled by an electric winch which has a shock absorber and two slip rings for power and signal.

Cable: 36.8 mm diameter x 200 m length  
Cable winch: 1.0 ton x 9/18 m/min spool speed (Fig. 7)

TEST

Operation tests were carried out in May and July 1975 at 40 m and 100 m depth of water, and could get good performance results as per design and much useful knowledge for its future modification and development.
Concentration of population and development of industrial activities has enlarged and excessively over-populated the cities. Wastes are increasing considerably, in variety as well as in quantity, because of the diversified industrial activities and rise of living standards, involving labor-saving patterns of daily life.

Waste treatment is now becoming one of the most serious urban problems due to unsatisfactory treatment methods of wastes and difficulty of obtaining necessary installation sites.

Most refuse and wastes are dumped at land reclamation sites, land fills or incinerators. However, because of the disruptive influence on environments, and public outcries for building-up waste treatment plants, it has become quite impossible to obtain necessary plant sites in major cities and their vicinities.

To overcome these bottlenecks and solve this problem by establishing new disposal facilities at offshore sites is now proposed. This will also be a solution to the difficulty of obtaining onshore disposal sites.

This conception is not only aimed at intensive disposal of wastes on a large scale, but also to find a method for their effective reutilization. For undertaking the design and study of "Offshore Waste Treatment Technology and Systems", the following four major sub-themes are taken up (for specializing in each one of these topics, a division committee has been established to achieve the conceptional design of the individual system).

1. Total system
2. Wastes generation source, and wastes collection and transportation system
3. Offshore waste disposal plant (or facilities)
4. Offshore structure

In establishing intensive offshore waste disposal facilities, it is quite necessary to assess the influence on the environments of the areas of sea and sea-side cities surrounding the offshore plant sites. Therefore, another committee has been established to carry out assessment of the influence on natural conditions ecologically and physically. (These two committees will study both the natural and social conditions which offshore waste disposal facilities have to satisfy.)
DESIGN AND STUDY OF OFFSHORE WASTE DISPOSAL SYSTEM

Preface

Concentration of population and industrial facilities in seaside cities has been accelerating. Those cities have become excessively densely-populated, which has resulted in traffic congestion and environmental pollution, as far as waste treatment is concerned. This trend is particularly so in metropolitan areas surrounded by satellite cities, where wastes are increasing considerably in quantity, and variety, accelerated by vigorous economical activities and rise of living standards. Thus, it becomes impossible to rely on the present means of treating wastes, due to difficulty for obtaining necessary sites and funds. This trend is aggravated every year. Under the circumstances, problems in treatment and disposal of the metropolitan areas, considered as typical model areas in Japan, were classified as four subjects to be reviewed from various points of view.

Total system In this subject, we reviewed the background and concept of offshore waste treatment, necessity of resources recovery from wastes, administrative problems in realizing waste disposal at sea, and others.

Wastes generation source and waste collection and transporting system This subject includes, investigation and prospect for the present and future plans for treatment of wastes from generation to disposal, and study of the multiple integrated system (totalled system) of wastes transportation such as pneumatic transport, railroad transport, and barge transport which are expected to be realized in the near future.

Wastes disposal plant In this subject, in relation to a waste disposal plant and resources recycling plant, information concerning the techniques now in service and the extent of utilization of recycled resources at present were studied. Based on "the development of wastes recycling technology", which is now being undertaken by the Institute of Industrial Technology, the conceived design was made of the disposal plant having a capacity of 8,000 to 10,000 ton per day.

Offshore structures In this subject, three types of offshore structures; pile system, bottom support, and pontoon system were selected for concept designing of installing a waste disposal plant of 10,000 ton per day at the Bay of Tokyo to dispose of the wastes discharged in metropolitan areas. Additionally, concept designing for self-propelled types was undertaken to install a disposal plant of 600 ton per day, which will play a role of the separated and parallel disposal plant site to connect this disposal plant at sea (offshore plant) with the local cities distant from this offshore facility.

Selection of a suitable site for an offshore treatment plant and its economy will be finalized through study and evaluation for the efficient use of onshore sites, reduction of costs for preventing the site from pollution, and resources recovery, in addition to consideration to forms of a subsystem.

TOTAL SYSTEM

Background and Conception of Offshore Waste Disposal System

During recent years, wastes, which are generated by human activities of production and daily living, have been considerably increased due to vigorous industrialization and the extraordinary rise of living standards. These wastes have now completely surpassed the capacity of the conventional treatment or disposal system where we have been relying exclusively on natural recycle or incineration, to the extent of numerous pollutants disrupting the environments.
Such environmental disruption was generated due to the country's narrow space of land. An excessive concentration of population into urban areas accompanying an inevitable increase of wastes, has strengthened regulations of wastes treatment originating from; intensified interest of the residents in environments boosting of land prices, and deterioration of the municipal environment.

It has now become quite difficult to find a suitable onshore disposal site, and in this situation, it is now high time for us to earnestly concentrate on realizing the offshore waste disposal system aimed for diversification of plant sites, rather than on making endeavors to obtain onshore plant sites. In addition to such a situation, in face of the "Resources Nationalism" or "Oil Crisis" by the oil producing countries brought about since the latter part of last year (1974), limitation of natural resources was seriously recognized by the nation and reconsideration is now being given to the conventional way of living which had become so liberal in the consumption of natural resources.

Doubled by these situations, an idea of recovering useful things from waste to reutilize them has been intensively recognized by the people as an efficient means of waste disposal, different from the conventional and simple concept of dumping or incineration. Various projects oriented for saving natural resources in wastes disposal are being undertaken, strongly, on a nation-wide scale.

Offshore waste treatment system has the following features:

1. Diversification of sites is enabled.

   In solving solid wastes disposal problems, the most important problem is where to install the disposal plant. For such a narrow country as Japan, to have a disposal site at sea is highly recommendable as one of the promising solutions to the difficulty for obtaining onshore sites.

2. Intensive and large-scale treatment of wastes is enabled.

   It is quite desirable to collect wastes to one site from wide-spread areas and to treat them intensively on a larger scale from the viewpoint of achieving economy and protecting the environment.

   These purposes can be realized and accomplished by this system. On the other hand, in achieving resources recovery from wastes, the quality of recovered materials can be made more reliable by large-scale treatment of wastes and the market value of the recovered materials can be improved by its stable supply of constant quantities.

3. Flexible selection of facilities enabled.

   Offshore facilities make it possible to have a flexible selection on equipment arrangement. Also, more flexibility is permissible in remodeling, improving and changing the treatment process.

   On the contrary, because of the offshore facility, vibration, winds, waves and corrosives must be considered.

4. Ease of disposal and transportation of finally treated wastes (residues).

   It is easy to transport and dispose of the residues after the recovery of useful materials, when wastes are treated and recycled.
Ease of evaluation of influence (drawback) on the environment.

Different from onshore facilities, offshore facilities involve less social factors which are difficult to evaluate, and its evaluation, if any, can be made very easily.

WASTE GENERATION AND WASTE COLLECTION AND TRANSPORTATION SYSTEM

Present State of Municipal Waste and Future Prospects

Municipal refuse increased at a considerable pace since the beginning of early 1955 and it should be a markable record increase of more than five times for the decade between 1960 and 1970.

The character of municipal refuse has a tendency to become more diversified and complicated, especially, an increase of mixture of plastics into municipal wastes is highly remarkable. Thus, the calory of normal municipal wastes is estimated to be 800 Kcal to 1,000 Kcal/kg, (Kilo calory per kilogram). However, in recent years they have come to have 4,500 Kcal to 11,000 Kcal per kg, which has generated an air pollution problem from the incinerator and actually lowered its incineration capacity.

In this situation, it is an urgent matter to try to achieve less entry or mixture of plastics into ordinary municipal wastes, through an aggressive public campaign and guidance for keeping separate collection and promoting higher recovery of plastics for reutilization.

In addition to this, bulky wastes such as household furniture and electrical appliances are on the increase every year and on a nation-wide scale, such wastes occupy about 10 percent of the municipal wastes.

Waste disposal cost is increasing at an accelerated pace due to the aforementioned increase of wastes, not only in quantity but also in variety and complexity, traffic congestion, farther remote treatment and disposal sites, and hikes of personnel expenses. During the eight years from 1965 to 1972, the total treatment cost increased two and one-half times. Among the wastes treatment cost, 70 to 80 percent is collection and transportation expenses, and 80 percent of these expenses is occupied by personnel expenses.

It can be said that the waste-treatment business is a typical labor-intensive enterprise, requiring a number of men in collection and transportation. This is why the rationalization of collection and transportation is strongly advocated.

Waste Intended for Offshore Treatment

The decision was made to install an offshore treatment facility within the Bay of Tokyo for treatment of wastes generated in six seaside metropolitan cities. This area was selected, because it presents a condition which makes it relatively easy to establish such an offshore facility.

As for the quantity of refuse to be handled by this offshore treatment facility, the future quantity of collected refuse of these six cities was estimated on the basis of the data from 1965 to 1973, and among which wastes, the undermentioned quantity of refuse will be treated (Table 1).

Kinds of refuse to be brought to this offshore treatment plant were classified in types of ordinary household wastes; garbage, domestic refuse, separated refuse and bulky wastes.
TABLE 1. Future quantity of collected refuse

<table>
<thead>
<tr>
<th></th>
<th>1980</th>
<th>1985</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max wastes to be</td>
<td>9,949 ton/day</td>
<td>12,432 ton/day</td>
</tr>
<tr>
<td>supplied to offshore</td>
<td></td>
<td></td>
</tr>
<tr>
<td>plant per day</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average wastes to be</td>
<td>7,979 ton/day</td>
<td>10,088 ton/day</td>
</tr>
<tr>
<td>supplied to offshore</td>
<td></td>
<td></td>
</tr>
<tr>
<td>plant per day</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The composition of those wastes was evaluated on the assumption that only the ordinary household refuse, that exceed the capacity of incineration plants, will be brought into the offshore plant, while separated and bulky wastes will all be brought into this plant.

The following is the fundamental design criteria for the offshore treatment plant, viewed from collection and transportation.

(1) Scale

<table>
<thead>
<tr>
<th>Actual transported quantity</th>
<th>1980</th>
<th>1985</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8,000 ton/day</td>
<td>10,000 ton/day</td>
</tr>
<tr>
<td>Peak quantity</td>
<td>20% up</td>
<td></td>
</tr>
</tbody>
</table>

(2) Wastes to be treated

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary wastes</td>
<td>50%</td>
</tr>
<tr>
<td>Separated wastes</td>
<td>37%</td>
</tr>
<tr>
<td>Bulky wastes</td>
<td>13%</td>
</tr>
</tbody>
</table>

(3) Origin of Wastes Transportation

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tokyo</td>
<td>90%</td>
</tr>
<tr>
<td>Others</td>
<td>10%</td>
</tr>
</tbody>
</table>

(4) Transportation distance at sea: 10 to 15 km

(5) Methods (or systems) of transportation

a. barge.....offshore treatment plant
b. railway.....barge.....offshore treatment plant
c. vehicles (wastes collection trucks, garbage trucks, etc.).....barge.....offshore treatment plant (wastes to be transported mainly in containers of 8 m x 8 m x 20 m)
d. pipeline.....offshore treatment plant

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Noguchi and Ishida
OFFSHORE TREATMENT PLANT

Investigation of the Present Techniques Made Available by Manufacturers and Prospect of the Future Techniques to Come

As mentioned above, the required treatment capacity of the offshore plant of the future was evaluated to be 8,000 to 10,000 ton/day, and under this prospect, conception designing was decided to be undertaken.

For this purpose the present techniques were investigated, as well as those in development of the domestic manufacturers in Japan, which can satisfy the condition that the treatment plant should be composed of 2,000 ton/day module units, and be highly feasible in the near future.

Study of Offshore Waste Treatment Plant

(1) Present reutilization of by-producing substances from wastes.

A. Present state and level of reutilization of Recovered Energy:

In Japan, the first energy-recovery type plant was established at the Nishi-Yodogawa Factory of Osaka City, in 1965, which was constructed with the unique idea of supplying the excessive electricity generated by the factory in return to Kansai Electric Service Company.

However, this example was not followed due to the public's underdeveloped sense of social environment. Nevertheless, recent drastic variations in the economical and social environments necessitated the need for resources saving and energy saving even in waste treatment. Under this background, the people are more enthusiastic in accomplishing the recovery of energy and heat in the establishment of the waste treatment plant as realized in such cases as Tokyo Katsushika Plant, having a battery system (electric supply system) supplying to Tokyo Electric Service Company, etc., in Sapporo City where heat and energy is supplied by the plant to the municipal corporation of energy supply servicing.

B. Present state and level of reutilization of Recovered Materials (substances):

i. Papers

That proportion which papers occupy in municipal wastes is 33.2 percent, highest among the compositions (constituents) of the wastes. Recovery methods for papers are still old-fashioned, labor-intensive types. The only plant that can be put into immediate service is the Wet Systems developed by Blacklawson Corporation with Ishikawajima Harima Heavy Industries Company Ltd, (IHI) has a license agreement in Japan.

There are only a few plants now under development in Japan, with the exception of the semi-wet type separation and crushing techniques by the Institute of Industrial Technology.

ii. Plastics

Among the generation sources of plastic wastes, households have the highest proportion of more than 50 percent, and plastics percentage has reached 22 percent of the separately collected wastes in their composition.
The techniques which are available now in practical use are the simple type of reproduction and incineration. Future techniques being considered are; thermal dissolving, oil recovery techniques and those techniques oriented for developing new applications for recovered plastics (such as for asphalt reinforcing materials, adhesives, soil improvers, etc.).

iii. Aluminium

There are no established techniques currently available for separating aluminium from municipal refuse.

iv. Glasses, iron, compost, etc.

About 60 percent of the waste glasses are generated from households, and recovery has been practiced to a very high extent. The rate of the recovered glasses is more than 30 percent of the annually produced glasses.

Compost is being again considered as an effective means of solving the hazard brought about due to our country's fertilizer-intensive farming; to feed sufficient amount of organic fertilizers to soils for prevention of the soil acidation and for recovery of the reproducible capacity of farming soils.

(2) Conception of the Ideal Plant.

The basic conception of waste treatment is to accomplish volume reduction economically and the ideal pattern to accomplish volume reduction exists in complete recycling and reutilization of wastes.

The patterns of recycling which are considered to be ideal for main constituent materials composing the municipal wastes are listed as follows:

a) garbage.....recycled to earth.....composting
b) papers.....recycled to substances for papers.....utilization of recycled papers.....recycled to earth.....composting
c) plastics.....recycled to substances for plastics.....pelletized.....recycled to substances (materials).....oil separation
d) glasses.....recycled to substances.....reutilized as reproduced glass wares

A most important problem which should be discussed in this area is the purity of the recycled materials. Either more accurate separate collection techniques than those now in existence, or more sophisticated techniques that can enable the economical and automated separation to meet the requirements of the above mentioned recycling pattern are required.

There are many problems to solve. Therefore, incineration, energy recovery through thermal dissolving, sanitization or non-hazardization of inorganic materials and land-filling techniques can be important and useful as the means for volume reduction for the present.

As an effective utilization of by-product resources, energy recovery and materials recovery can be considered. However, in the present situation, no concrete proposal, as to the effective use of them has been submitted and it seems some time ahead before the recovery and reutilization of recovered materials can be put into practical use.

Thus, for the time being, reutilization of recovered heat energy, which is considered to be established as stable techniques, will play a major role.
OFFSHORE STRUCTURES

Conditions for Installation

Natural conditions in relation to the installation of the offshore structures have been studied in the Bay of Tokyo. Design criteria, or required conditions for offshore structures, were established taking into consideration that the daily handling capacity of the offshore plant will be 10,000 ton/day in 1985.

A different installation site can be available in the Bay of Tokyo for three types of offshore structures, piled bottom-support and pontoon, according to specific features that each type has embodied, and natural conditions necessary for concept designing were established upon investigation of the marine climatographical factors.

The required condition for the treating quantity of wastes were established as follows (for three types other than a self-propelled barge, on which concept designing was decided to be based):

<table>
<thead>
<tr>
<th>Condition</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>wastes handling capacity</td>
<td>10,000 ton/day</td>
</tr>
<tr>
<td>deck area of structures</td>
<td>75,000 m²</td>
</tr>
<tr>
<td>overhead fixed load</td>
<td>35,000 ton</td>
</tr>
<tr>
<td>overhead variable load</td>
<td>185,000 ton</td>
</tr>
<tr>
<td>wastes storage pit</td>
<td>100,000 m³ (for three days)</td>
</tr>
<tr>
<td>by-product resources pit</td>
<td>20,000 m²</td>
</tr>
<tr>
<td>upper deck height from w.l.</td>
<td>± 8 m</td>
</tr>
</tbody>
</table>

Pontoon Type Offshore Structures

Pontoon type offshore structures are floating structures of steel weldment box type, and are anchored at a fixed position by use of a large number of chains and anchors. Two methods of installing a waste treatment plant in plane arrangement and in multi-stage arrangement were studied:

(1) Horizontal Arrangement System

This system is designed to arrange five modules of the treatment plant of 2,000 ton/day on the pontoon decks, which were conceived to be sized 300 m x 200 m. The pontoon has a depth of 12 m, having about a 340,000 ton capacity of displacement at a full-loaded condition.

(2) Multi-Stage Arrangement

This system is designed to make such arrangements that three modules of 2,000 ton/day plants are installed on the decks and another two modules under decks, the sizes of which are conceived to be 250 m x 200 m. The pontoon has a depth 14 m, having about a 300,000 ton capacity of displacement at a full-loaded condition.

(3) Results of Rough Investigation

In designing such a large-scale pontoon, its anchoring and movement in storm waves are of great concern and concept design was undertaken for these problems.

(a) Anchoring - ordinary outer forces to be applied on the pontoon such as wind pressure resistance, tidal current resistance, wave current force are summed up to be 800 to 1,000 ton.
(b) Movement

As a result of studies made on the movement of the pontoon under the condition of wave height 3.5 m and wave cycle 7 sec, it was found that horizontal movement would be generated by 2.7 to 4.5 m. Pitching and rolling angles are quite small, because the length of wave, are about one-third of the length of pontoon, resulting in the range of about 0.04 to 0.07.

From the result of the conception designing for a large-scale pontoon, the conclusion was that such a scale of floating type structures at an offshore site is quite feasible for construction and mooring anchor from the technical point of view, and there will be no major problems in installing a waste treatment plant.
GOVERNMENTAL ADMINISTRATIONS
ON PRESERVATION OF OCEAN ENVIRONMENT IN JAPAN

Taisuke Samethima
Bureau of Ports and Harbors
Ministry of Transport
Japan

PRESENT STATUS OF MARINE POLLUTION

Japan is surrounded by sea and the coast line is very long (29,000 km). The sea is very close and important to the people and has been used for many purposes, such as sea transportation, marine recreation (e.g., fishing and sailing), catching marine products, etc. In the area of sea transportation, approximately 2.8 thousand million tons of cargo was handled in Japanese ports in 1973 and this supports the livelihood of the people and the industrial activities in Japan.

Approximately 0.5 thousand million tons of imported crude oil and refined oil products have also been transported by sea. Assuming that they are transported by 5,000 D.W.T. oil tankers in average, approximate 0.2 million oil tankers navigate in Japanese waters every year.

As the results of these high and large-scale utilizations of sea, marine physical and ecological conditions have been unstable and the marine pollution is becoming outstanding in forms of water deterioration due to increase of oily content in the water, floating oil, floating refuge, red tide, etc.

Table 1 shows the occurrence frequency of marine pollutions which were observed by the Marine Safety Agency in Japanese waters. It shows that oily pollutions occurred 1985 times and pollutions due to other than oil 381 times including 175 times red tide in 1974. Marine pollutions were concentrated in Tokyo Bay, Ise Bay, and Seto Inland Sea where big cities and large scaled waterfront industrial areas are located along their coastal line. These regions showed about 63 percent of total occurrence through the whole country.

About half of these marine pollutions were due mainly to human causes such as illegal discharge of waste and oil from ships or careless valve operations of oil tanks, etc. The accident which occurred 18 December 1974, in the Mitsubishi Oil Company, Mizushima Refinery Industry was one of the most serious marine pollutions. At first, heavy oil began to spill out from the No. 270 oil tank and when the crack was made on the wall of the tank, contained heavy oil gushed out. The tank capacity was 50,000 kl and approximately 37,000 kl heavy oil was stored at that time. Total volume of oil which flowed out to the Seto Inland Sea through Port Mizushima was estimated from 7,500 to 9,500 kl. This oil was spread over the sea 100 km towards the east and 30 km towards the west, that is, one third of the Seto Island Sea was contaminated. Damage to the fishing industry was estimated at approximately 50 million dollars.
### TABLE 1. Occurrence frequency of marine pollution by sea area

<table>
<thead>
<tr>
<th>Year</th>
<th>Sea Area</th>
<th>Hokkaido Adjacent Sea</th>
<th>East Side of Honsyu Adjacent Sea</th>
<th>Tokyo Bay</th>
<th>Ise Bay</th>
<th>Osaka Bay</th>
<th>Seto Inland Sea</th>
<th>South Side of Honsyu Adjacent Sea</th>
<th>Kyushu Adjacent Sea</th>
<th>Mihonkai Sea</th>
<th>South East Sea Area</th>
<th>Total</th>
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<tr>
<td></td>
<td>Oil Pollution</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
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<td></td>
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<td>11 (3)</td>
<td>49 (47)</td>
<td>39 (28)</td>
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<td>284 (47)</td>
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<td>269</td>
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<td>72</td>
<td>104</td>
<td>93</td>
<td>137 (1,985)</td>
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<tr>
<td></td>
<td>142 (2)</td>
<td>224 (2)</td>
<td>258 (3)</td>
<td>234 (36)</td>
<td>322 (44)</td>
<td>672 (69)</td>
<td>85 (8)</td>
<td>145</td>
<td>126</td>
<td>158 (2,366)</td>
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<td></td>
</tr>
</tbody>
</table>

**Note:** Numbers in parenthesis means the occurrence frequency of red tide
With regard to waste discharge other than oil, 514 million tons of city waste such as excrement etc., 28.9 million tons of industrial waste such as waste acid, waste alkali, etc. and 82.7 million tons of dredged spoil were reported to the Commandant of the Maritime Safety Agency under the Marine Pollution Prevention Law by the owners of ships who discharged waste into the sea.

To create new land for various utilizations, land reclamation by filling up has been undertaken and amounted to 36,000 hectares for ten years (from 1954 to 1975).

**JAPANESE LAWS ON MARINE POLLUTION PREVENTION**

As to marine pollution prevention, it is prescribed in the Basic Law for Environmental pollution Control, Water Pollution Control Law, Marine Pollution Prevention Law, Port Regulation Law, etc. The principle articles are extracted and shown as follows.

**Basic law for environmental pollution control**

In article 9, it is prescribed that the government should establish the standard to protect men's health and to preserve the life environment and that the government should take the effective administrative activities to achieve the standard. For the sea area the standards to be maintained are prescribed in "Environmental Standards for Man's Health (as to heavy metals such as cadmium, mercury, lead, etc., contained in the water)" and "Standards for Preservation of Life Environment (as to COD, PH, etc. contained in the water)".

In articles 19 and 20, the central and local governments are obliged to establish the environmental pollution control program and to take necessary administrative action on pollution prevention for the particular regions.

**Water pollution control law**

In article 3, it is prescribed that the central and local governments have to establish the discharging standard for drainage from factory to public water area. According to article 15, the local government should observe the public water area and make an effort to prevent water pollution there.

**Marine pollution prevention law**

In articles 4, 10 and 18, it is prohibited to discharge oil and waste from ships and offshore facilities except for:

1) that discharge oil or waste observes a certain standard or the ship is in an emergency to discharge them for her safety, or

2) the discharge of dredged material to be made in accordance with the established standard for the method of discharge into the place to be reclaimed under license by the Public Water Reclamation Law.

With regard to ports' areas and within 10 km of ports' limits, ships are prohibited from dumping oil residue and other waste by the Port and Harbor Law and The Port Regulations Law. And it is stated that the Ministry of Transport may recommend to the port authority to construct necessary waste oil disposal facilities and also subsidize five-tenths of the construction cost.

**Port and Harbor Law**

The central government and port authorities are to undertake the following port and harbor works to improve and preserve the port environment. The central government may provide a subsidy within the limit of budget to these works undertaken by the port authority.
When these works are necessitated by a particular person, or give noticeable benefit to private factories located in the port or waterfront area, the port authority may cause them to bear some part of the cost of these works.

1) Public Pollution Control Works to remove sea bed deposit containing toxic substances or organic substances or to clean contaminated waters.

2) Port and Harbor Environmental Protection Works, to construct the waste disposal facilities such as dikes for waste dumping area, waste receiving facilities, beaches, greens, etc.

3) Marine Environment Protection Works to remove floating materials such as oil and waste from the sea.

**MARINE POLLUTION CONTROL**

**Ocean Monitoring and Regulations**

Maritime Safety Agency patrols with patrol boats and aircrafts to control marine pollution and is strengthening monitoring techniques by adopting a ultra-red ray night monitoring system of oil spilling, video tape recorder for public pollution monitoring, etc.

As to regulations to restrict discharge of waste into the sea, water pollution control laws and regulations are established by local situations. It obliges concerned persons to keep the oil boom for preventing diffusion of spilled oil and to dump industrial waste at the restricted sea area.

**Prevention of Marine Pollution**

Waste oil disposal facilities to dispose oily ballast water and bilge have been positively constructed to prevent oily pollution and now their capacity is 60 to 70 million cubic meters per year for approximately 90 facilities.

On the other hand, it is recommended that ships should have the load-on-top system and separate ballast tank form oil tank to eliminate the oily ballast water and for the bilge they should have oil-water separating facilities and they should have disposal facilities to treat all waste within the ship itself.

**Marine Environment Protection Works**

**Port environment protection works**

Sludge accumulated in the port containing toxic substances (mercury, PCB, etc.) or organic substances is dredged to clean ports such as Tokyo, Yokkaichi, Omuta, etc. For instance, in the 1974 financial year, 30 million dollars was used for 16 ports. Nowadays, for sludge removal, the standards of mercury, and PCB are prescribed (concerning mercury there is a standard calculation method for concentration of sludge to be removed, concerning PCB, sludge which contains more than 10 ppm PCB in dry base should be removed). The standards of the sludge to be removed, which contains organic substances and oil, are being examined.

The dikes for waste dumping disposal areas not only for port originated waste but also industrial waste, general city originated waste, etc., and waste incinerators which dispose of marine waste, were constructed. In the 1974 financial year, it cost about 35 million dollars for 17 ports, such as Tokyo, Osaka, etc.

Regarding the cleaning of ports, debris collecting ships have been constructed for 23 ports, such as Chiba, Yokkaichi, etc. In the 1974 financial year, it cost one million dollars.

1332 Samethima 4
Marine environment protection works

In order to collect floating waste and floating oil, the Ministry of Transport has three debris collecting ships and three oil recovery ships. When the above-mentioned accident in Mizushima happened, one of oil recovery ships took an active part in collecting oil spread over the sea.

In addition to these, for the purpose of marine environment protection, protection of fishing areas and coastal environment, the researches on geographical distribution of pollutants and effective techniques for removal of them are being carried out, not only in the governmental organization but also in private firms.
APPLICATION OF DIELECTRIC RADAR REFLECTOR (LENSREF)

John T. Takarabe
Tokyo Keiki Company, Ltd.
Japan

INTRODUCTION

At the UJNR 5th Marine Facilities Panel held in Tokyo, 21 and 22 October 1974, the writer introduced the subject equipment. This has been followed by a number of inquiries from various quarters. As the range of its application has been extended, the writer would like to report on its present status.

APPLICATION TO ANCHOR BUOY

Yokohama Rubber Company, Ltd. (Japan) has developed a pneumatic rubber buoy with a subsidy from the Japan Ship Machinery Development Association. This company has also started manufacturing a larger type, "Globuoy", to be used for anchoring drilling boats engaged in offshore oil extraction. This latter type buoy is now being supplied to many countries in the world including the U.S.A., England, Holland, and Norway, as well as Japan.

This "Globuoy", at the specific request of one of the large petroleum firms in the U.S.A., has the subject Lensref incorporated inside for the purpose of making radar detection easier. It is reported that several tens of "Globuoy" with Lensref are currently in actual use in the field.

There were, of course, some questions to be considered before using Lensref in the buoy, e.g., the question of its height above the surface not being adequate because it has to be incorporated into the buoy approximately 5.9 ft in diameter (1,800 mm in ø) or certain attenuation caused because it has to be placed in the thick layer of rubber, etc. However, more current testing has proved that there was nothing wrong and application has been quite successful and effective. As the result of an excellent evaluation, the project is now proceeding, in several places, to insert Lensref by cutting open the buoy already delivered or in actual use.

"Globuoy" is being used at an increasing rate because of its greater safety, durability and reduced drifting, as well as its superior radar reflecting/identification capability.

APPLICATION TO UNLIGHTED ICE BUOY

In the U.S.S.R. a national specification is set up for the above referred buoy, the All Soviet National Specification 19466-74. It was established on 28 January 1974 and will stay effective up to 1 January 1980 (from 1 January 1975).

It specifies that a radar reflector must be installed 2 m above the surface. This is perhaps because it is going to be used on the north-east passage in the northern part of...
Siberia. We have been informed that our Lensref, delivered through the Soviet Embassy at Tokyo, is now undergoing an evaluation to determine its endurance capabilities in arctic environments in the U.S.S.R.

In view of the expected future requirements for navigational aids (installations adapted not only to the north-east and north-west passages but also to the polar passage) we are also contemplating the possible use of our Lensref to such installations. The Lensref is, of course, an entirely passive device and requires no maintenance.

The Ship Research Institute of Japanese Ministry of Transportation has just completed testing of our Lensref and how it should work in extremely low temperatures. It has been confirmed that the Lensref functions normally without any trouble even in such low temperature as -60°C. The successful result is likely to expand further the application range of Lensref.

APPLICATION TO SIGNALS ON THE RHINE RIVER

As the signals on the Rhine River, a corner reflector is installed on both sides at intervals of several tens of meters and ships are sailing, marking the picture which appears in the scope of river radar. Maintenance of the corner reflector is being carried out punctually and periodically. To make all such installation more powerful and reliable, we are engaged in negotiation talks with those concerned in Holland, concerning the possibility of adding our Lensref to this installation.

APPLICATION TO BUOY ON THE AUSTRALIAN SEAS

A project discussion concerning installation of Lensref-equipped buoys - on the gulf zone facing Whyalla and Port of Lincoln and also on the reef zone surrounding Queensland - is being held by the Australian Maritime Service Board (and joined by the Department of Transport and local harbor masters). Evaluation of sample Lensref already forwarded is proceeding, with successful results. In addition, those concerned are contemplating the possibility of using Lensref with the newly adopted plastic buoys. Testing for this purpose is being continued in various parts of Australia using a dozen Lensref which have been additionally purchased.

THE PROBLEM OF MALACCA-SINGAPORE STRAIT

It is our sincere desire, to maintain the safety of sea-traffic of Malacca-Singapore Strait and Lombok-Makassar Strait. Discussions concerning a separate scheme plan and its facilities are now being held among the officials of the governments concerned. As a link in the chain of this project, experiments with the "Lensref" are now being carried out at the above places. The "Lensref" is installed in the Sarus Tower located outside the port of Singapore (1°16'N 103°46'E) and at the TG. Perok (6°5'S 106°50'E) in Djakarta.

In order to determine if good performance is obtainable under the environmental conditions directly under the equator, as well as the super-cold test, a 750 H run of the environmental test has just been completed at the Ship Research Institute.

TECHNICAL COMMITTEE OF RADAR REFLECTOR

The Japan Association for Preventing Marine Accidents has organized a technical committee to review the radar reflector (passive and active) and the comparative test on the performance of the related product such as "Lensref", Conner Reflector, Radar Reflect Sheet and Transponder is being carried out. The results of this committee are expected within this year. The experimental test indicates that satisfactory data on necessary features for long-distance should be acquired by the "Lensref" rather than the combined use of Conner reflector consisting of eight planes capable of peak radar cross section 50 m².
KINDS OF DIELECTRIC LENS REFLECTOR

There are two principles of Dielectric Lens Reflector, i.e., Luneburg Lens utilizing both the reflection and refraction, the other being the Eaton-Lipman Lens utilizing only the refraction.

<table>
<thead>
<tr>
<th>Dielectric Constant</th>
<th>Dielectric Constant</th>
<th>Number of Layers</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Surface)</td>
<td>(Center)</td>
<td></td>
</tr>
<tr>
<td>Luneburg</td>
<td>1</td>
<td>9</td>
</tr>
<tr>
<td>Eaton-Lipman</td>
<td>1</td>
<td>20</td>
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</table>

Both the experimental and performance tests of the Lens Reflector have now been completed and in view of practical use. We believe the "Lensref", which utilizes the principle of Luneburg lens, will be used as before.

AN EXAMPLE OF LENSREF APPLICATION

The Globuoy, manufactured by Yokohama Rubber Company, Ltd., is incorporated inside Lensref. Figure 1 shows that sea trial is operating for the purpose of making radar detection.

The attached drawings (Figs. 2 through 8) show a Lensref and two Lensref's installed inside.

Figure 1. Sea trial operating for the purposes of radar detection
Figure 2.
Figure 3.
Figure 4. Rays in a Luneburg lens

\[ \varepsilon = 2 - r^2 \]

\( \varepsilon \): Dielectric constant

\( r \): Normalized radius
Figure 5. Ray paths inside an Eaton Lippmann lens

\[ \varepsilon = \frac{2 - r}{r} \]

\( \varepsilon \): Dielectric constant
\( r \): Normalized radius
Figure 6. Construction and theory of the OMNI-AZIMUTH radar reflector
THE SARUS TOWER STORY

Figure 7. The Sarus Tower story
What is a Sarus Tower?

A Sarus Tower, as designed by Nautical Service (Aust) Pty. Ltd., is basically a vertical cylindrical metal structure, incorporating a buoyancy chamber, which is joined by a universal coupling to a sea bed deadweight anchor. The articulated link provides the required elasticity which enables the tower to bow under excessive stress without loss of security or function.

Originally conceived as a navigation light tower for use in areas where buoys and beacons were either uneconomical or impractical, the Sarus Tower—of which there is a variety of types—relies on controlled rigidity which is induced by buoyancy.

The deadweight base, which is of special design and construction according to the type of tower used, absorbs imposed forces through the universal coupling and thus the tower maintains exceptional stability.

Heel of the Sarus Tower

The heel of the Sarus Tower causing possible loss of visible range of the light is compensated by a gimbled arrangement which will allow the tower to heel up to 10° with the light remaining in the horizontal plane with no loss of visible distance. Engineering tests are now in progress to fit a revolving light for greater visible range. Radar and transponders can also be fitted.

The area of seabed that is occupied is minimal and the tower location is always positive. Again the tower has very high survival criteria and can withstand winds of 200 knots. A 75 ft. lower sited in 60-65 feet of water will heel not more than twelve degrees in a sixty knot wind.

Sarus Towers as Channel Markers

Two typical methods used for the demarkation of a channel are, first, the land-based installation of beacons which, of necessity, must be located close to the edge of the channel or, second, the placing of marker buoys suitably anchored in the channel itself.

Both of these methods have disadvantages which are effectively illustrated in diagrams 1-4 on the facing page.

Where buoys are concerned, underwater turbulence caused by passing ships can eventually erode the sub-marine sides of the channel causing partial or total collapse with a resultant need for either continual maintenance or complete replacement and re-siting.

Marker buoys are effective up to a point but changes in sea levels coupled, as is usually the case, with the incidence of currents can cause buoys to drift away from their original, carefully selected, positions and thus the precise location and dimension of the channel are no longer recognisably definite.

The use of either method can place the pilot in an invidious position where he has to rely on intelligent guesswork in order not to hazard the ship.

The main advantage of the Sarus Tower as a marker is that, once it has been positioned, it is virtually immovable by natural forces or mechanical turbulence and can safely be relied upon to continue its functions accurately and automatically for an indefinite period with minimal maintenance.

It is relevant that, for example, a Sarus Tower installed off Port Hedland in Western Australia two years ago has, on examination been found not to require maintenance and not to have moved its position, although being subjected to a number of severe cyclones.

It is a proven fact, therefore, that a channel marked by Sarus Towers stays clearly and accurately marked even in severely adverse weather conditions and eliminates the potential risks inherent in other systems.

Figure 8. Sarus Tower
CONSTRUCTION, TOWAGE AND INSTALLATION
OF THE FLOATING CITY "AQUAPOLIS"

Masayuki Tamehiro
Mitsubishi Heavy Industries, Ltd.
Japan

INTRODUCTION

The project for the International Ocean Exposition Okinawa, in 1975 has been carried forward with "The Sea we would like to see" as its basic theme. The Aquapolis was planned by the Japanese Government as an intensive representative of the theme and has been completed as such by the combined efforts of Japanese manufacturers.

The Aquapolis, a semi-submersible offshore structure of the largest class in the world, is oriented to the floating city of the future, and gives significant suggestions to the industrial circles who utilize the space of the sea - one of the potential capacities of the sea, namely space, resources and energy.

To embody the theme, a joint organization with experts in the related industrial fields was established. Their ideas and subsequent report for the embodiment of the theme on the basis of the basic concept is as follows:

1) The Aquapolis should be befitting as a symbol of the International Ocean Exposition.
2) It should be the essence of the modern technology.
3) It should be a perfect embodiment of harmony with environment.

The water area surrounding the Aquapolis constitutes a beautiful natural environment where tropical fishes and corals grow, although from September until November, it is in the path of typhoons. Moreover, a large number of visitors are expected during the exposition period. From these viewpoints serious consideration has also been given to the security and safety of the structure and the prevention of environmental pollution. As this exposition is an international one, strict observance of the delivery term was another important subject.

The Aquopolis, a special offshore structure, also has characteristics of a land structure in construction and functions. For this reason, a technical standard has been established for its construction by the three ministries concerned, i.e. the International Trade and Industry Ministry, the Transportation Ministry, and the Construction Ministry. On the other hand, the "Semi-Submersible Exhibit Ship Standard" has been set recently by the Nippon Kaiji Kyokai. In constructing the Aquapolis, the manufacturers applied or referred to a wide range of rules and regulations for ships, buildings, environmental pollution prevention and aircraft radio waves which are now in force.
AN OUTLINE OF THE DESIGN AQUAPOLIS

Features of Aquapolis

The following are the specific features of the Aquapolis:

1) In view of towing operations from the mainland of Japan to Okinawa, a lower hull-type with little towing resistance in the selection of semi-submersibles was adopted.

2) In order to secure structural advantage and stability, adopted a 4-lower hull-type of semi-submersible so as to get enough breadth of the structure was adopted.

3) Structural strength is an essential condition for the safety of visitors and workers on the structure. Therefore, in the structural design of the Aquapolis, several trial designs were conducted with computer programs parallel with the study of the data on various kinds of oceanic structures, and confirmed the safety of the Aquapolis by various kinds of structural tests. (May 1973*)

4) The seaworthiness of the structure in the states in which external forces such as waves and wind are working on it has an important effect on the pleasant feeling of the people on it. From this viewpoint thorough consideration has been paid to its motion against extended forces. It has been confirmed by water tank test (May 1973*) and wind tunnel test (October 1973*).

5) Like offshore mobile drilling units, the Aquapolis adopts the closed system for its various facilities avoiding the use of land facilities. It is equipped with various facilities including a power plant, a sewage disposal facility and distiller equipment.

General Arrangement

As shown in the general arrangement drawings (Figs. 1 through 5), the Aquapolis is a cubic trussed rahmen structure composed of four lower hulls, columns, braces, main, middle, and upper decks.

The shape of each lower hull is designed so as to possess enough buoyancy to bear the total weight of the structure and also to reduce wave resistance at the time of towing. Provided in the lower hulls are ballast tanks, pump rooms, fuel oil tanks, fresh water tanks and a sewage disposal room.

Between each lower hull and the main deck 16 columns are installed. These columns not only serve as the main structural members to support the upper structure of the Aquapolis but also play the role of maintaining sufficient stability in semi-submerged condition.

A horizontal brace is fitted between each column, providing stiffness to the horizontal plane of the lower structure. It is also combined with a diagonal brace, making the whole structure a rigid framework.

Provided under the main deck are, a passage leading from the bridge laid across the sea to the main deck, an underwater observation room, and an escalator tube.

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*Mitsubishi Heavy Industries, Ltd.
"The investigation before construction of offshore structures 'AQUAPOLIS' at International Ocean Exposition Okinawa"

<table>
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<th>Test Type</th>
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<td>Water tank test</td>
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<tr>
<td>Wind tunnel test</td>
<td>October, 1973</td>
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</table>
Figure 1. Diagrams for basic construction and arrangement
Figure 2. Plan of upper deck

Figure 3. Plan of middle deck

Figure 4. Plan of main deck

Figure 5. Overall cross section
The space on the main deck is used primarily for exhibition, demonstration and services for general visitors. In the large space in the center there are; a marinorama projection room, a marinorama room, an algae forest and a passage intended for enabling the visitors to experience underwater walks along with a model of the Aquapolis, an information booth, a panorama of an oceanic pasture, resting room and a telecasting booth. The office and dining room, cooking room, medical treatment room for the employees and a machinery compartment are also arranged here. In the fore part of the deck there is a pier which is large enough to allow simultaneous access of two ferries with a 100 ton capacity each in a semi-submerged state of 20-m draft.

The middle deck is used for control and operation. An exhibits demonstration control room, a computer room, a special VIP room, a utility control room and living quarters for the employees are provided in this space.

The upper deck is a multi-purpose open space and has an area of 100m x 100m including the overhanged fences on four sides. The central part of the upper deck is used for a helicopter deck.

Principal Particulars and Equipment

The principal dimensions and equipment of the Aquapolis are shown in Tables 1 and 2 respectively.

### TABLE 1. Principal Dimension

<table>
<thead>
<tr>
<th>ITEMS</th>
<th>DIMENSIONS(m)</th>
<th>CONDITION</th>
<th>Draft(m)</th>
<th>Displacement(t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overwhole</td>
<td>104(L) x 100(B) x 32(D)</td>
<td>Towing</td>
<td>5.4</td>
<td>17,350</td>
</tr>
<tr>
<td>Large lower hull</td>
<td>2-104(L) x 10(B) x 6(D)</td>
<td>Floating</td>
<td>5.8</td>
<td>18,600</td>
</tr>
<tr>
<td>Small lower hull</td>
<td>2-56(L) x 10(B) x 6(D)</td>
<td>Semi-submerge</td>
<td>20.0</td>
<td>28,070</td>
</tr>
<tr>
<td>Column</td>
<td>12-7.5$, 4-3.0$</td>
<td>Storm</td>
<td>15.5 - 12.5</td>
<td>24,950-23,210</td>
</tr>
<tr>
<td>Brace</td>
<td>10-3.0$, 16-1.8$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fuel</td>
<td>2 x 352m³</td>
<td></td>
<td>704m³</td>
<td></td>
</tr>
<tr>
<td>Potable water</td>
<td>2 x 293</td>
<td></td>
<td>586</td>
<td></td>
</tr>
<tr>
<td>Sanitary water</td>
<td>1 x 45</td>
<td></td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>Sewage</td>
<td>1 x 13</td>
<td></td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Sewage storage</td>
<td>1 x 116</td>
<td></td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>Sludge</td>
<td>1 x 21</td>
<td></td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>Sanitary</td>
<td>1 x 12</td>
<td></td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Ballast</td>
<td>33 x ___</td>
<td></td>
<td>16,511</td>
<td></td>
</tr>
<tr>
<td>Upper Dk</td>
<td>7,400 m²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle Dk</td>
<td>2,500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Main Dk</td>
<td>5,800</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Housing capacity is 2,000 persons for ordinary time (2,400 max.)

Accommodations: 20 living rooms and 41 beds.
**TABLE 2. Principal equipment**

<table>
<thead>
<tr>
<th>ITEMS</th>
<th>EQUIPMENT</th>
<th>AMOUNTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mooring equipment</td>
<td>Windlass</td>
<td>8-80/40/20t x 2/4.5/9m/min.</td>
</tr>
<tr>
<td></td>
<td>Anchor chain</td>
<td>16-76 mm φ x 350 m</td>
</tr>
<tr>
<td></td>
<td>Anchor</td>
<td>4-15 t</td>
</tr>
<tr>
<td>Ship access equipment</td>
<td>Two ferry boats can be accessed simultaneously</td>
<td></td>
</tr>
<tr>
<td>Safety equipment</td>
<td>Shooter, rescue boats, etc.</td>
<td></td>
</tr>
<tr>
<td>Transport equipment</td>
<td>Escalators</td>
<td>1-30m/min. x 5,500 P/H(down)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/16m/min. x 1,900 P/H (up)</td>
</tr>
<tr>
<td></td>
<td>Moving belt</td>
<td>1-16m/min. x 990 P/H</td>
</tr>
<tr>
<td></td>
<td>Elevator</td>
<td>4-260 kg x 20.5m/min.</td>
</tr>
<tr>
<td></td>
<td>Heliport</td>
<td>1/22m x 35m</td>
</tr>
<tr>
<td>Environment preservation equipment</td>
<td>Sewage treatment</td>
<td>1-60 m³/day BOD: 10 ppm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-30 BOD: 10</td>
</tr>
<tr>
<td></td>
<td>Sludge incinerator</td>
<td>1-150 l/h</td>
</tr>
<tr>
<td></td>
<td>Waste incinerator</td>
<td>1-360 kg/h</td>
</tr>
<tr>
<td></td>
<td>Oily water separator</td>
<td>1-10 m³/h</td>
</tr>
<tr>
<td>Lifting equipment</td>
<td>Elec. hoist</td>
<td>2-5t x 8 m/min. x 25m</td>
</tr>
<tr>
<td></td>
<td>Air hoist</td>
<td>1-0.5 x 12  x 18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-1 x 6.5 x 23</td>
</tr>
<tr>
<td>Air conditioner</td>
<td>Turbo-type</td>
<td>1-250 RT (1RT=3,320 Kcal/h)</td>
</tr>
<tr>
<td></td>
<td>Recipro-type</td>
<td>1-100 RT, 1-30 RT</td>
</tr>
<tr>
<td>Distiller equipment</td>
<td>Vapor compression type</td>
<td>1-66 m³/day</td>
</tr>
<tr>
<td>Electric generator</td>
<td>Main Engine</td>
<td>2-1, 800 PS x 720 RPM</td>
</tr>
<tr>
<td></td>
<td>Main Generator</td>
<td>2-1, 500 KVA x 450V x 60HZ</td>
</tr>
<tr>
<td></td>
<td>Emerg. engine</td>
<td>1-400 PS x 900 RPM</td>
</tr>
<tr>
<td></td>
<td>Emerg. generator</td>
<td>1-312.5 KVA x 450V x 60HZ</td>
</tr>
<tr>
<td>Pump equipment</td>
<td>Ballast pump</td>
<td>4-500 m³/h, 4-350 m³/h</td>
</tr>
<tr>
<td></td>
<td>Sea water pump etc.</td>
<td>2-250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2-200</td>
</tr>
</tbody>
</table>

**Steel materials**

The Aquapolis adopts all-welded cubic trussed rahmen structures. For its main structural parts, steel materials conforming to the Regulations of the Nippon Kaiji Kyokai are used within the allowable stress stipulated in the Regulations.

For the main girders and braces where especially high internal stress accrue, high tensile steel of the 50 kg/mm² class is used, and for parts where complicated stress works such as the connection parts, E Class steel having a high ductility is used.
Protective coating

The protective coating used include; epoxy resin paint, tar epoxy resin paint, chlorinated paint and alkyd resin paint for ship use. All of these have superior uni-corrosive and erosive characteristics, and are selected case by case.

As for the cathodic protection system, the aluminum anodes having 50 m amp/m² current density on wetted surface under 20-m draft were adopted, and their life expectancy along bottoms and sides of lower hulls is 10 years, and 5 years for the others, respectively.

Basic characteristics

Mobility

The Aquapolis, which is a non-self-propelled offshore structure, moves on the sea with the help of tugboats. However, in a calm sea area with a water depth less than 50 m, its position can be shifted up to about 200 m by operating the windlass on the Aquapolis.

Floating and submerging performance

Shift from semi-submerged state (draft: 20 m) to floating state (draft: 5.8 m) or vice versa can be attained within about 3.5 hr.

Static stability

The large GM, larger than 10 m in all conditions, and MTC (Table 3) secure the small inclination of the structure at 0.6° even in the event of 2,400 people leaning to one side. Synchronization with waves is not conceivable as its natural period is 18 to 25 seconds. Its stability ratio is 7 times in a semi-submerged state and 1.4 times in stormy weather (Tables 3 and 4; Fig. 6).

Dynamic stability

The dynamic stability of the Aquapolis in waves was presumed from the results of the water tank test in irregular waves and denoted in Table 4.

Mooring characteristic

The Aquapolis is moored with 16 lines of the mooring buoy system composed of permanent anchors, mooring buoys and anchor chains. The mooring buoys play effective roles of preventing the abrasion caused by the friction of the chains with the sea bed and for reducing the impact force working on the anchors generated by the oscillation of the Aquapolis.

The mooring arrangement is safe, as the lines have a safety factor of 1.75 against the breaking strength of the chain even if one of them is broken during stormy weather (Fig. 7).

Threshold to operation

The worst weather and sea conditions as designated are as follows:

- Maximum instantaneous wind velocity: 80 m/sec
- Maximum wave height: 15 m
- Tidal Current: 1.5 kt
<table>
<thead>
<tr>
<th>Item</th>
<th>Symbol</th>
<th>Semi-submerged condition</th>
<th>Semi-submerged storm condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(Opening to the public)</td>
<td>(Opening to the public)</td>
</tr>
<tr>
<td>Displacement</td>
<td>w (ton)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Draft</td>
<td>d (m)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trim &amp; Heel</td>
<td>th (m)</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Water Plane Area</td>
<td>Aw (m²)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ton cm Immersion</td>
<td>TPC (t/cm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ctr. of Floatation</td>
<td>XF (m)</td>
<td></td>
<td>0.76(A) - 0.04(P)</td>
</tr>
<tr>
<td>Ctr. of Buoyancy</td>
<td>XB (m)</td>
<td></td>
<td>0.13(A) - 0.10(P)</td>
</tr>
<tr>
<td>Vert. C. of Buoy.</td>
<td>KB (m)</td>
<td></td>
<td>6.22</td>
</tr>
<tr>
<td>Center of Gravity</td>
<td>EG (m)</td>
<td></td>
<td>0.13(A) - 0.10(P)</td>
</tr>
<tr>
<td>Vert. C. of Grav.</td>
<td>KG (m)</td>
<td></td>
<td>11.15</td>
</tr>
<tr>
<td>Dist. from B to G</td>
<td>BG (m)</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Meta. Ht. from K</td>
<td>KM (m)</td>
<td></td>
<td>23.98</td>
</tr>
<tr>
<td>Meta. Ht. from G</td>
<td>GM (m)</td>
<td></td>
<td>12.83</td>
</tr>
<tr>
<td>Free Surf. Corr.</td>
<td>GC₀ (m)</td>
<td></td>
<td>0.67</td>
</tr>
<tr>
<td>Actual GM</td>
<td>G₀N (m)</td>
<td></td>
<td>12.16</td>
</tr>
<tr>
<td>Mt. for 1 cm trim</td>
<td>MTC (tm/cm)</td>
<td></td>
<td>40.64</td>
</tr>
<tr>
<td>Heave</td>
<td>Th (sec)</td>
<td></td>
<td>18.3</td>
</tr>
<tr>
<td>Natural Pitch</td>
<td>Tp (sec)</td>
<td></td>
<td>22.1</td>
</tr>
<tr>
<td>Period</td>
<td>Tr (sec)</td>
<td></td>
<td>22.9</td>
</tr>
</tbody>
</table>

Remarks:
1) KG, KB, KM measured from bottom most of Lower hull
2) t, h, MTC, based on outer column center
TABLE 4. Wave height and motion

<table>
<thead>
<tr>
<th></th>
<th>Lower hull floating</th>
<th>Semi-submerged condition (opening to the public)</th>
<th>Semi-submerged storm condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Significance</td>
<td>Max.</td>
<td>Significance</td>
</tr>
<tr>
<td>Wave height</td>
<td>1 m</td>
<td>2.2 m</td>
<td>4.4 m</td>
</tr>
<tr>
<td>Heaving</td>
<td>0.1 m</td>
<td>0.22 m</td>
<td>0.13 m</td>
</tr>
<tr>
<td>Surging</td>
<td>0.18 m</td>
<td>0.4 m</td>
<td>0.21 m</td>
</tr>
<tr>
<td>Pitching</td>
<td>0.5°</td>
<td>1.1°</td>
<td>0.3°</td>
</tr>
</tbody>
</table>

Ebb and flow: 3 m
Sea water temperature: 32°C
Atmospheric temperature: -10° to 40°C
Humidity: 85%

The threshold of permitting the access of the bridge laid across the sea to the Aquapolis for receiving visitors is set at 15 m/sec for the maximum wind velocity and 1.5 m for the maximum wave height in order to secure their safety.

Permanent Anchor

Through an on-the-spot investigation it was discovered that the sea bed under the Aquapolis is a limestone stratum covered with coral sands, remains and drift sands, therefore, a permanent anchor system (design securing power: 250 tons) was adopted. First, two permanent anchors were driven into the sea bed to estimate their holding power in full size. After confirming their proof stress and making necessary technical improvement, the remaining 14 permanent anchors were installed.

CONSTRUCTION

Subassembly

As the latest construction techniques were needed to build the Aquapolis, participation was requested of as many Japanese enterprises as possible, thereby contributing to the progress of the ocean development techniques of our country. In accordance with this plan, the block construction system was adopted. The main structural part was constructed by six large shipbuilding companies (Mitsubishi Heavy Industries, Hitachi Shipbuilding and Engineering Company, Ltd., Ishikawajima-Harima Heavy Industries Company, Ltd., Kawasaki Heavy Industries, Ltd., Nippon Kokan K. K. and Sumitomo Shipbuilding and Machinery Company, Ltd.) and the interior finish works were executed with the participation of two construction companies (Shimizu Construction Company, Ltd. and Takenaka Komuten Company, Ltd.).

Compared with the conventional offshore structures, the Aquapolis is very large in scale, the body was divided and the prior fitting-out system was adopted. That is, many out-fittings were installed on each subassembly block in construction or fabrication stages. Small blocks to be mounted were designed to be less than 300 tons and large subassembly blocks were designed to be less than 3,000 tons.

In order to attain high accuracy and uniform quality of the subassembly blocks, each of the above-mentioned shipbuilding companies determined the specifications of the blocks.
Semi-submerged condition (opening to the public)

\[ d = 20 \text{ m} \]

Wind force at \( \theta_1 = 0^\circ \)
\[ V_1 = 39 \text{ \text{kt}} = 20 \text{ \text{m/sec}} \]
\[ R_1 = 30 \text{ \text{t}} \]
\[ M = 2110 \text{ \text{t-m}} \]
\[ D_w = 0.08 \text{ \text{m}} \]
\[ \frac{(A+B)}{(B+C)} = 7.20 > 1.30 \]

Figure 6. Static stability curve (longitudinal)

Semi-submerged storm condition

\[ d = 12.5 \text{ m} \]

Wind force at \( \theta_1 = 0^\circ \text{C} \)
\[ V_1 = 117 \text{ \text{kt}} = 60 \text{ \text{m/sec}} \]
\[ R_1 = 411 \text{ \text{t}} \]
\[ M = 22,070 \text{ \text{t-m}} \]
\[ D_w = 0.95 \text{ \text{m}} \]
\[ \frac{(A+B)}{(B+C)} = 1.42 > 1.30 \]
Figure 7. The condition on the sea
with minute attention and built them under a severe quality control system while under­
going the inspection of the Transportation Ministry and Nippon Kaiji Kyokai.

Assembly

Each subassembly block separately constructed by the above-mentioned shipbuilding compan­ies was brought into the Hiroshima Shipbuilding and Engineering Works of Mitsubishi Heavy Industries for overall assembly.

As the existing dock was unavailable for the construction of the large scale Aquapolis, a
large wet dock (140 x 170 x 9.5m) was specially constructed. The blocks and machinery
were loaded both from the land and sea sides by collective loading methods employing a
300-ton floating crane barge and a 3,000-ton floating crane barge for large subassembly
blocks. This resulted in a saving of labor charges and a reduction of construction per­
iod.

For the overall assembly of a large offshore structure like this Aquapolis, the following
methods are conceivable, namely:

1) Using plural launching ways like a ship assembly.
2) Assembling each block in afloat condition.
3) Assembling them in a dry dock.
4) Assembling them on a floating pontoon installed horizontally.

As the main body of the Aquapolis is very large, a phased building method was adopted,
that is, in order to assure the high connecting accuracy of each joint of the subassembly,
blocks were loaded by the floating crane barges. The lower hulls were placed on the racks
in the wet dock and the subassembly blocks were assembled upward from lower hull to upper
deck (Figs. 8 through 13).

PROCEDURES

The assembling procedures in
wet dock are explained below.

1) Arrangement of the Aqua­
polis in the dock (Fig. 8)

2) Rack of lower hull (Fig.
9) The lower hulls were
placed on the racks in
the wet dock. These
racks were provided to
get the good workmanship
of the workers on the
lower hull decks and to
keep ample clearance
above the water surface
so the welded joints of
the columns might not be
exposed to the splashes
of sea water during ass­
embling operations.

Figure 8. Arrangement of Lower Hull in Wet Dock
Figure 9. Rack of lower hull

Figure 10. Setting of lower hull
3) Setting the lower hulls (Fig. 10)
   a) The racks were installed at the seating positions of the lower hulls just below the columns.
   b) Each lower hull towed to the Hiroshima Shipyard and Engine Works of Mitsubishi Heavy Industries by each shipbuilding company was filled with ballast water, and its position was adjusted so that its horizontal plane would become exactly horizontal. Its draft was kept shallow in consideration of the low tidal level at the time of seating.
   c) The seating positions of the lower hulls were marked on the dock side beforehand. The lower hulls were towed to the seating positions with these marks as targets and by use of measuring instruments, put on the positions and fixed with wire from the dock side.
   d) The lower hulls were installed on the predetermined positions accurately at the time the tide began to flow back.
   e) The ballast tanks in the lower hulls were filled with ballast water at the time when the tide began to rise. It was considered that due to the partial buoyancy of the void space, the eccentric working stress might accrue.
   To avoid this excess buoyancy of the structure, i.e., PL, CPL and SL lower hulls, a balancing weight was fitted on the deck of each.
   f) Seating of the four hulls was carried out in the order of PL, CPL, CSL and SL.

Figure 11. Fabrication of column and brace
4) Fabrication of columns and braces (Fig. 11)
   a) The columns other than PC-1 and PC-3 and braces were loaded and installed after the setting of the lower hulls was completed.
   b) The columns and braces were loaded by use of the 300-ton floating crane.
   c) In loading each column, the lowest and the successive upper part of column up to 20 m in height were made by one brock with bracket pieces of braces.
   d) The braces for the port side and starboard side only, were installed.
   e) Installation of columns and braces of stage c) and d), were done in the order of port side to starboard side, columns, horizontal braces, and vertical braces and so on.
   f) Between upper and lower part column blocks, 4 connection pieces were provided around the circumference of columns for temporary support. Necessary adjustment for securing accuracy was made at the same time.

Figure 12. Fabrication of side main deck and upper deck
5) Fabrication of Main and Upper Deck side blocks (Fig. 12)
   a) The main decks and upper decks were loaded after the columns and braces were installed.
   b) For easier loading by the 300-ton floating crane, installation of these main and upper deck blocks were done in the order of port side to starboard side, central parts to aftward and forward parts respectively. The upper deck blocks were divided into four parts and installed after the main deck construction was completed.
   c) Large-sized equipment (i.e., windlass) was also loaded in this stage.

6) Fabrication of Center parts
   After the completion of the fabrication of the main deck and upper deck side construction, loading on the central part was carried out in the order of the number as shown in Figure 13.
Launching and Sea Trial

The Aquapolis was drawn out from the wet dock and launched at high tide early in the morning on 25 February 1975. Then it was moored to the anchorage in the Hiroshima Bay and underwent overall finishing work.

After the completion of this work, it was subjected to a series of performance tests in the bay including inclining experiment, floating and submerging test, windlass operating test and watertight door performance test, and its safety was confirmed by comparing the initial design values.

TOWAGE AND INSTALLATION

Preparation for Towing

Prior to towing, the Aquapolis was given thorough protection against towing and thereafter underwent the severe inspection of the Nobel Company. It was provided with strong and perfect towing ropes; the exhibits and demonstrative presentation areas for EXPO '75 were also securely fastened to prevent their overturn by the oscillation of the Aquapolis in towing condition (period: 10 sec, angle of oscillation 20°).

Towage

The Aquapolis is designed to be towed with a floating draft ($d = 5.4$ m), but it is also designed to be towable in semi-submerged state with a storm resisting draft ($d = 12.5$ to $15.5$ m) by filling the lower hulls with water in case of stormy waves with a maximum wind velocity of 20 m/s and a maximum wave height of over 5 m are anticipated. Otherwise, it can take refuge in a port and reduce the angle of oscillation to prevent the overturn of the exhibits, etc.

The horsepower required for the group of tugboats was designed on the basis of the results of the water tank test (1973*) on towing resistance (Fig. 15) and the curve of the increased resistance in waves.

The towage of the Aquapolis was started at 9:00 am on April 18th and it arrived at the Exposition Headquarters in Okinawa early in the morning on the 23rd of April.

The tugboat group shown in Figure 16 was composed of one main tugboat (7,200 IHP) and two auxiliary tugboats (each 5,000 IHP) (total 17,400 IHP). The group cruised on the calm sea from the Hiroshima Bay to Okinawa, about 570 nautical miles in distance, at the rate of about 5.5 kt.

The total horsepower of the tugboats was a little big for the Aquapolis compared to the result of experiments as shown in Figure 15. Nevertheless, tugboats with suitable capacities were not obtainable in time and the added power was desirable to avoid accidents.

The Aquapolis was manned by a total of 36 people including those in charge of draft adjustment and temporary mooring in stormy weather during towage. As the sea was calm during the period of towage and moreover, its dynamic stability is quite well (as shown in Figure 14) and the angle of the moment of equilibrium is very small, there was no practical difficulty throughout the period of towage.

*Water Tank Test: May 1973
Wind force at $\theta_1 = 0^\circ$
$V_1 = 39 \text{ kt} = 20 \text{ m/sec}$
$R_1 = 61 \text{ tons}$
$M = 2800 \text{ t-m}$
$D_w = 0.16$
$(A+B)/(B+C) = 17.0 > 1.3$

Wind force at $\theta_t = 0^\circ$
$V_1 = 39 \text{ kt} = 20 \text{ m/sec}$
$R_1 = 58 \text{ ton}$
$M = 2250 \text{ t-m}$
$D_w = 0.13 \text{ m}$
$(A+B)/(B+C) = 24.5 > 1.3$

Figure 14. Stability curve on towing condition
Various data on the Aquapolis in tow
   1) Properties of stability (Table 5)
   2) Stability curve (Fig. 14)
   3) Towing resistance (Fig. 15)
   4) Arrangement of tugboats (Fig. 16)

Installation of Permanent Anchors and Mooring Buoys

Prior to the arrival of the Aquapolis, 16 permanent anchors and mooring buoys were installed at the appointed positions at the sea bed with a depth of about 40 m under the water surface. The mooring buoys were kept afloat on the sea water in the style as shown in Figure 17 so that they could be fastened to the Aquapolis on arrival.

In order to confirm the safety of the anchors in stormy weather, they were subjected to a tension of 230 tons (design tension = 250 tons). They were proven to have a sufficient holding power for the Aquapolis.

![Figure 15. Towing resistance curve](image)

Remarks

1 Significant wave height: 3.1 m + wind velocity: 20 m/s
2 Significant wave height: 3.1 m + wind velocity: 10 m/s
3 Significant wave height: 3.1 m
4 In calm water
A Towing mean speed of full size unit
B Towing max. speed of full size unit
on the Inland Sea

3000 HP

3000 HP

3000 HP

5200 HP

7200 HP

5000 HP

800 HP

800 HP

800 HP

on the open sea

before installation

2400 HP

2400 HP

2400 HP

2400 HP

2400 HP

2400 HP

Figure 16. Arrangement of tug boats
TABLE 5. Properties of stability

<table>
<thead>
<tr>
<th>Item</th>
<th>Symbol</th>
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<th>Trans.</th>
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<tr>
<td>Displacement</td>
<td>w (ton)</td>
<td>17,350</td>
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<tr>
<td>Draft</td>
<td>d (m)</td>
<td>5.4</td>
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<tr>
<td>Trim &amp; Heel</td>
<td>th (m)</td>
<td>0.50(A)</td>
<td>0</td>
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<tr>
<td>Water Plane Area</td>
<td>Aw (m²)</td>
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<td>Ton cm Immersion</td>
<td>TPC (t/cm)</td>
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<tr>
<td>Ctr. of Floatation</td>
<td>XF (m)</td>
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<td>Ctr. of Buoyancy</td>
<td>XB (m)</td>
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<td>Center of Gravity</td>
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<td>Meta. Ht. from G</td>
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<td>Period Roll</td>
<td>Tr (sec)</td>
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Remarks:

1) KG, KB, KM measured from bottom most of Lower hull
2) t, h, MTC, based on outer column center
Before the entry of the Aquapolis into its mooring point, the tugboats were redistributed as shown in Figure 18, that is, the main tugboat (7,200 IHP) and one auxiliary tugboat (5,200 IHP) were distributed before and behind the Aquapolis respectively, and a total of three smaller tugboats (2,400 IHP each) were placed on both sides. The Aquapolis entered the mooring point at a very slow speed in tow of the five tugboats. As there was only one admission passage to the mooring point (coral reefs were projecting on other areas) the Aquapolis was towed under heavy guard of several escort ships and one large tugboat (5,000 IHP) which was standing by ready for emergency.

At a point about 200 m from the mooring point, two 15-ton anchors, for temporary mooring, (Fig. 19) were driven. When the Aquapolis arrived at the appointed point, two anchors (Nos. 2 and 7) were driven from a 80-ton anchoring boat equipped with a side thruster as shown in Figure 20. The position of the Aquapolis was maintained by the tugboats throughout the driving operation in close contact with one another.

Then, two anchoring boats connected the chains drawn out from the Aquapolis with the mooring buoys of the permanent anchor line simultaneously (Fig. 17).
Figure 18. Tug boat and buoy arrangement before installation
A total of 13 anchors were driven by this method on the first day and the remaining 3 were driven on the next day. Thereafter, the tension of all the mooring chains was adjusted and confirmation was made concerning the correct mooring position for the Aquapolis.

The detailed procedures of the mooring work are mentioned below.

1) Two 15-ton anchors for temporary mooring (Nos. 10 and 15) were thrown into the seas (Fig. 19).
2) Two 15-ton anchors for temporary mooring (Nos. 2 and 7) were thrown by two anchoring boats (Fig. 20).
3) Temporary mooring work was completed (Fig. 21).
Figure 20. Throwing conditions of No. 2 and No. 7 temporary anchors

4) Four anchors (Nos. 1, 8, 9 and 16) for permanent mooring were completed. The Aquapolis was moored to these four anchors to withstand wind velocities up to 30 m/sec (Fig. 22).

5) The above four 15-ton anchors for temporary mooring (Nos. 2, 7, 10 and 15) were pulled out.

6) Four anchors for permanent mooring (Nos. 2, 7, 10 and 15) were completed.

7) Eight anchors for permanent mooring (Nos. 3, 4, 5, 6, 11, 12, 13 and 14) were completed.

8) The Aquapolis was brought to its mooring position and the tension of each chain of each mooring line was adjusted to 30 tons.

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Figure 21. Final condition of temporary anchor
Figure 22. Final anchoring (Nos. 1, 8, 9, and 16)

Confirmatory Test on the Spot

Upon completion of the mooring work, it was subjected to the confirmation test on the spot and proved to have the initial design performance. It was then delivered to the Exposition Headquarters on May 13th.

The kinds and contents of the confirmation tests are briefly mentioned below.

Operation test of machinery and equipment

The machinery and equipment on the Aquapolis were operated for the purpose of proving that nothing unusual occurred during navigation and at its moorings.

Performance test on air-conditioning equipment

Performance test on each machine of the air-conditioning equipment was carried out prior to the departure of the Aquapolis from Hiroshima. An overall test (whole pavilion cooling test) was conducted after its arrival in Okinawa.
Floating and submerging test

Floating and submerging tests were carried out by filling and discharging ballast water to obtain three kinds of draft conditions: afloat (d = 5.8 m), storm-resistant (d = 12.5 m, 15.5 m) and semi-submerged (d = 20 m). A further confirmatory test was performed to check the performance of the watertight doors (in the case of d = 5.8, d = 20 m, visitors can enter the Aquapolis).

Mobility test

A mobility test was conducted under the three kinds of draft conditions mentioned above by operating the windlass.

Windlass stretching test

In order to confirm the performance of the windlass, the windlass was given a stretching test under the storm-resistant draft condition and the initial tension and delivery length of each anchor chain were examined (design tension = 250 tons).

Test of the joint of the bridge spanning between land and Aquapolis

After mooring the Aquapolis at a point facing the shore, the moving part of the bridge was brought about close to the Aquapolis. A measuring instrument was installed on the bridge and relative movements to the Aquapolis such as vertical displacement, horizontal displacement (parallel and perpendicular to the center line of the bridge), and the period of oscillation, were measured.

CONCLUSION

As a national undertaking, the International Ocean Exposition Okinawa, 1975 has as one of its aims, the improvement of the nation's ocean development techniques by the concentration of the latest techniques to be offered by many Japanese enterprises. Performing a role to attain this aim, the Aquapolis was manufactured by the collective efforts of many Japanese manufacturers and assembled in the Hiroshima Shipyard and Engine Works of Mitsubishi Heavy Industries under the guidance of the Headquarters of the Aquapolis Project of the International Ocean Exposition Society. It left the Hiroshima Shipyard for Okinawa on April 18th and after mooring, and adjustment and on-the-spot tests, it was delivered to the Headquarters. The imposing figure of the Aquapolis now afloat off the open space named the "Setting Sun" in the exposition ground has been opened to the general public since July 20th.

We believe that it is a great achievement that the large-scale ocean structure has been completed without any accident and with high accuracy, towed to the destination and installed successfully (Table 6).

The Aquapolis will play a significant role as a pioneer of the floating city of tomorrow. At the period of construction, we could gain a valuable lesson in the construction of such a gigantic structure by using a small number of facilities. The cooperation of a great number of people, the passage of narrow sea areas like the Inland Sea and its installation in a place with many coral reefs were the matters worthy of attention.

We hope that such a concept of a floating city shall be expanded to a larger floating city which may be utilized for the benefits of mankind. In the very near future we also believe that the completion of the Aquapolis will give great suggestions as an offshore supply base in an oil exploration field remote from a coast. Concluding this paper I would like to express my sincere thanks to the people of government, and manufacturers who are concerned with Aquapolis, and applaud their sincere effort to achieve this epoch undertaking.

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TABLE 6. Schedule of Aquapolis

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<th>1975</th>
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<td>Outfitting works</td>
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<tr>
<td>Towing and installation</td>
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<tr>
<td>Preparation for opening</td>
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ON THE MECHANICAL PROPERTIES AND RESULTS OF SOAK TEST IN SEA WATER FOR ANTI-CORROSIVE WIRE ROPES

Isao Ueno
Ship Research Institute, Japan

INTRODUCTION

As mooring lines, used for the floating structures for ocean exploitation, and other wire ropes for marine use are exposed to severe corrosive environment (e.g., saline water or tidal currents) their usefulness is limited. Therefore, development of highly corrosion resistant materials has been investigated. Taking into view that stainless steel containing 3 to 5 percent Si (hereinafter referred to as "High Si") is excellent in corrosion resistance in boiling sulphuric acid and some other corrosive agents, we have confirmed by trial that this material can be drawn to wires and then fabricated into wire ropes. Further, mechanical properties and corrosion resistance to sea water of High Si wire ropes were investigated, and compared with five kinds of wire rope; uncoated, zinc coated, aluminium coated, lead coated and SUS 304 stainless steel wire rope.

OUTLINE OF MANUFACTURE OF HIGH SI WIRE ROPES

For the purpose of improving corrosion resistance in sea water and also making full use of its mechanical properties, wire ropes were manufactured in the following ways.

Raw Material, Melting and Ingot-Making

Scrap stainless steel and pure iron were used as raw materials. The ingot was of square shape made by top touring and weighed approximately 600 kg. The chemical composition of new material is shown in Table 1.

Forging and rolling

The ingot was forged at temperatures of 1050 to 1200°C into two billets weighing about 200 kg and measuring 110 mm x 110 mm x 2000 mm. Then each billet was rolled at temperatures of 1000 to 1200°C into a 9-mm diameter rod weighing about 180 kg.

Drawing and closing - Wires of 2.24 mm in diameter were manufactured from the 9-mm diameter rods by repeating the process of water-toughening, pickling, coating and drawing. Those wire ropes were 20 mm in diameter and of 7 x 7 (independent wire strand core) and 6 x 7 (polypropylene core) constructions.

MECHANICAL PROPERTIES OF WIRE ROPES

Before corrosion tests, mechanical properties of the High Si wire ropes and the other five kinds of ropes were examined.
INDIVIDUAL WIRES

Microstructure

A microstructure of an individual wire of High Si wire rope is shown in Figure 1 and those of other wire ropes will be shown by film slides.

Mechanical properties of individual wires

The tensile strength and the torsion value of the individual wire are shown in Table 2. As shown in this table, the tensile strength is about 190 kg/mm² and the torsion value is about 4 on the average as seen in the table.

Breaking load of wire ropes

The breaking loads of 7 x 7 and 6 x 7 ropes are shown in Figures 2 and 3. The breaking load and the elongation of 7 x 7 rope are 30,100 kg and 3.2 percent and those of 6 x 7 rope are 24,800 kg and 3.3 percent respectively.

Fatigue test - In order to examine the fatigue characteristics of High Si wire rope, conventional 20-mm diameter wire ropes with a construction of 6 x 7 langs lay were also tested. The results of the fatigue test are shown in Figure 4, which illustrates the relationship between the number of cycles of reversed bending and the number of broken wires produced. The sheave of the fatigue testing machine was 2000 mm in diameter, namely D/6 910, and test pieces were held under the constant tension of 2000 kg. The results indicate High Si wire ropes are likely to produce more wires than the above mentioned conventional wire ropes. But the difference is not so remarkable to judge High Si wire ropes as inferior, as the same trend is found with other stainless steel wire ropes.

SEA WATER IMMERSION TEST

To determine the corrosion resistance in the sea water, High Si wire ropes were immersed in the sea together with the other five wire ropes mentioned above, and the progress of corrosion in the test pieces were observed at intervals. Finally they were taken out of the sea to examine the mechanical properties of their individual wires.

Comparative Ropes

These five kinds of wire rope, tested together with High Si wire ropes, were of the same construction of 6 x 7 and of 20 mm in diameter.

Immersion method

The immersion test was made as shown in Figure 5. Test pieces of 10 m each were held at a space of 0.2 m and wooden pieces were placed in-between to prevent their contact with each other. This test was carried out at the new pier of Chiba Port where the water was approximately 3.4 m deep.

RESULTS OF IMMERSION TEST

Test Results

Test specimens were immersed in the sea for 27 months. They were picked up to investigate the surface conditions four times; the first investigation was made 6 months after the immersion, the second 9 months, the third 21 months and the last 27 months. Then the test pieces were subjected to mechanical tests.
State of corrosion

Figure 6 shows the four stages of corrosion progress on test pieces held at a water depth of 3 m. Comparison of surface conditions on all test pieces before and after the immersion test at other levels of depth will be shown by film slides.

Breaking load test on individual wires after immersion test - Details on the residual breaking load and torsion value of the individual wires immersed for 27 months are shown in Table 3. The notation "m" in the upper line in Tables 3 and 4 shows the distance from the sea bottom and the mark "-" in Tables 3 and 4 indicates that the test could not be made owing to development of corrosion on the test pieces. Tables 4 and 5 show the aggregate breaking load of each wire rope calculated from the breaking loads of the individual wires.

CONCLUSION

It has been confirmed through this trial that the wire ropes can be manufactured from the new material (High Si) through the process of melting, rolling, drawing and roping. Experience tells that for mass production, further improvement is needed in the process of hot-rolling and wire-during.

The almost satisfactory results were obtained concerning the mechanical properties of High Si wire ropes.

Judging from the surface corrosion and the drops in the mechanical properties after the sea water immersion test, High Si wire ropes proved to be fairly superior to other wire ropes tested together.

This wire rope is manufactured by Sinko Wire Co., Ltd. (2, 7 chome, Amagasaki-city, Hyogo-ken, Japan).

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<th>TABLE 1.</th>
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<td>kg/mm²</td>
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<td>(mean)</td>
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<td>High Si</td>
</tr>
<tr>
<td>SUS</td>
</tr>
<tr>
<td>Al</td>
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<td>Black</td>
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<td>Zn</td>
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1377
Figure 1. Microstructure of an individual wire

Figure 2. Breaking loads

7X7 20mm
Breaking Load 30100 kg
Elongation 3.20 %

Figure 3. Breaking loads

6X7 20mm
Breaking Load 24500 kg
Elongation 3.30 %
Figure 4. Results of fatigue test

Figure 5. Immersion test

Figure 6. Corrosion progress
### TABLE 3.

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<tr>
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<th>Elemental wire</th>
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<td></td>
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### TABLE 4. Residual breaking load.

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