◉ Technical Paper

# Ice Impact on Arctic Gravity Caisson+

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# 극지용 중력식 해양구조물의 유빙충격 해석

유 병 권

**Key Words** : Ice Impact(유빙충격), Arctic Gravity Caisson(극지용 중력식 구조물), Ice loading(빙하중), Ice-Structure-Soil Interaction(유빙-구조물-토양 상호작용), Foundation Failure(기초붕괴), Dynamic analysis(동적해석)

초 록

극 지역용 중력식 해양구조물의 설계시 고려되어야 할 문제중의 하나가 구조물이 유빙(ice)과 충돌시 야기되는 foundation붕괴 현상인데, 본 논문에서는 정적 해석을 위해 sliding 및 bearing failure 현상에 대하여서만 연구하였고, 또한 동적 해석을 위하여 soil과 ice의 특성으로 부터 structure-ice-soil의 상호작용 운동 방정식을 설정하여 구조물과 ice의 변위, 속도, 가속도와 ice force와 soil force의 history를 시간영역 해법으로 풀었다. 한 예제 (Table 1 참조)로 Beaufort Sea의 37 *feet* 수심과 granular soil 상태에서 구조물의 최대변위는 0.4 *feet*이고 가속도는 약 0.3$g$이며 이때 구조물이 sliding에 대하여 안전하다는 것이 입증되었다.

## 1. Introduction

The design of safe, economical structures in the offshore Arctic requires accurate prediction of ice loads for a wide variety of ice conditions (See Fig. 1). In general, the ice conditions prevailing at a specific site depend primarily on the existing water depth. The transition zone, which is considered here, extends into the the Beaufort Sea where it merges with the Arctic pack ice zone. This transition zone is characterized by more ice movement and more intense ice deformation compared to the fast ice zone.

Developing an estimate of the design ice load involves several components. First, the design ice feature must be selected and its properties specified. From this, the load required to fail the fea-

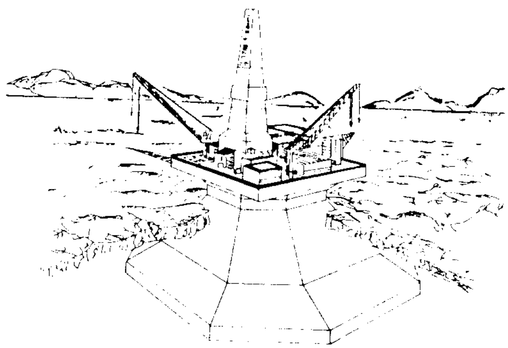

Fig. 1 Arctic offshore caisson

+ Presented at the 1987 KCORE Autumn Conference

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ture against the structure can be calculated. Implicit is the assumption that a sufficiently large driving force exits to push the ice feature into the structure with sufficient movement to cause complete failure of the ice. Conversely if the available driving force is less than that required to cause failure, the design load can justifiably be reduced.

Based on an understanding of the ice conditions at the site, statistically significant loading can be investigated.

In this paper analysis is made for break-up impact load which refers to the condition after in breakup which a large ice floe fails against the entire width of the structure at a high velocity.

## 2. Analysis

### 2.1 Quasi-static Analysis

The risk of a shear failure in the soil strata below the base of a gravity platform is of paramount importance and is normally investigated for two possible failure modes[1]:

1) Sliding failure along or near to the base of the structure
2) Deep seated failure of bearing failure.

For a horizontal failure the classical approach is to compare the mobilized shear strength of the soil with maximun shear stress caused by the maximum horizontal thrust.

A lower bound of sliding resistance can be established by determining the resistance developed between a smooth foundation with a flat base and the sea floor soils (See Fig. 2). The equation is described by Ref. (2)

$$P_{max} = Q tan\delta + AC_a$$

where

$P_{max}$: max. horizontal load before sliding occurs
$Q$ : net vertical load
$\delta$ : angle of friction beween structure base and soil, approximated by $\phi$-5°
$C_a$ : adhesion between soil and base of foundation, appoximated by $C_u$
$A$ : area of foundation base.

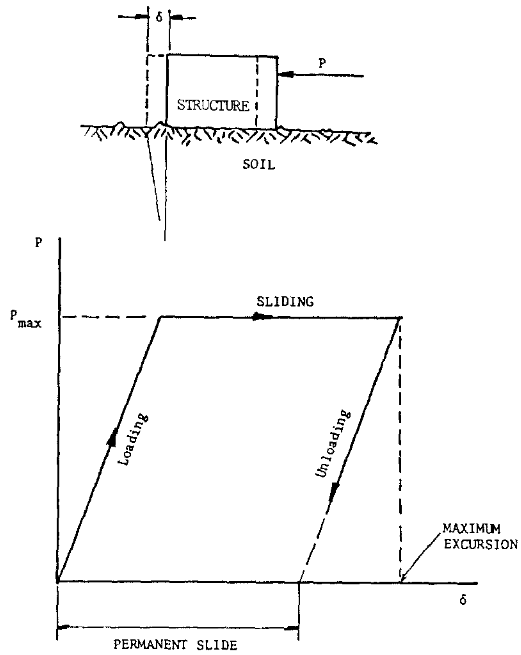



**Fig. 2** Soil-structure interaction

An upper bound of sliding resistance for a flat base foundation is defined by the above equation for $\delta$ equal to $\phi$ (internal angle of friction of the soil) and $C_a$ equal to the undrained shear strength, $C_u$, of the soil.

When considering a deep seated or bearing failure, the combination of gravity load and horizontal load from the ice effectively produces both a gravity load, thrust load and overturning moment relative to the area moment of inertia of the foundation base. In this case the general capacity equation is given by Ref. (3)

$$q_0 = (rBN_r)/2 + C_u N_c + qN_q$$

where

$q_0$ : bearing capacity of soil
$B$ : effective width of foundation
$r$ : density of soil
$C_u$ : undrained shear strength
$q$ : surcharge
$N_r, N_c, N_q$: bearing capacity factors (See Fig. 3).

In the case of dense sands or stiff clays, the depth to which the structure is buried will be small. The term $qN_q$ will be neglected.

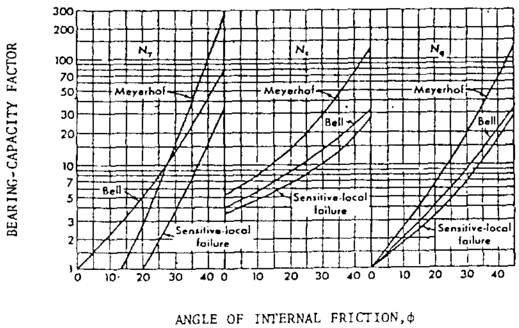



**Fig. 3** Bearing-capacity factors

The overturning moment created by the thrust load of the ice can be simulated by moving the vertical load off by the eccentricity, $e$. The foundation responds as if it had a reduced witdth, $B'$:

$$B' = B - 2e.$$

The value of $q_o$ computed from the reduced width is the average, and is used with the reduced width again in computing total capacity, Q. For a nonhomogeneous soil, a reasonable approximation can be made by using mean soil properties. Alternatively, if a weak stratum overlies a strong one, bearing capacity should be computed from the strength of the weaker. If a strong layer overlies a weak stratum, the bearing capacity is computed from the strength of the weaker stratum using the reduced bearing pressure in the weaker stratum due to its vertical distance from the applied load.

The critical load which can be resisted is given by the minimum obtained from each of these failure modes. If dynamic thrust loads are acting, the maximum value of the dynamic load is used-thus the reference quasi-static.

### 2.2 Dynamic Analysis

If the offshore structure is regarded as rigid when compared with the soil, six independent rigid body displacements are possible. For the case where the loads are in a vertical plane, only three independent displacements are possible: 1) translation in the vertical, 2) translation in the horizontal, and 3) rocking. The horizontal translation and rocking are coupled.

The dynamic analysis of the soil-structure system will be based on the assumption that the soil can be replaced by a linear elastic weightless spring for displacements less than those which cause failure. For a block undergoing vertical oscillations, horizontal translation and rocking independently, the stress below the block are uniform compression, uniform shear and non-uniform shear, respectively[4].

Unlike the theory of elasticity in which the shear modulus is $G$ and modulus of elasticity is $E$ independent of whether the stress state is uniform or non-uniform, soil coefficients are defined for uniform and non-uniform stress distributions respectively. Coefficient of elastic uniform compression, $C_u$, is considered analogous to $E$ and coefficient of elastic uniform shear, $C_\tau$, analogous to $G$. The relation between $C_u$, $C_\tau$ and the coefficient of elastic non-uniform compression, $C_\phi$, can be approximated by Ref. (5)

$$C_\tau = C_u/2$$
$$C_\phi = 2Cu$$

From the definition of these coefficients, the spring constants are given by:

1) For displacement in the vertical

$$k_z = C_u A$$

2) For displacement in the horizontal

$$k_x = C_\tau A$$

3) For rotation about the $y$ axis

$$k_\phi = C_\phi I$$

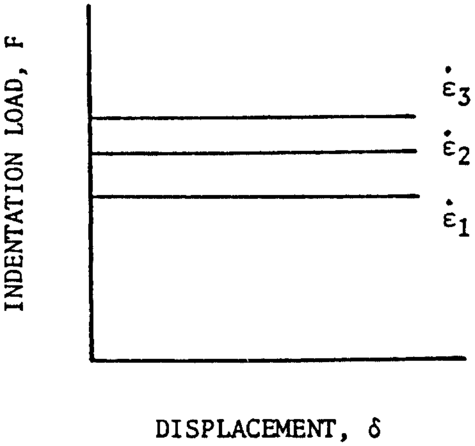
where

$A$ : base contact area

$I$ : moment of inertia of the contact area about an axis passing through the centroid of the base contact area.

If the indentation force-deflection relationship for ice is assumed perfectly plastic with strain rate (or indentation velocity) as a parameter, the ice-structure interaction can be represented by a dashpot since the force transmitted across it is strain rate (or velocity) dependent (See Fig. 4)

Since the lateral ice load is above the foundat-



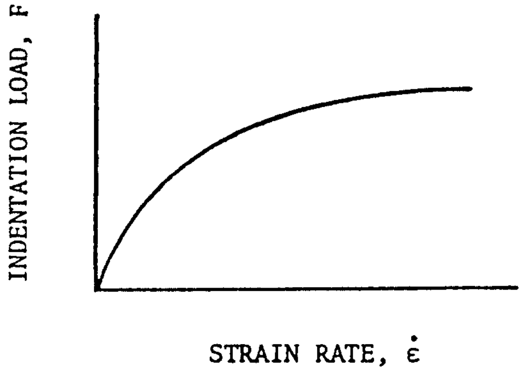




**Fig. 4** Idealized indentation force-displacement relation with strain rate as parameter

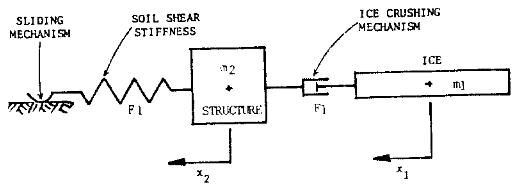



**Fig. 5** Idealized soil-structure-ice model

ion base, both rocking and sliding occur simultaneously. If the vibrations in rocking are small and can be neglected, only the horizontal displacement of the foundation will occur. Based on this and previously developed ice-structure and soil-structure characteristics, the soil-structure-ice model can be idealized by the 2 degree of freedom system shown in Fig. 5, with the dashpot and spring having non-linear properties. The simultaneous system of differential equations are given:

For the ice:

$$m_1\ddot{x}_1+F_1=0$$

For the structure:

$$m_2\ddot{x}_2-F_1+F_2=0$$

Initial condition at $t=0$

$$x_1=x_2=\dot{x}_2=0,\quad \dot{x}_1=V_{io}$$

where

$m_1$: virtual mass of ice

$m_2$: virtual mass of structure

$x_1, \dot{x}_1, \ddot{x}_1$: displacement, velocity and acceleration of the ice, respectively

$x_2, \dot{x}_2, \ddot{x}_2$: displacement, velocity and acceleration of the structure, respectively

$F_1$: ice-structure interaction (function of relative velocity)

$F_2$: soil-structure interaction(elastic, perfectly plastic function of relative displacement).

These simultaneous equations are non-linear and can be integrated numerically using Runge-Kutta methods.

## 3. Sample Calculations

Sample calculations are made based on environmental data in Beaufort Sea and structure model of Super CIDS[6] in Table 1.

**Table 1** Example data used for analysis

| | |
|---|---|
| Ice condition | |
| Diameter($ft$) | $=1000.00$ |
| Thickness($ft$) | $=15.00$ |
| Mass($slug$) | $=$ 0.23444E+08 |
| Added mass($slug$) | $=$ 0.35166E+06 |
| Total mass ($slug$) | $=$ 0.23796E+08 |
| Initial velocity | ($ft/sec$)$=1.69$ |
| Structure condition | |
| Foot print($ft^2$) | $=51177.00$ |
| Diameter ($ft$) | $=277.00$ |
| Mass ($slug$) | $=$ 0.13105E+08 |
| Added mass($slug$) | $=$ 0.14817E+06 |
| Total mass ($slug$) | $=$ 0.13253E+08 |
| Soil condititions | |
| Spring rate($lb/ft^2/ft.def$) | $=5983.00$ |
| Ultimate strength($lb$) | $=$9.13425E+09 |
| Site condition | |
| Water depth($ft$) | $=37.00$ |

### 3.1 Soil Properties

Three generalized near-surface soils condition in the Prudhoe Bay area were identified for the purposes of this study. They include 1) granular, 2) fine-grained, low plasticity and 3) fine-grained, high plasticity. Brief description of the generalized soil columns are given below.

Granular: The upper part of soil column consists of medium stiff, low plasticity sandy silt. This silt is moderately compressible and highly overconsolidated. Soils in this soil column are unbonded permafrost(See Fig. 6).

Fine-grained, low plasticity: The upper portion of soil column consists of low plasticity silt. Beneath the silt lies stiff to very stiff clay of low plasticity. The silt and clay layers are moderately compressible and overconsolidated.

Fine-grained, high plasticity: The upper part of soil column consists of highly plastic soil including medium stiff silt overlying soft clay. Both soil layers are very compressible. The silt is slightly overconsolidated while the plastic clay is normally consolidated. For calculation, granular soil column was used.

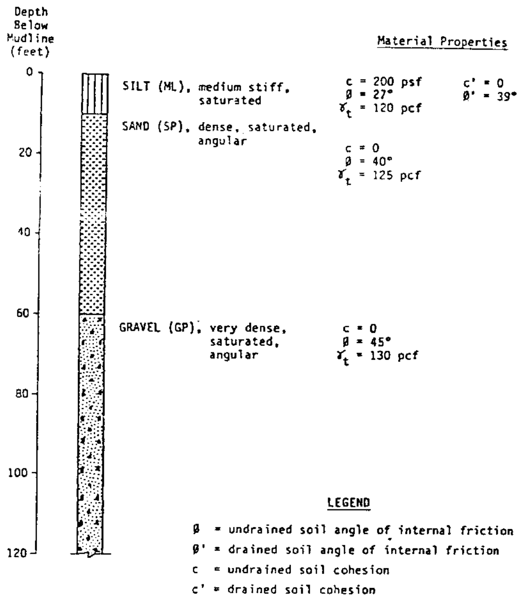



**Fig. 6** Granular soil column

### 3.2 Ice Properties

*Life Cycle of the Ice Field*

In the nearshore area, freeze-up occurs around September, with the ice sheet reaching a final thickness of about 6 to 7 *feet* inside the Barrier Islands before the late spring break-up period.

Based on air temperature data, the maximum sheet ice design thicknesses are estimated at 6.4 and 7.5 *feet*[7] for the 25 year and 100 year return period, respectively.

When sea ice starts growing, the intial development consists primarily of small granular crystals. As the ice sheet increases in thickness, the crystal type changes from granular to columnar. The so-called C-axis of these columnar crystals lie in the horizontal plane with a random orientation and are referred to as unoriented columnar. Because of this growth sequence, it is typical to assume that at a given time the ice sheet is proportioned into granular ice and unoriented columnar ice. According to Ref. (7), 1/3 and 2/3 are used respectively, whereas it is suggested 29% and 71% respectively by Ref. (8). Since granular ice is stronger, the 1/3 to 2/3 proportion is more conservative and will be followed. For the consolidated zone of a rubble pile, the ice structure is assumed to be all granular.

*Upper Bound on Ice-Structure Interaction*

Kozhavin's expression[9] is used to determine the crushing or indentation load for an ice structure as

$$F = f_C I f_\tau \sigma_C D t$$

where

$F$ : horizontal ice force

$f_C$: contact factor(=0.3)

$I$ : indentation factor(1.2 and 3.0 for granular and columnar respectively)

$f_\tau$ : temperature correction factor(=0.3)

$\sigma_C$: unconfined compressive strength

$D$ : width of ice failure

$t$ : ice thickness.

These ics load parameters are a function of a number of variables, including strain rate, local

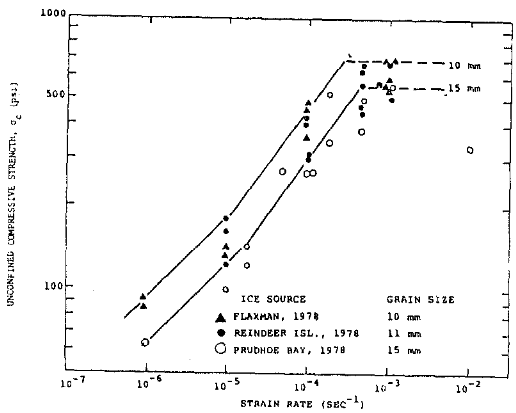



**Fig. 7** Unconfined compressive strength of unoriented sea ice at $-10°C$

geometric effect and ice type. Fig. 7 shows the unconfined compressive strength of unoriented columnar ice as a function of strain rate at a standard test temperature of $-10°C$. The relation between the strain rate and the indentation velocity is given by Ref. (8)

$$\dot{\varepsilon} = V/2D$$

where

$\dot{\varepsilon}$ : strain rate

$V$ : relative velocity between indenter and ice mass

$D$ : width of indenter.

For the break-up impact load, initially the structure is stationary, i.e., $V_{so}=0$, and the ice formation is approaching at the break-up velocity, $V_{io}$. The indentation velocity, $(V_{io}-V_{so})$, is therefore the break-up velocity, $V_{io}$. As the initial energy of the moving ice mass is reduced by the work done in crushing the ice or displacing the structure, the velocity of the ice mass decreases and the velocity of the structure increasss, with a corresponding deceases in the relative velocity.

During breakup, the initial relative velocity, $V_{io}$, will be on the order of 1 or more knots with a corresponding strain rate on the order of $3\times10^{-3}$ $sec^{-1}$ or more.

### 3.3 Quasi-static Calculation

For the structure under consideration, the maximum load or upper bound of the ice-structure was calculated. Based on this, Table 2 shows the allowable bearing capacity for a sliding failure or a deep seated failure for granular soil column. From this sliding is seen to be the critical failure mode.

### 3.4 Dynamic Calculation

The non-linear equations of motion can be approximated by two linear equations with constant coefficients for the spring and dashpot elements. This assumption is reasonable as long as the initial velocity of the ice mass is small and the soil-structure interaction force does not exceed the elastic region. The virtual masses of the ice and structure will, of course, remain unchanged.

Then equations of motion become:

For the ice:

$$m_1\ddot{x}_1 + c_1(\dot{x}_1 - \dot{x}_2) = 0$$

For the structure:

$$m_2\ddot{x}_2 - c_1(\dot{x}_1 - \dot{x}_2) + k_2x_2 = 0$$

Initial conditions at $t=0$

$$x_1 = x_2 = \dot{x}_2 = 0,\ \dot{x}_1 = V_{io}$$

The linear damping coefficient $c_1$, was determided by assuming the force-velocity relationship can be approximated by a straight line drawn through the origin and the ice-structure interaction load evaluated with strain rate of $10^{-3}$ $sec^{-1}$.

The solution to above equations is of the following form:

**Table 2** Comparison between deep seated and sliding failure

| Deep seated failure | | | Sliding failure | | |
|---|---|---|---|---|---|
| Allowable vertical load (*kips*) | Actual load (*kips*) | Factor of safety | Allowable horizontal load (*kips*) | Actual ice load (*kips*) | Factor of safety |
| $3.39\times10^6$ | 300,000 | 11.3 | 131,400 | 63,100 | 2.0 |

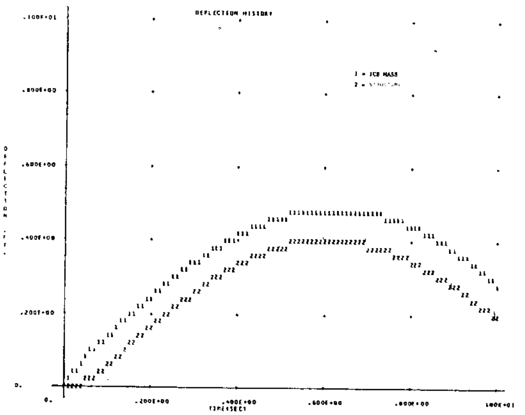


**Fig. 8** Deflection history

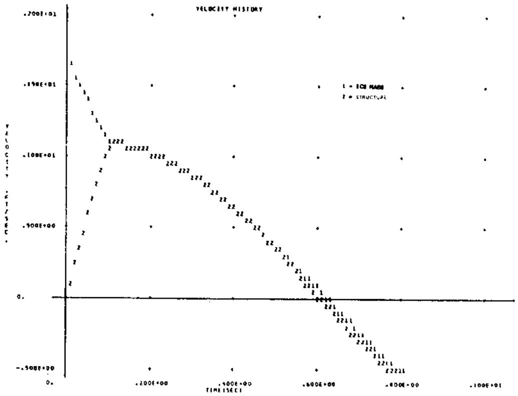


**Fig. 9** Velocity history

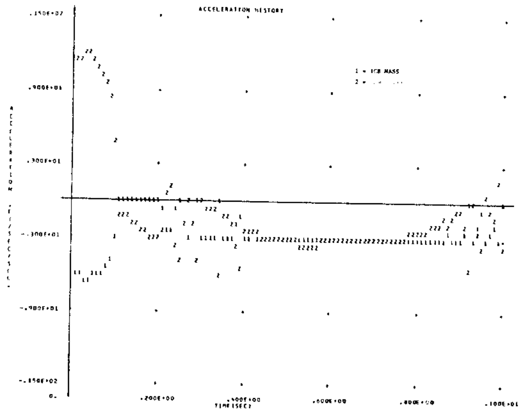


**Fig. 10** Acceleration history

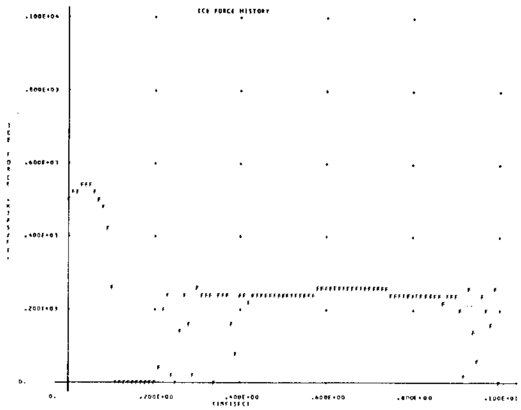


**Fig. 11** Ice force history

$$x_1 = Ae^{rt},\quad x_2 = Be^{rt}$$

By substituting this into the simultaneous differential system, the eigenvalues and finally the eigenvectors can be obtained, with the unknown coefficients determined from the initial conditions.

The theoretical solutions are obtained from linear equations using example data in Table 1. The maximum excursion is 0.40 *ft*. The maximum velocity is on the order of 1.1 *ft/sec*. The maximum acceleration of the structure is approximately 0.3*g*. The maximum ice force is 540 *kips/ft*. The maximum soil reaction of 117 *kips* is lower than that of maximum horizotal load before sliding occurs ($P_{max} = 131$ *kips*).

Consequently it is observed that the structure has safety factor of 1.15 against gross failure of the supporting soil. These results are shown in figures 8 through 12.

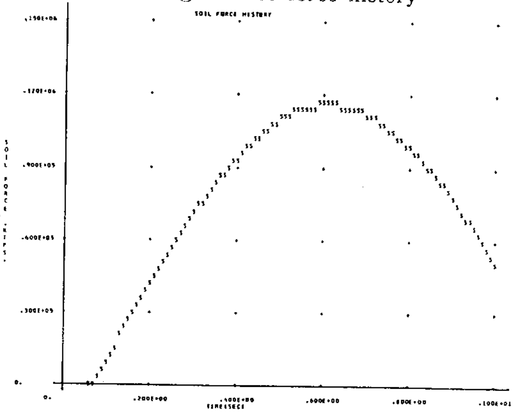


**Fig. 12** Soil force history

## 4. Conclusions

Following conclusions could be drawn based on above sample calculation analysis:

1) Sliding is the critical failure mode for Arctic gravity caisson.
2) It is noticed that caisson has stable foundation.
3) More rigourous analysis should be required for sophisticated ice and soil properties.

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**(Continued from page 51)**

**EXHIBIT PROFILE**

**1. DEVELOPMENT OF MARINE MINERAL RESOURCES**

Ocean Platforms
Structural Materials
Vessels
Submersibles
Vessel Instrumentation
Helicopters
Marine Resources Exploration Equipment
Oil Drilling Facilities and Eguipment
Desalination Equipment
Robots
Measuring Instruments and Devices
Communication Equipment and Devices
Related Technology
Satellites
Underwater Well Equipment
Marine Riser
Others

**2. THE USAGE AND DEVELOPMENT OF MARINE ENERGY**

Equipment and Devices for Power Generation by Thermal Energy Conversion
Equipment and Devices for Power Generation by Wave Energy
Equipment and Devices for Power Generation by Oceanic Currents
Related Technology
Others

**3. DEVELOPMENT OF MARINE BIOLOGICAL RESOURCES**

Vessels
Submersibles
Vessel Instrumentation
Helicopters
Measuring Instruments and Devices
Communication Equipment and Devices
Sonar Systems
Man-Made Reefs
Flotation/Buoyancy Equipment
Materials for the Fishing Industry
Satellites
Related Technology
Others

**(To be continued on page 66)**