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U.S. DEPARTMENT OF COMMERCE, Malcolm Baldrige, *Secretary*
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, *Director*

ABSTRACT

Design limit states for offshore gravity structures in the Alaskan Beaufort and Eastern Chukchi continental shelves are discussed. The report contains a description of geological conditions, design loads, and type of structures used. Three foundation types are considered: foundations for artificial islands; foundations for caisson retained island with sand cores; and rigid foundations for various types of gravity structures which are positioned on the ocean floor with a minimum of preparation. Design limit states for these foundations are identified and the required reliability against the occurence of these limit states is discussed. Our ability to determine foundation resistance is assessed.

Key Words: artificial islands; geotechnical engineering; ice forces; mat foundations; ocean engineering; offshore platforms; oil production; sand and gravel berms; soil exploration; soil testing.

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1. INTRODUCTION

Offshore structures for oil and gas exploration in Arctic regions, and particularly in the Beaufort Sea, must cope with unique environmental conditions. The configuration, design and construction of these structures and their foundations are to a large degree dictated by these conditions. The most critical environmental effects are: the short period of time available for construction (about 30 days of open water conditions); the logistic difficulties associated with the transport of structural materials and components to the site; the scarcity of nearby sand and gravel deposits at most sites; the large lateral ice forces acting on the structures; and the unique soil conditions prevailing at most sites presently under consideration.

The transfer of lateral loads from offshore structures to the supporting soil on the sea floor is a design constraint which has a major effect on the cost and configuration of these structures. Foundation types presently under consideration are large mat foundations. The structural systems comprising these foundations can either be constructed on artificial sand or gravel berms or placed directly on the seafloor with a minimum of preparation.
The load capacity of such mat foundations can be augmented by adding skirts or large diameter piles (spuds), by various methods of soil improvement which increase the shear strength of the supporting soil, and by underfilling methods which improve the contact between the mats and the supporting soil. Alternately, lateral loads or required safety margins can be reduced by various design and operations strategies.

The purpose of this report is to define critical design limit states for mat foundations in the arctic, discuss design strategies used to prevent occurrence of these limit states, and assess our capability to determine the reliability against foundation failures associated with these limit states.

Chapter 2 of the report deals with soil conditions; chapter 3 discusses design loads; design limit states and strategies are discussed in chapter 4; and chapter 5 contains a summary and suggests needed research.

2. SUBURFACE CONDITIONS

2.1 General

The currently available data base for continental shelf areas in the U.S. Beaufort and Eastern Chukchi seas is derived from multichannel seismic reflection profiles (see figure 1) and other geotechnical exploration data taken by U.S.G.S. and interpreted in Reference [1], and from proprietary soil exploration studies carried out in conjunction with specific projects. Some of the available information does not lend itself to accurate interpretation and is likely to be re-interpreted as more geotechnical information becomes available.

The National Petroleum Council estimated [2] that " proven technology and sufficient expertise for advanced design work is available for the industry to proceed confidently with operations in water as deep as 650 feet (200m) in the Southern Bering Sea and to about 200 feet (60m) in the more severely ice covered areas of the Northern Bering, Chukchi, and Beaufort Seas". Consequently, this report is primarily concerned with conditions to a water depth of about 60m (the 60m isobath). In the U.S. Beaufort Sea this covers most of the area landward of the continental shelf break.

Since this report deals with the stability and load-deflection characteristics of structural foundations, the discussion of suburface characteristics is confined to the unconsolidated deposits which affect structural performance. It is, however recognized, that structural performance may also be affected by drilling problems encountered in much deeper seated formations (reservoir subsidence). Such problems are not within the scope of this report.

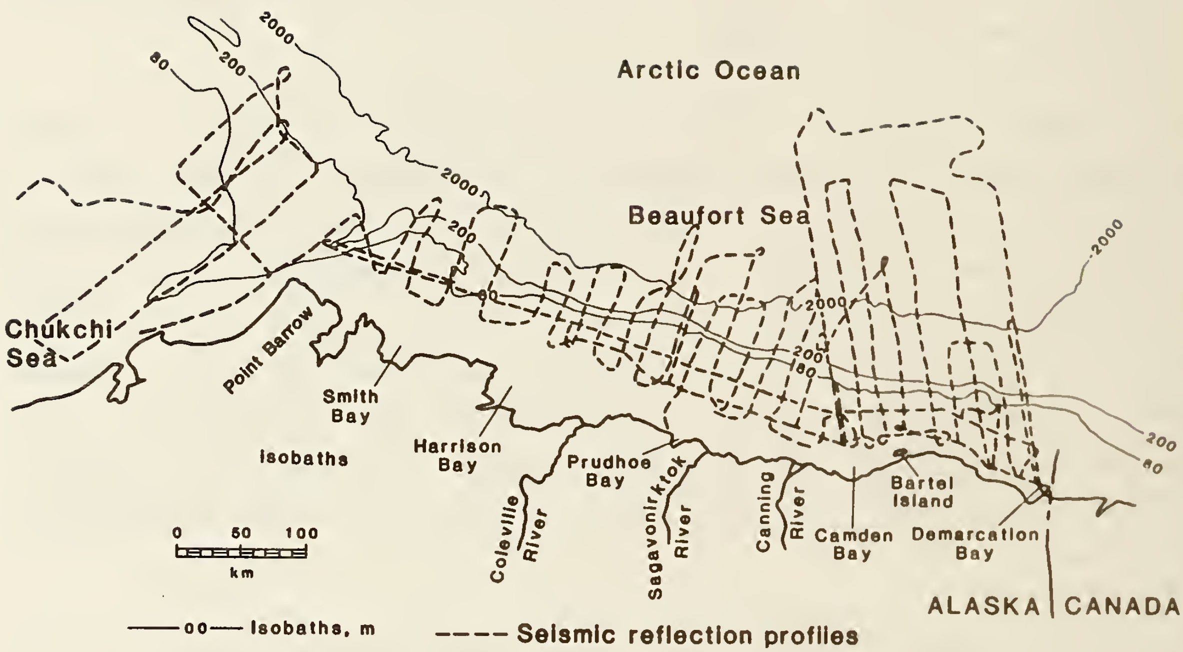


Figure 1. Seismic Reflection Profiles Taken by U.S.G.S. in the Alaskan Beaufort and Northeastern Chukchi Seas [1].

2.2 Geological Characteristics of Unconsolidated Deposits

2.2.1 Surficial Geology

The continental shelf of the U.S. Beaufort Sea is underlain by the Quaternary deposits of the "Gubik Formation", which consists of shallow water marine and terrestrial sediments, containing sands, gravels, silts and clays. These deposits vary in thickness from 10 to 200m. The Gubik formation is generally covered by much looser Holocene deposits which are 5 to 45m thick and typically tend to form a wedge which thickens in the offshore direction and reaches its maximum thickness near the shelf break. This wedge of Holocene deposits tends to be thicker in the Eastern half of the Beaufort shelf. The Holocene deposits in the shallow parts of the Eastern Chukchi Sea appear to be in most places less than 5m thick but to increase locally to about 12m.

Figure 2 shows contours of the thickness of the most poorly consolidated marine sediments. Thewe contours were based on USGS high resolution profiles, with some of the information corroborated by drilling and diving samples. Along the coast, the Holocene layer appears to be thin or absent because of erosion effects. Also, there is relatively little sediment accumulation in deltas and offshore from rivers.

2.2.2 Evidence of Instability

In the middle and outer shelf there is some evidence of development of very low angle bedding plane slides. This may be taken as an indication that the shear strength of the deposits, or of some discrete strata within these deposits is probably low. High resolution seismic profiles spaced 15 to 50 km apart along the entire Western part of the shelf show evidence of instability terranes seaward of the 50 to 65m isobath along the shelf break. These include tabular sheets up to 38 km long and typi-

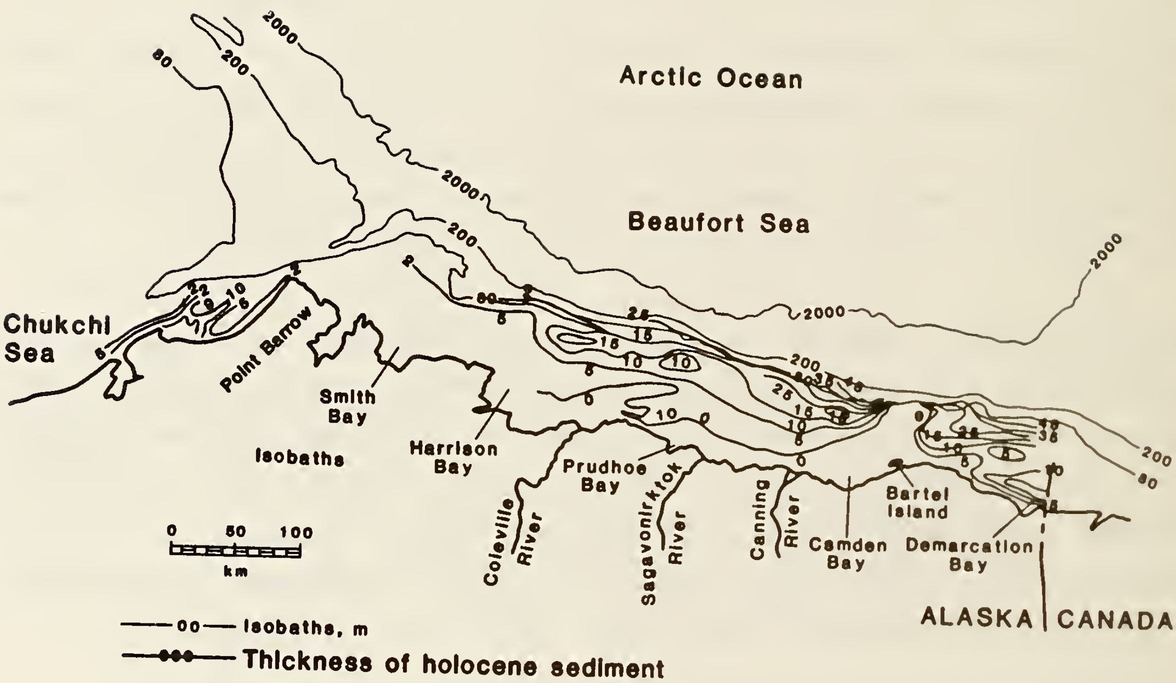


Figure 2. Thickness of Holocene Marine Sediments on the Alaskan Beaufort and Chukchi shelves [1].

cally 20 to 230 m thick which move seaward along slip planes which dip only 0.5 tO 1.5 degrees, and thus must include layers of very low shear strength. Figure 3 shows a preliminary mapping of these youthful landslide terranes.

Along the Beaufort coast, coastal erosion and scour are widespread and result in the seaward migration of barrier islands. Large scour craters (strudel scours), 15 to 25m in diameter, are formed near the mouth of rivers during the spring flooding of fast ice areas. Erosion and scour data are shown in figure 4.

2.2.3 Seismicity and faults

Most of the area under discussion has been historically aseismic, however a single earthquake has been located 200 miles north of the Coleville River delta, and there is a zone of concentrated seismic activity in the vicinity of Barter Island. Figure 5 shows a U.S.G.S mapping of epicenters for earthquakes of Magnitude 3 or greater. The largest of these recorded earthquakes had a Magnitude of 5.3.

Young faults are abundant in the Camden Bay area. They are thought [1] to be associated with an area of Holocene uplift. Available data suggest that at least some of these faults are active. Some additional faults which displace Pleistocene deposits were also recorded. It is suggested [1] that the midshelf faults, which would be of concern in conjunction with offshore structures, are either quiescent or have a very long recurrence interval.

2.2.4 Phenomena Associated With the Arctic Region

(1) Relict Permafrost

Quaternary deposits below the Holocene deposits were frozen during the last glacial sea level lowstand to a depth estimated to exceed 300m [1,4,5]. Much of this permafrost

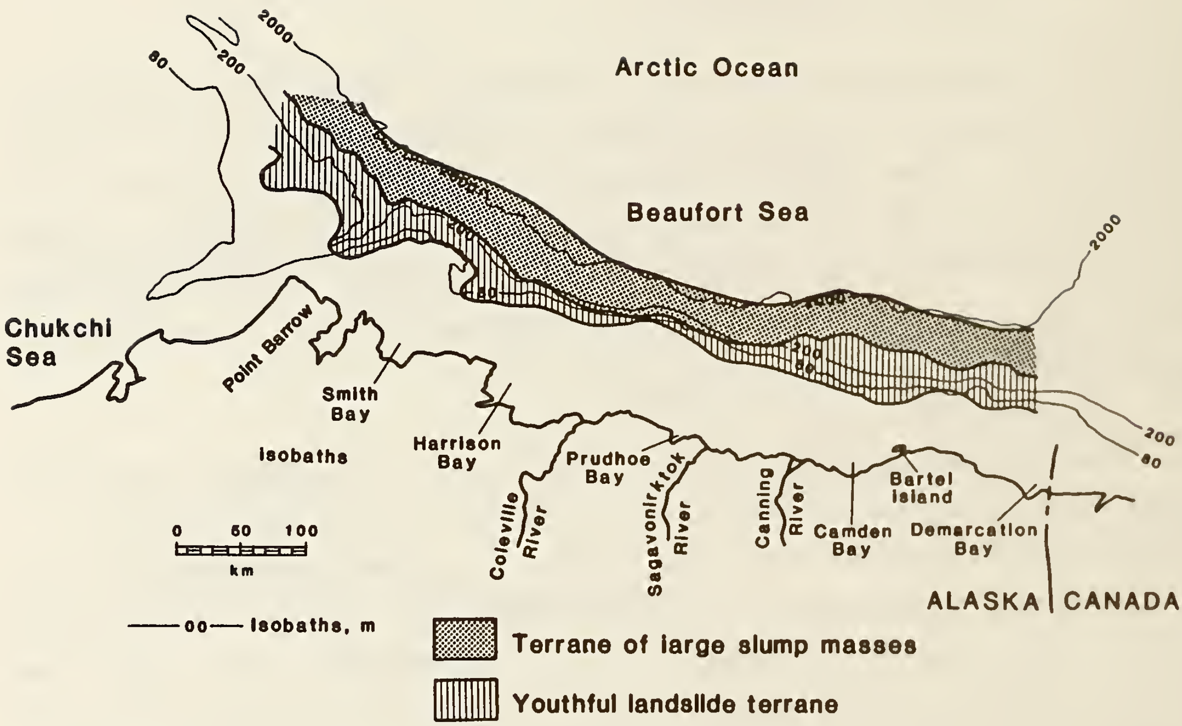


Figure 3. Youthful Landslide Terranes [1].

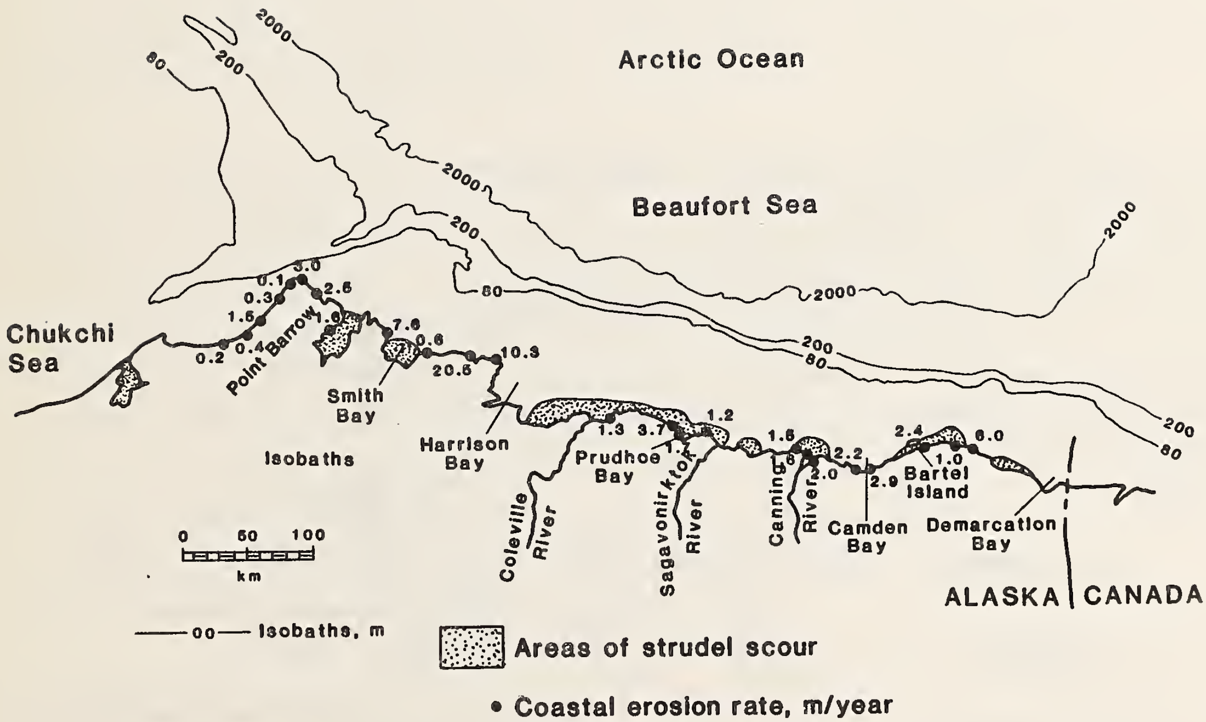


Figure 4. Data on Scour and Erosion [1].

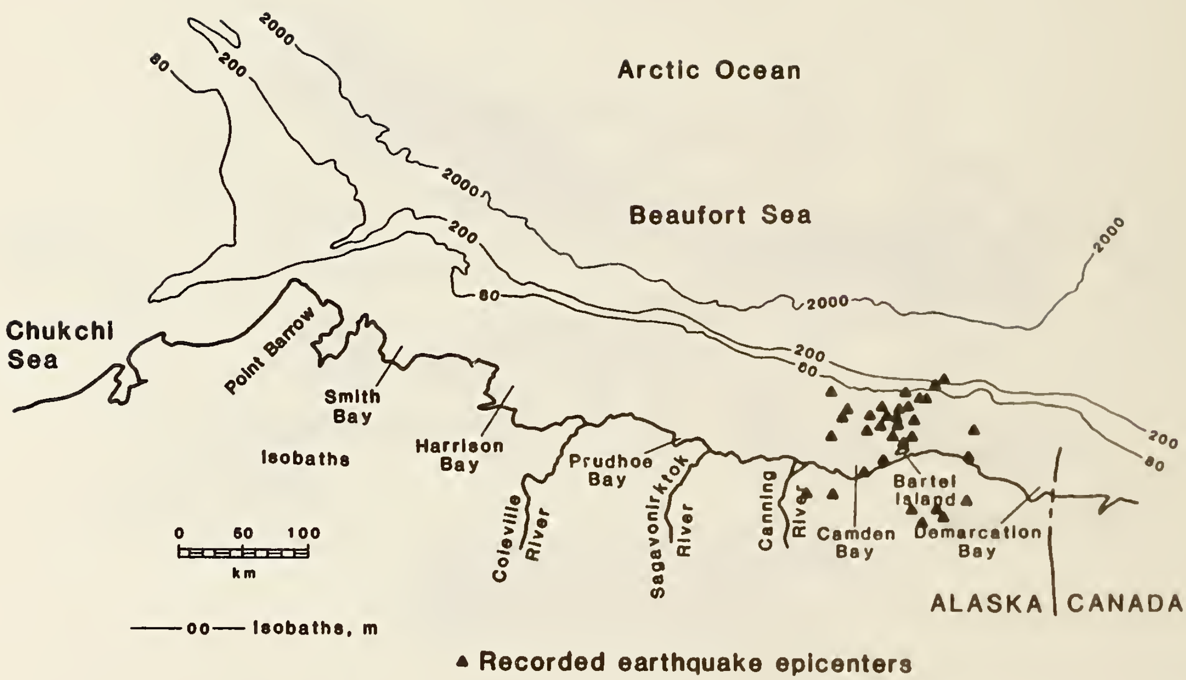


Figure 5. Recorded Earthquake Epicenters [1].

melted after exposure to saline water above the freezing point. However, a significant portion of the deposits is still fully or partially ice bonded. The distribution of this "relict" permafrost was not extensively explored. However, it is reasonable to assume that there is no relict permafrost in the offshore Holocene deposits which are geologically younger than the Pleistocene permafrost. Areas on the inner shelf that were characterized [4] exhibit ice bonded permafrost at variable depths which may be several hundred meters, overlain by partially bonded permafrost. Nearshore, permafrost is also likely to be present in areas of rapid coastal erosion. Subsea permafrost recorded in the Chukchi Sea is discussed in Reference [5].

The presence of relict permafrost beneath other soil deposits poses important constraints to the design of structural foundations. These include settlements and loss of bearing strength associated with the thawing of these deposits. Thawing can be caused by changes in the thermal regime of the subsoil induced by the presence of the offshore structure and by production or exploration wells. Very large settlements can result from the effect of oil production or exploration, even if the relict permafrost is located at a considerable depth.

(2) Shallow Gas and Gas Hydrates.

Shallow gas deposits, either of thermogenic origin (originating from natural gas deposits at greater depth, or of biogenic origin (decomposition product from buried organic material) have accumulated in many areas beneath the shelf of the Beaufort and Chukchi Seas [1] (see figure 6). These gas deposits pose structural hazards by inhibiting the normal consolidation of soils and thereby causing pockets of abnormally low shear strength. They also can cause blowouts during drilling operations.

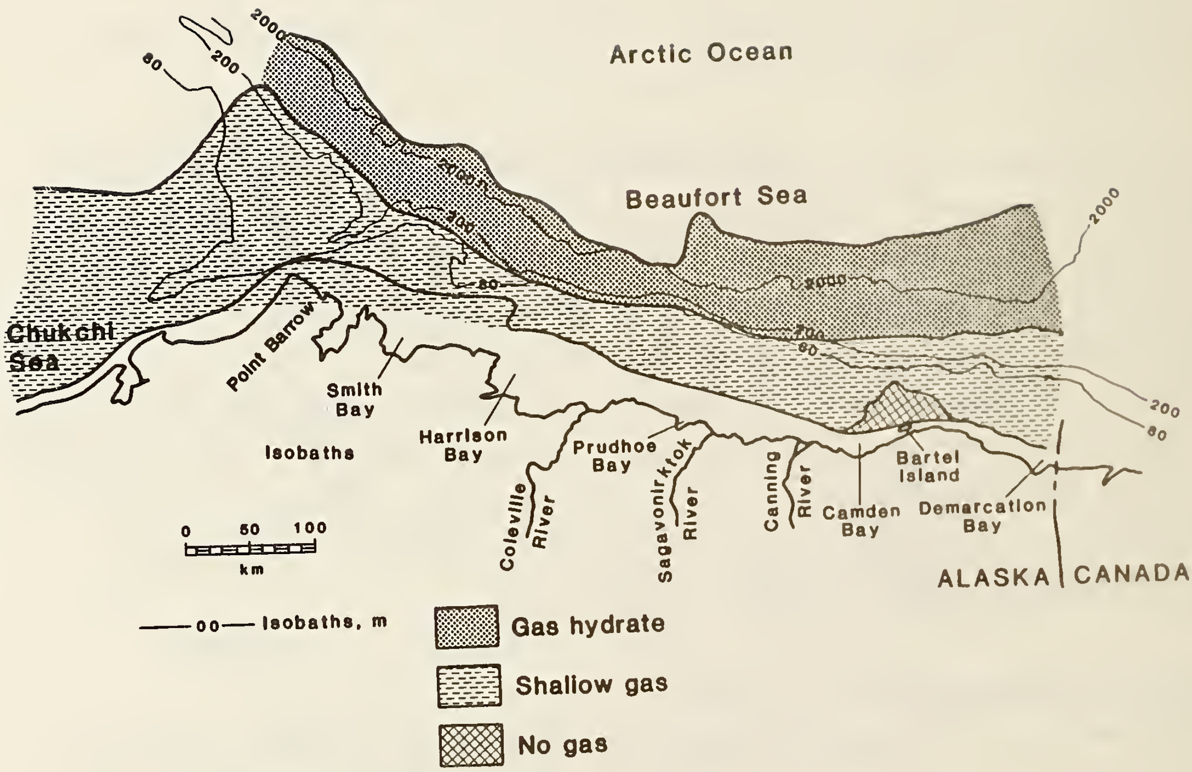


Figure 6. Areas of Shallow Gas Deposits and Hydrated Gas [1].

Gas hydrates, which are gases caged in the interstices of an expanded ice crystal lattice, occur in the relict permafrost deposits at greater depths, as seen in figure 6. If encountered, and there is no evidence that this would happen landward of the 60m isobath, they could pose a hazard if changes in the thermal regime cause the permafrost to thaw.

(3) Ice Gouges

Ice gouging is discussed in References [1, 6, and 7] which in turn reference many studies. The process of ice gouging can be understood in terms of the ice regime, which fluctuates seasonally. With the progress of winter, the area inside the barrier islands freezes and forms a zone of fast ice (very little or no ice motion). Part of this fast ice at a shallow water depth extends to the full water depth ("bottom fast ice"), while the other part at greater depth is floating. At the seaward boundary of the fast ice is the polar ice pack, which consists of ice floes, many of them multi year ice, which move generally in a westerly direction. The polar ice pack intrudes on the zone of fast ice and exerts pressure on it. As a result, a zone of first year and multi year ice ridges (the "Stamuki" zone) forms. These ridges build up to a great thickness and have deep keels which are frequently grounded and generally form in water depths from 15 to 45m. Figure 7 shows a schematic sketch of the ice regime during early spring before the onset of thawing. Figure 8 shows the approximate limits of the various ice zones. In spring Arctic rivers flood the fast ice canopy which eventually breaks up and melts, while the polar icepack retreats. Maximum open water generally occurs in September and early October, however grounded remnants of the Stamuki zone may persist during the open water season.

Gouging is caused by the keels of grounded ice ridges which are dragged along the sea floor. The most intensive gouging

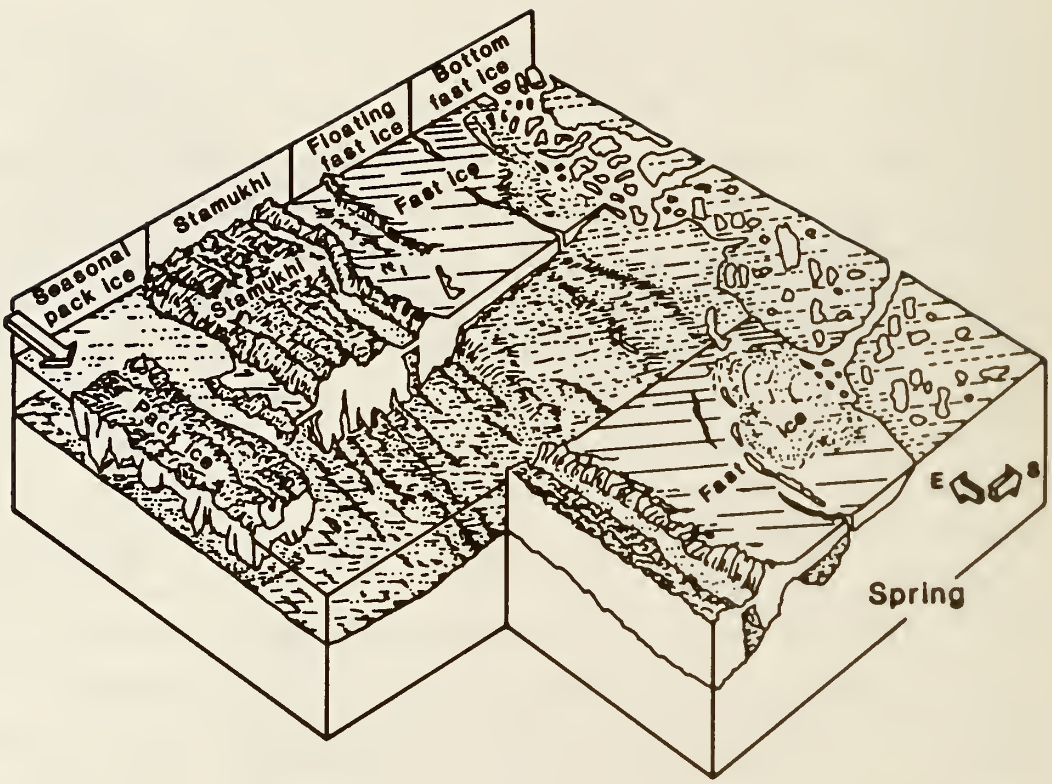


Figure 7. Schematic Sketch of Ice Regime in Early Spring [6].

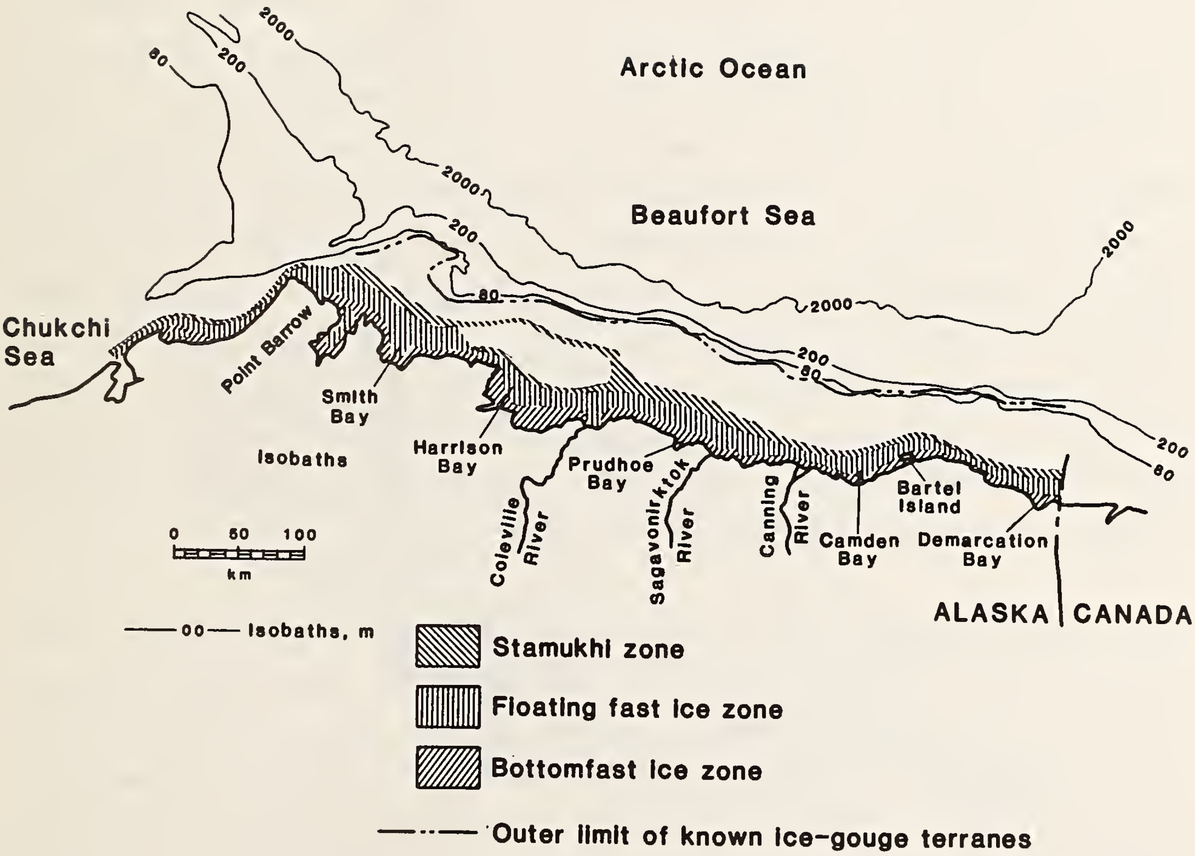


Figure 8. Ice Zonation in the Alaskan Beaufort and Eastern Chukchi Seas [6].

in the Beaufort Sea occurs in the Stamuki zone between the 15 and 45m isobaths. In accordance with Barnes et al.[6], a "typical" maximum gouge per 1km tracking length, embodying the mean value of extensive survey data, would occur in 18m deep water, form a furrow 0.56m below the mean sea floor and a ridge 0.47m above the mean sea floor (total relief of 1.03m), and have a width at the seafloor level of 7.8m. There would be 70 gouges per km^2 with an average tracking length of 1km in each km^2. The dominant orientation of the gouges is E-W, as would be expected with an ice movement in a Westerly direction. An idealized sketch of a gouge and a gouge multiplet (typically caused by first year ice ridges) is shown in figure 9. Figure 10 shows a mapping of observed gouge intensities, where gouge intensity is the product of maximum gouge depth in m, maximum gouge width in m, and gouge density in No. of gouges per km^2, and is used as a statistical measure. Maximum gouge dimensions observed in the survey in Ref.[6] were 67m width, 4m depth (with a single value of 5.5m) and 5m height of flanking ridges. Maximum density observed was 500 gouges per km^2.

The engineering implications of these gouges are very serious, particularly when it is planned to use pre fabricated mats resting directly on the ocean floor, in which case the gouges would result in only partial contact between the mat and the ocean floor. The effect of backfilling these gouges in the case of underfill and gravel berms would also be uneven contact stresses, since it is not feasible to construct compacted fills.

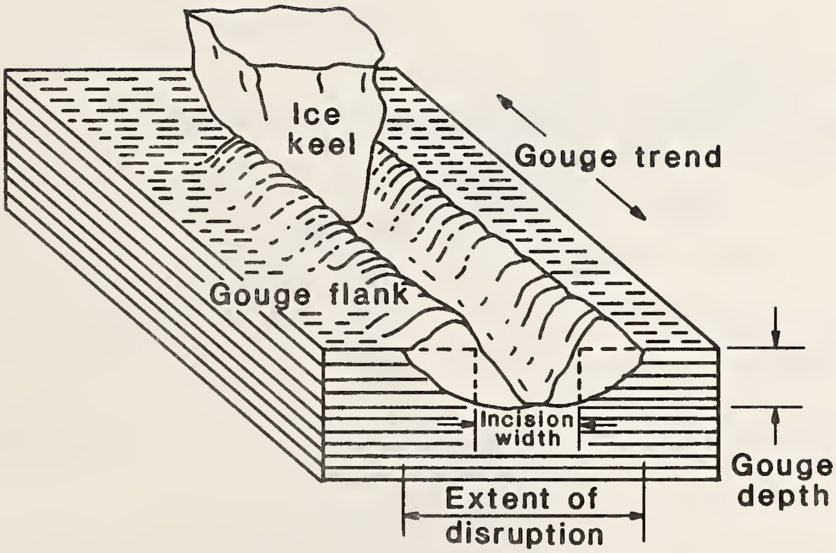


Figure 9. Idealized Sketch of Ice Gouge.

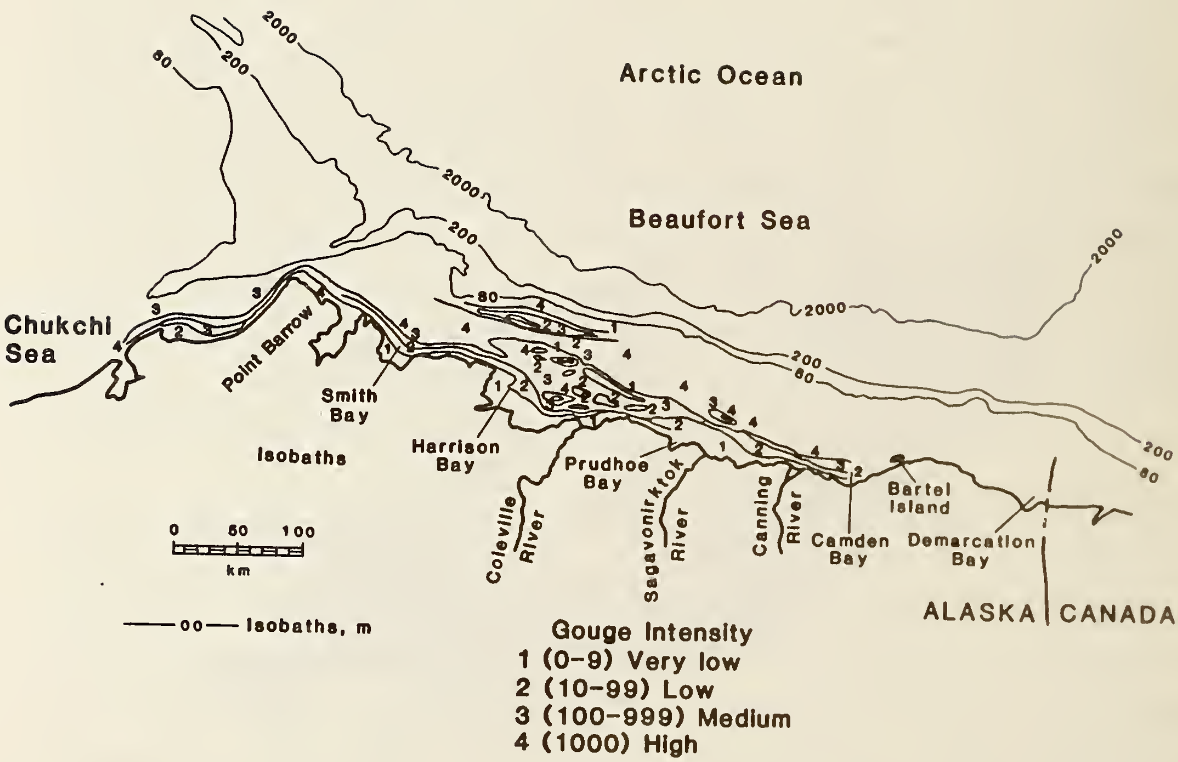


Figure 10. Ice Gouge Intensities in the Alaskan Beoufort and Eastern Chukchi Seas [6].

2.3 Engineering Characteristics of Unconsolidated Deposits

2.3.1 Soil Types Encountered

Figure 11 shows a distribution of bottom sediments in the Beaufort Sea shelf, taken from Reference [8]. Soil classification is by "mean sediment size", which is interpreted as the particle diameter below which 50% of any sample by weight is smaller (D_{50}). It can be seen that particle sizes ranging from gravels to clays were identified, with silts generally predominating at smaller depths and clays at greater depths. A partial map of sand and gravel resources in the Beaufort Sea is shown in Figure 12. More comprehensive information on sand and gravel resources is presented in figure 21 in Reference [1] and in Ref.[8a]. These data show surface deposits and do not necessarily reflect soil conditions important for foundation design, since the Holocene deposits shown in the figures could be quite shallow. Some specific information is shown in the soil profiles in figure 13, which were taken in a notherly direction in Prudhoe Bay. An idealized schematic profile is shown in figure 14. In general the data indicate that conditions tend to be quite variable and cannot be predicted in the absence of site specific data and that pockets of soft clayey silts and relict permafrost may be encountered within layers of more competent material. The spacing of exploratory borings must therefore be close enough to detect pockets of soft soil. As previously noted and shown in figure 14, there is a general trend for the thickness of the Holocene deposits, which cover the more competent Pleistocene deposits, to increase with increasing water depth.

2.3.2. Strength Characteristics of Soils

It is in general dangerous, and perhaps undesirable to try to generalize available information on subsurface conditions, since the soils in each individual location have their own unique

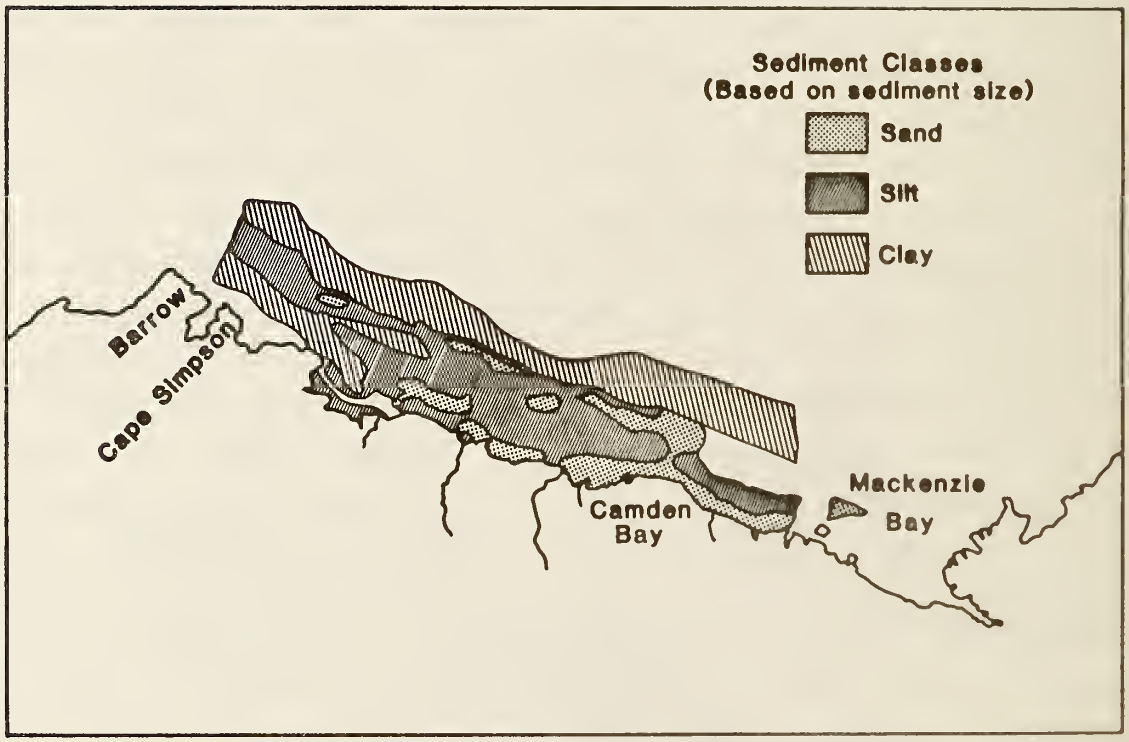


Figure 11. Distribution of Bottom Sediments on the Beaufoet Sea Shelf [8].

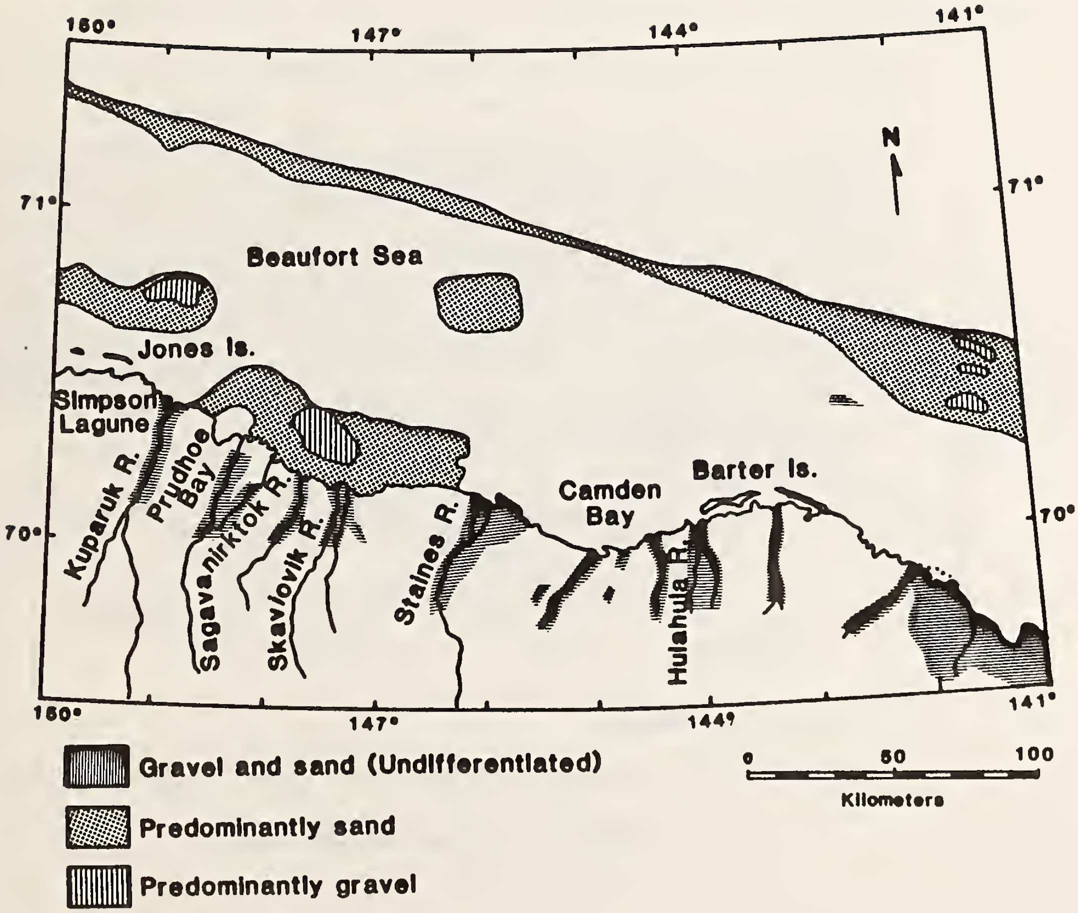


Figure 12. Sand and Gravel Resources on the Beaufort Sea Shelf [1].

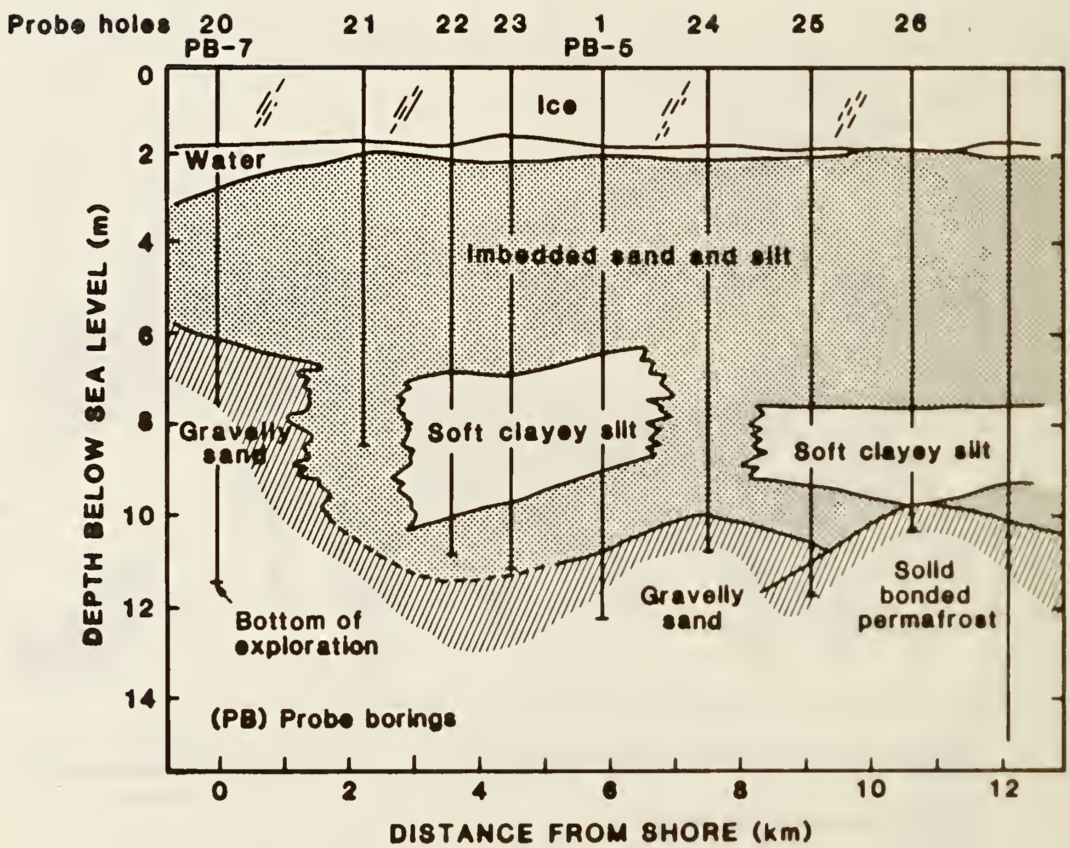


Figure 13. Soil Profiles in the Prudhoe Bay Area [11].

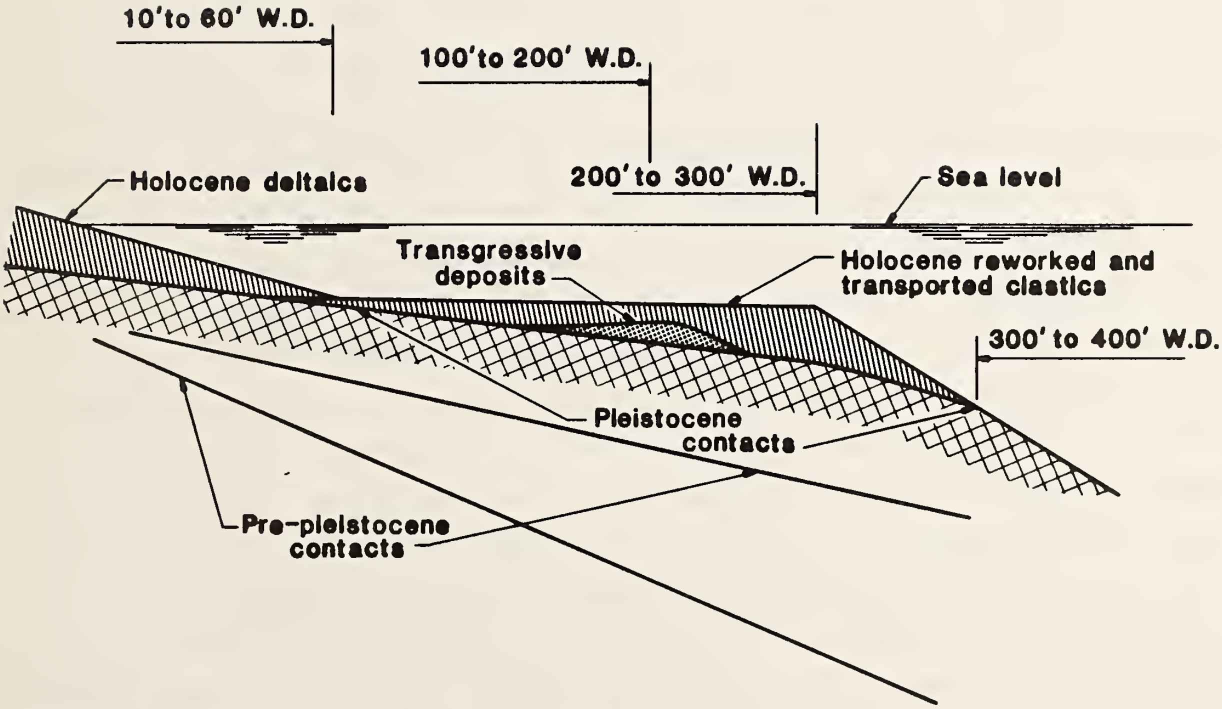


Figure 14. Idealized schematic profile of Unconsolidated Deposits in the Beaufort Sea Shelf [11].

characteristics and geological history which need to be explored and assessed, and the soils on the Beaufort and Eastern Chukchi shelves tend to be quite variable. However for the purpose of this report the trend of available data can give some insight into unique conditions existing in the region and problems that may be encountered with gravity structures.

Wang et al. [9] studied a wide variety of silts which seem to predominate in that region. Actually the soils they studied include soils which by the Unified Soil Classification (ASTM D2487)[10] would be classified as clays because of their high Plasticity Index, however the authors point out correctly that the engineering properties of these soils resemble those of silts, rather than clays. Figure 15 shows upper and lower bounds for shear strength and excess pore water pressures as a function of shear strains obtained in consolidated undrained triaxial compression tests. In the figure, σ_1 = major principal stress, σ_3 = minor principal stress, σ'_c = effective confining pressure at onset of test (consolidation pressure) and u = excess pore water pressure. The upper bound of shear strength is for the densest samples. It can be seen that the soil is dilative and shear strength increases with shear strain. The lower bound is for the loosest samples which are somewhat contractive but their shear strength is stable and does not decrease with increasing shear deformations. Figure 16 shows the range of undrained shear strengths with depth for Beaufort Sea silts and figure 17 shows overconsolidation ratios as a function of depth obtained from odometer tests. In figure 16, TXUU and TXCU are undrained unconsolidated, and undrained consolidated triaxial tests, respectively.

The data in the figures should be viwed with caution, because they are from laboratory tests and are probably affected by sample disturbance. Sample disturbance effects are particularly great for heavily overconsolidated samples. Nevertheless, certain

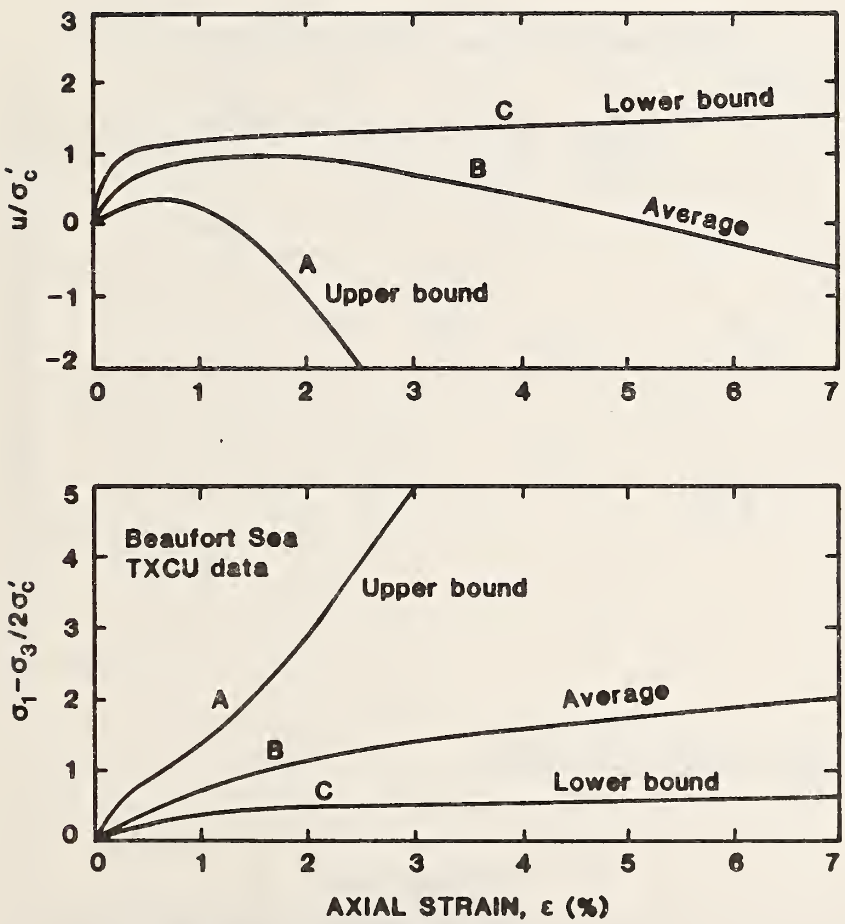


Figure 15. Normalized Shear Strength and Excess Pore Water Pressures vs. Shear Strain Obtained in Undrained Consolidated Triaxial Tests of Beaufort Sea Silts [9].

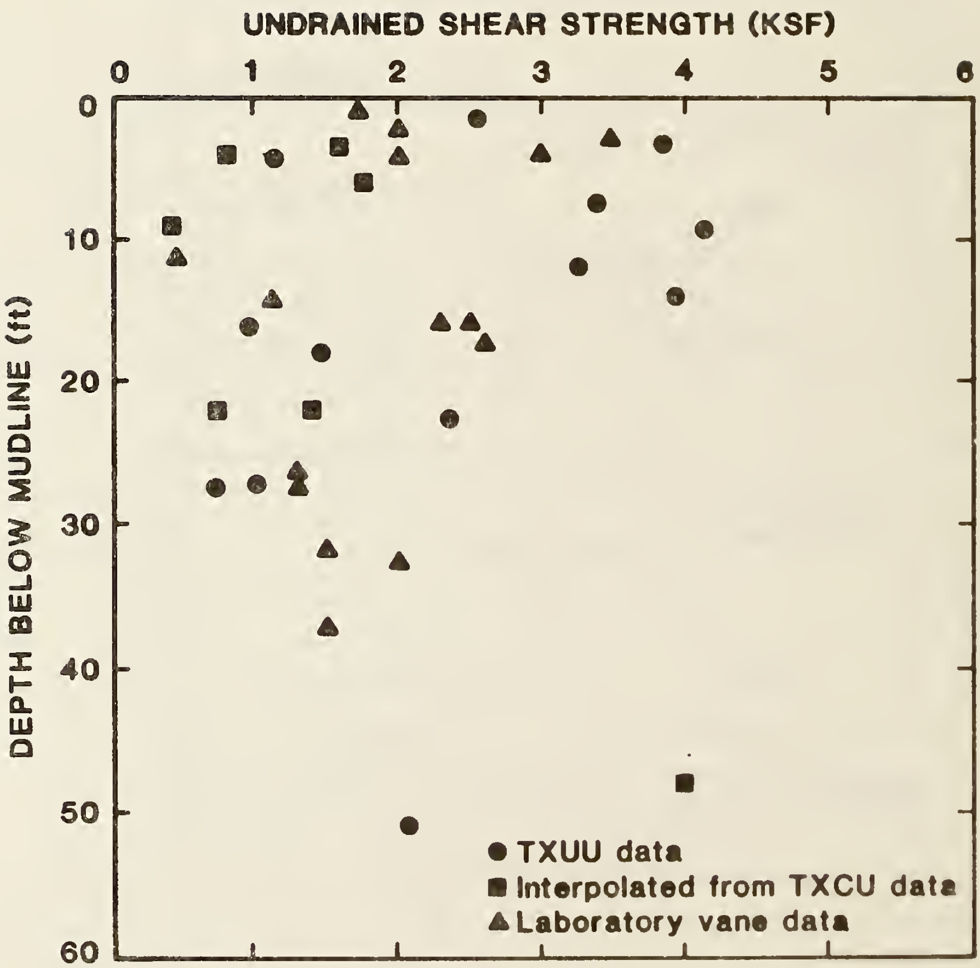


Figure 16. Undrained Shear Strength Profile for Beaufort Sea Silts [9].

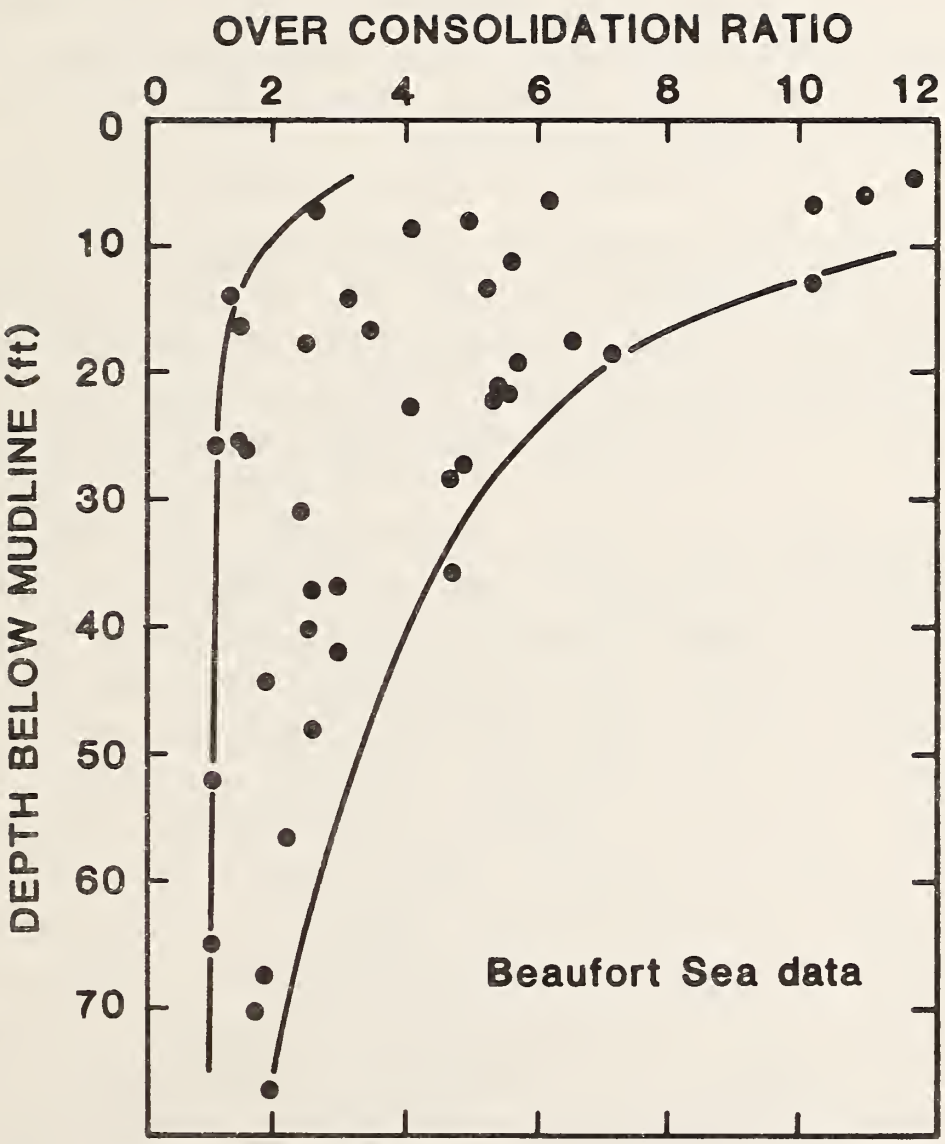


Figure 17. Overconsolidation Ratio Profile for Beaufort Sea Silts [9].

trends emerge from the data: (1) the soils are heavily overconsolidated, particularly at a shallow depth, with a considerable scatter of the overconsolidation ratio at a shallow depth. The authors attribute this overconsolidation to the effects of past freezing and call it "apparent overconsolidation". The origin of the overconsolidation may be responsible for the variability of the data. However some of that variability may also be caused by sample disturbance.

(2). While there is considerable scatter in the shear strength at a shallow depth, with some very high values, there is a trend, for depths greater than about 10 ft, for the shear strength to increase with depth. There is little doubt that the high strength values at shallow depths and their scatter correspond to the high and scattered values of overconsolidation ratios discussed under (1). If this is the case, strength values should not be relied on in design, because of their variability, and also because overconsolidation effects could be eliminated as a result of disturbances and large deformations which can cause a reduction in confining pressures. (3). The normalized shear strength values shown in figure 15 are for strains of up to 7%, and for the A curve strains up to 3%. The tests were not carried to a large enough strain to achieve a steady state strength, and probably could not have been carried to a large enough strain by triaxial testing. It may be that at larger strains the A curve would drop to a steady state (residual) shear strength which is considerably lower, particularly in overconsolidated samples. The tests also provide no information on cumulative displacements that may result from many strain cycles in one direction. Such information is of crucial importance for the assessment of the long term response of offshore structures subjected to ice loading.

Since it is difficult to obtain truly undisturbed samples and to measure the shear strength in situ, it is important to provide information that will enable us to estimate the shear strength from soil parameters that can be measured in situ. Figure 18 gives the range of correlations between the unconsolidated undrained shear strength and the natural water content W_n, which is a measure of in place density and is not as sensitive to sample diturbance as triaxial test results. Figure 19 provides information on the range of shear strengths as a function of depth which would be obtained for normally consolidated deposits. This would be a conservative lower bound, since the deposits are overconsolidated.

An example of soil profiles in Harrison Bay presented by Bea [11] is shown in figure 20. Note that there is a variety of sedimentary deposits, considerable variation of undrained shear strength with depth and location, and that the shear strength of the silts tends to be higher near the surface.

The preceeding discussion applies to soil deposits below the depth which is affected by seasonal disturbances resulting from ice gouging and wave action. The bulk of the observed gouges are less than 1m deep [6]. However, there are deeper gouges, and it also must be assumed that the ocean floor below the bottom of the gouges is disturbed by shear failures associated with gouging. In the near shore area, the gouge marks are frequently obliterated by wave action, but the sea floor is disturbed all the same. Thus it is reasonable to assume that down to the 50m isobath the seafloor deposts have been disturbed by gouging to a depth of 1, and possibly 2m. As a result the soils are likely to be loose (or soft) and their density non uniform. This factor must be taken into consideration, in addition to the gouges which are physically present.

2.3.3 Summary of Engineering Characteristics

In the preceding sections of chapter 2, the characteristics of the unconsolidated deposits on the Alaskan Beaufort and Eastern Chukchi shelves have been discussed. The engineering implications

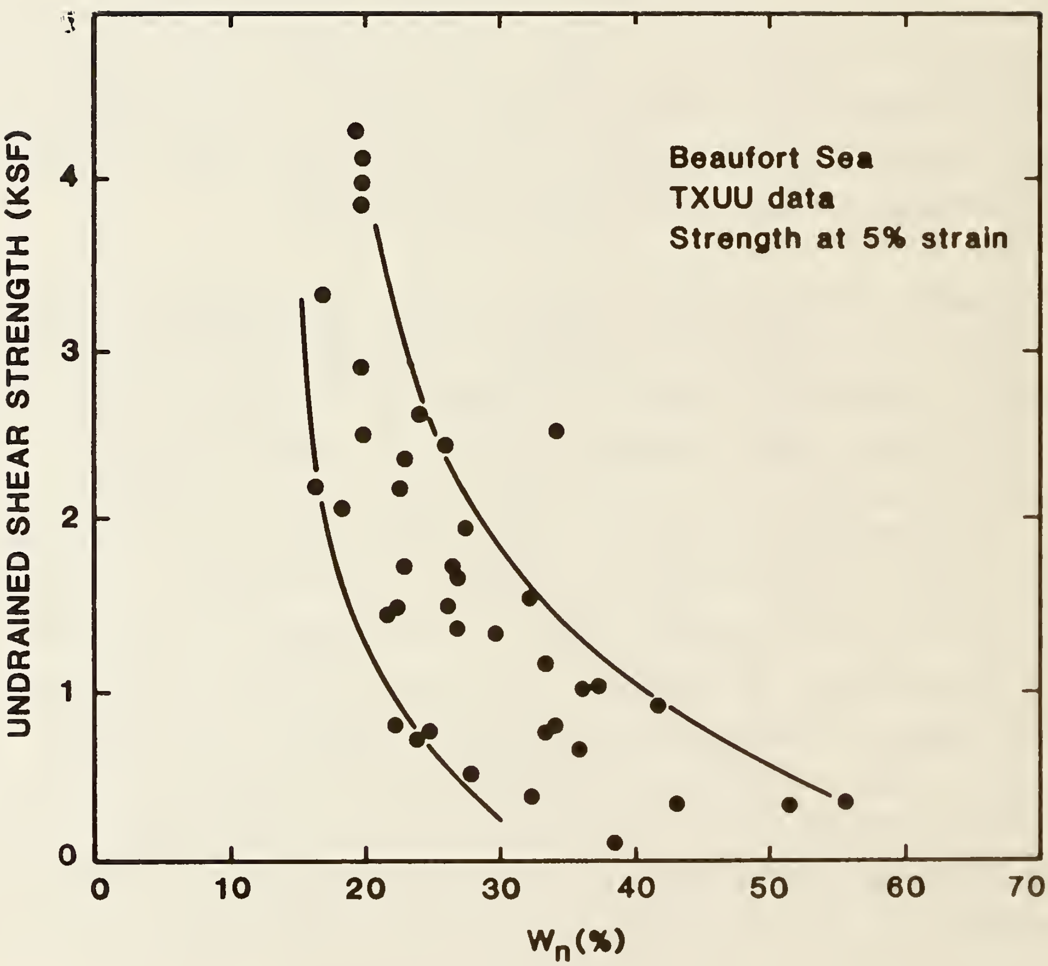


Figure 18. Water Content vs. Undrained Shear Strength for Beaufort Sea Silts [9].

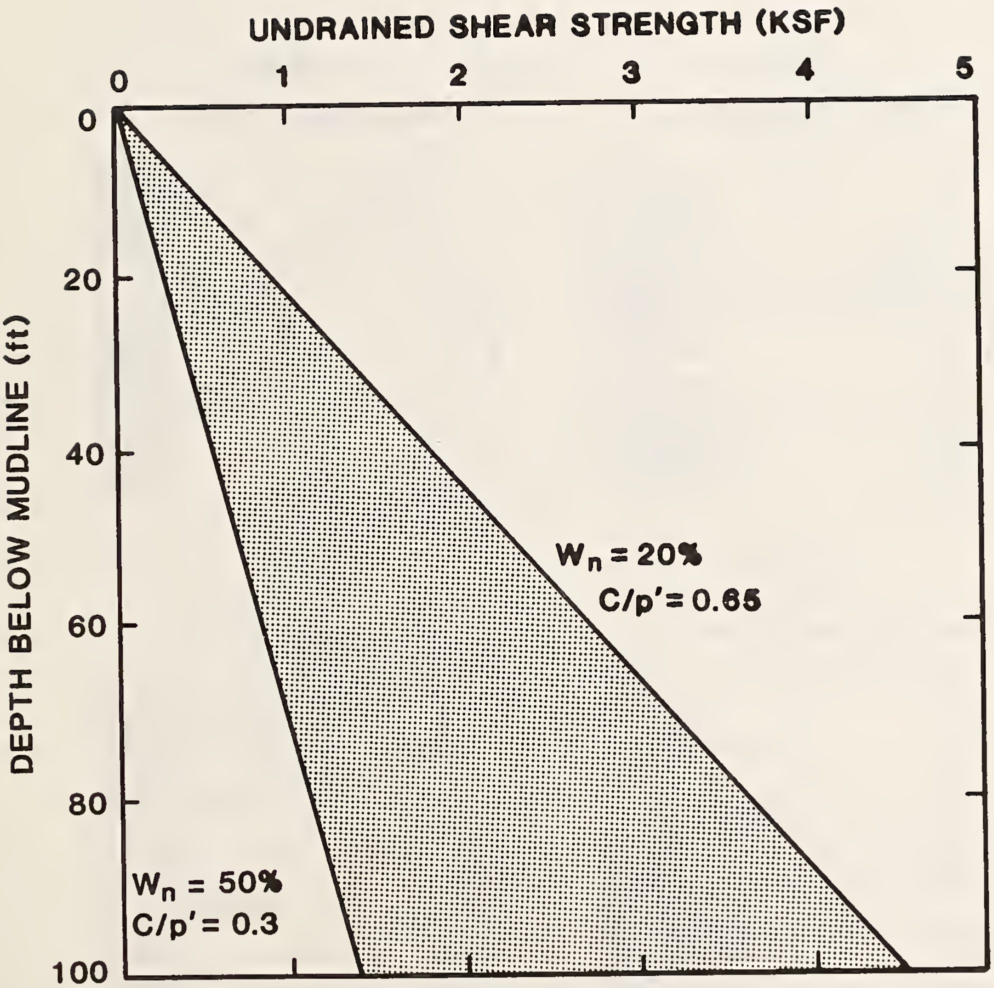


Figure 19. Profile of Expected Range of Undrained Shear Strengths for Normally Consolidated Alaska OCS Silts [9].

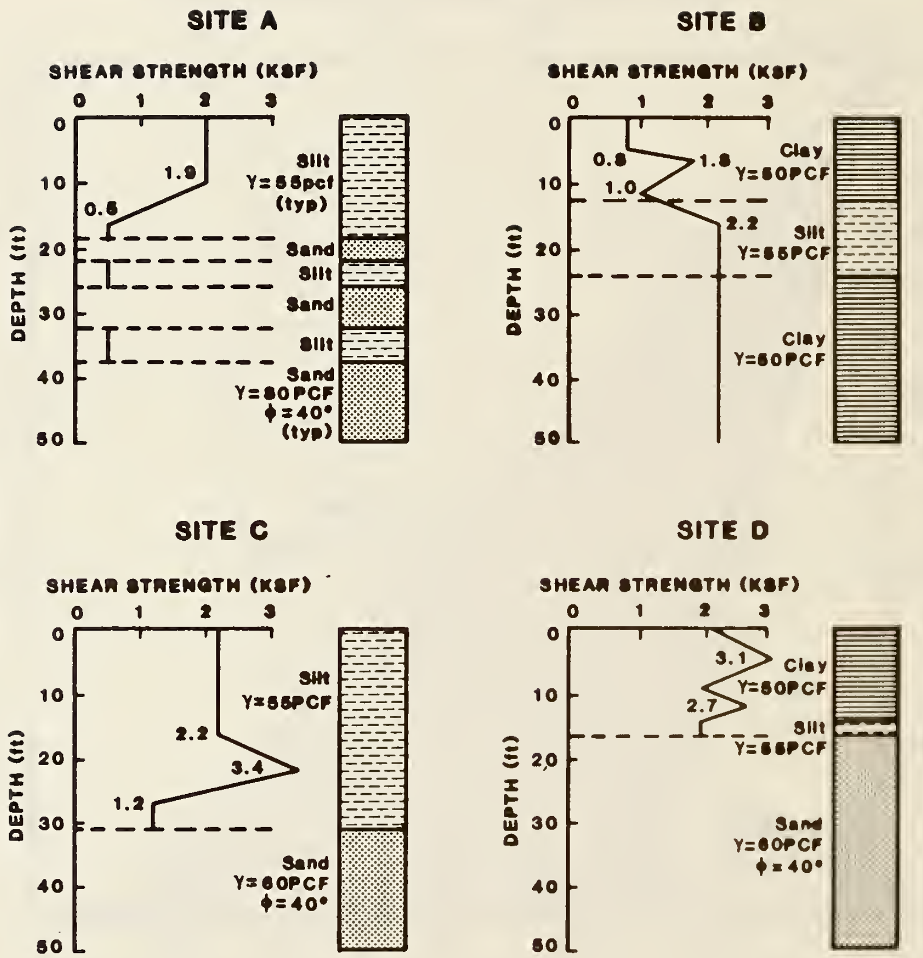


Figure 20. Soil Profiles in Harrison Bay [11].

of these characteristics are summarized hereafter:

(1) Evidence of Instability (2.2.2).

While the youthful landslide terranes tend to be close to the shelf break, and thus beyond the depth range considered in this report, their absence should not be taken for granted. If creep is taking place along a weak layer which is located at some depth, it may not be possible or desirable to prevent the gradual diplacement of structures supported on mat foundations. Flexibility or adjustments may have to be provided to accommodate anticipated displacement. Scour and erosion in coastal areas, including a rapidly receeding coastline, strudel scour near river deltas, and landward migration of barrier islands may also pose significant engineering problems in shallow water depths.

(2) Seismicity (2.2.3)

Except for the limited area outlined in figure 5, there seems to be no significant seismic risk. For the proposed Magnitude 6.5 design earthquake, liquefaction of granular deposits may pose a risk, particularly for artificial berms and Holocene sand deposits which may be naturally loose and difficult to compact. High plasticity silts are probably not liquefiable, but low plasticity silts may pose some risk.

(3) Relict Permafrost (2.2.4)

Relict permafrost, where present, poses a troublesome and unique engineering problem. It can be seen from figure 21 (from Ref.[11]), that the problem occurs over a wide area. Since offshore platforms and exploration and production wells are likely to change the thermal regime underneath the mat foundation, at least partial thawing may be unavoidable. The effect would be large settlements and loss of subsoil shear strength. Mitigation or prevention of these effects must be a design consideration.

(4) Shallow Gas and Gas Hydrates (2.2.4)

The pressure exerted by shallow gas deposits may considerably weaken the shear strength of soil deposits. The presence of

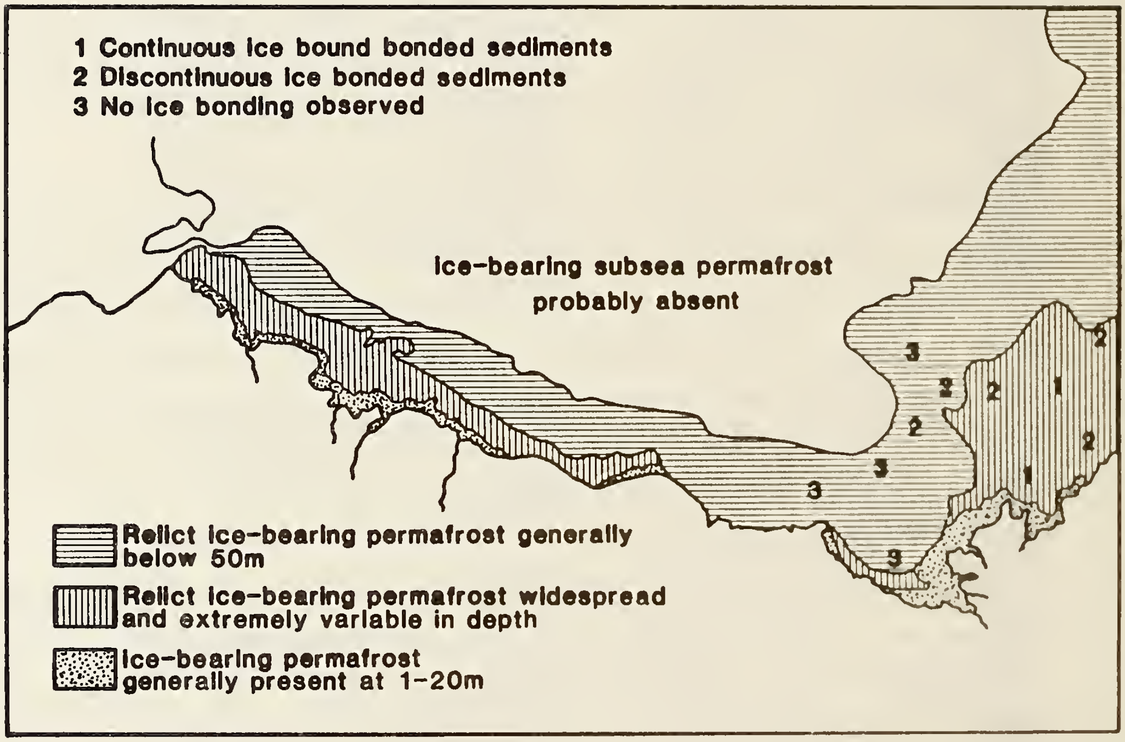


Figure 21. Provisional Map of Subsea Permafrost Distribution [11].

gas also causes sample disturbances which make it difficult to assess in situ soil characteristics. In addition, shallow gas can cause problems during drilling operations which may have to be considered. Gas hyderates may cause serious problems when changes in the thermal regime cause thawing of the enclosing permafrost. In accordance with available data, gas hydrates may not be encountered landward of the 60m isobath.

(5) Ice Gouges (2.2.4)

Ice gouges, and the associated irregularity of the geometry and strength characteristics of the seafloor deposits must be considered in the design of mat foundations. Resulting engineering problems are partial contact of prefabricated mat foundations and uneven stress distribution even when under-filling or low berms are used. Gouges are likely to be encountered landward of the 50m isobath and are most severe between the 15 and 45m isobaths (the Stamuki zone). The dominant orientation of the gouges is parallel to the isobaths.

(6) Shear Strength of Soils (2.3.2)

The Holocene deposits are generally loose and quite variable in characteristics. The Pleistocene silts, which underly much of the area considered in this report are by and large competent. The denser silts tend to dilate under moderate shear deformation and their shear strength increases as they dilate. To a depth of 10 - 12 ft the silts are heavily overconsolidated, probably due to effects of past freezing. The soil conditions are quite irregular and lenses of much softer material within the silt layers are not uncommon.

(7) Sand and Gravel Resources (2.3.1)

There are some sand and gravel deposits that were mapped. In general, the rate of sediment deposition in the river deltas is quite low and there are many areas with no convenient access to sand and gravel deposits.

3. DESIGN LOADS

3.1 General

The arctic offshore environment produces unique environmental loads, Including the thrust exerted by ice and dynamic loads generated by collisions with large bodies of floating ice, and loads that can result from thawing of relict permafrost or from cycles of freezing and thawing. The optimal engineering solution is not always construction of an immovable object that will be strong enough to resist the loads. Sometimes it may be possible to reduce loads by permitting the structure to yield or move, to avoid loads by protective measures, or to provide the means to augment the resistance of structures if such measures should be needed in the future.

Not all loading conditions are relevant to the design of mat foundations. For instance local stresses induced by ice loads are much higher than the average stress on a large area which produces the global ice force. Wind and wave impact loads may be very significant in the design of equipment and structural components, but they do not produce critical loads for foundation design.

3.2. Loads Acting on the Structure

(1) <u>Gravity Loads:</u>

Gravity loads include the weight of the structure, any ballast applied by flooding of cells or by other means to increase the gravity load, operational loads associated with drilling and storage, and any ice or snow loads that may accumulate.

In the design of mat foundations, The minimum gravity load is generally of interest. If minimum loads are determined, consideration should be given to the probability that gravity loads may be reduced, and to the possible effect of tidal uplift forces exerted on adfrozen ice.

(2) Wind loads:

Basic wind speeds near the Alaskan Beaufort and Eastern Chukchi coast as specified in ANSI A58.1-1982 [13] are between 90 and 100 miles per hour. For foundation design, wind load effects on the structure per se are not significant. However, wind loads acting on ice floes or rubble piles which bear against the structure can exert significant forces and are to a significant extent responsible for the fluctuation of ice forces with time. The maximum global ice forces used in the design of the foundations, as calculated by the present state of the art, depend on the strength, failure mode and sometimes on the velocity of movement of the ice, rather than the magnitude of the wind forces acting on the ice. Even though wind forces are not used in these calculations, they may be ultimately responsible for the movement of ice floes which produce critical collision forces. The dominant wind direction during late summer and early fall is Northeast, but high Westerly winds occur during storms [1].

(3) Waves, Tides, Currents, and Storm Surges:

Surface waves, which are restricted to the open water season are generally small because of the limited fetch resulting from offshore sea ice. They have generally 2 to 3s periods and heights less than 1m. The maximum wave heights reported in the Beaufort Sea were over 9m and at Pointe Barrow 6m and occurred during summer storms [14]. Lunar tides along the Beaufort coast of Alaska are of the order of 0.5m, but low barometric pressures and high Westerly winds which prevail during exceptional storms can cause storm surges which are 3m above mean sea level [1]. At Barrow and in the Chukchi Sea these surges can reach 3.5m. Current patterns and velocities are shown in figure 22.

(4) Seismic Forces:

As noted in chapter 2, much of the area under consideration is

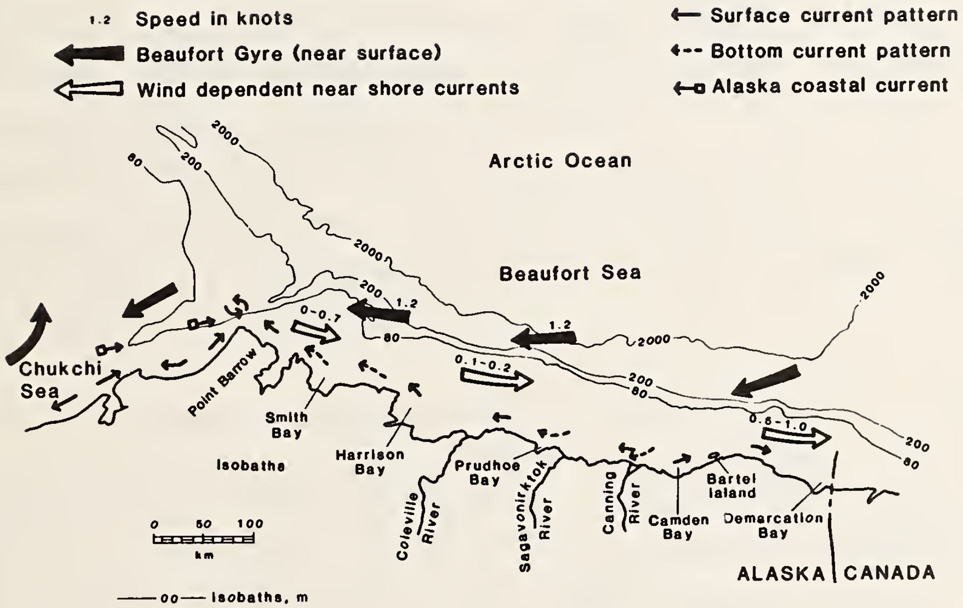


Figure 22. Current Patterns Recorded in the Beaufort and Eastern Chukchi Seas (from References [1] and [14]).

considered aseismic. Lateral forces generated by seismic loads would probably not exceed the ice forces, however there are two instances which under certain circumstances could be critical for design: interaction between the structure and the surrounding fast ice during a seismic event; and liquefaction of soil deposits supporting the structure.

(5) Ice Loads:

The most critical loading condition for mat foundations for Arctic offshore structures is a combination of minimum vertical and maximum lateral load. The maximum lateral load, in turn, is likely to be the global force exerted by the most critical ice loading condition. The conditions determining the magnitude of ice loads are difficult to define and predict with any degree of certainty. Thi is due to the difficulties in predicting seasonal ice regimes, the problems encountered in defining the strength of ice, and the many different ice features which can cause critical loading.

Ice regimes and features were qualitatively discussed in Section 2.2.4 (3). API Bulletin 2N [15], defines the following ice morphology features: 1. Sheet ice is the fast ice in protected bays and lagoons; 2. Rafted ice are two or more sheets of stacked ice which rapidly consolidate into a single unit and may appear as smooth-surface floes on either side of first year ridges; 3. First year ridges are ice ridges formed by compressive forces in a single season; 4. Rubble features (rubble piles) are grounded ice ridges, and can also form around offshore structures; 5. Multiyear floes are bodies of floating ice which survived more than one season. They have weathered rounded and consolidated sails and relatively solid keels about 3-4 times the sail height. They can incorporate ridges and rubble piles and in many instances have length and width dimensions in excess of 300m. 6. Arctic Ice pack is the floating pack ice normally present seaward of the 60m isobath which may occasionally invade offshore structure sites during the summer season; 7. Natural ice islands are tabular icebergs calved off the Ward-Hunt ice shelf, which can be greater than 250 km^2 and can have a keel thickness of 45m. They tend to drift with the Baufort Gyre and fragments of the islands occasionally invade shallower waters. The last recorded invasion was in 1973 [16]

Some quantitative data for the Beaufort Sea were presented by Vaudrey [17]:

Ice Coverage: Early October to mid July

Ice Thickness:		Rubble Piles:	
Sheet Ice	2m (6-7 ft)	Consolid. Thick.	3-4m
Rafted Ice	5-6m	Max. Height	11-13m
First-Year Ridge	8-9m	Max.Length	300m
Extreme	> 30m		
Multiyear Ridge	13-20m		
Multiyear Floe	6-8m		

Ice Movement (max)	
<10m Isobath	3-6m/h
10-20m Isobath	30-150m/h
>20m isobath	300m/h
Open Water	2-4km/h

The mechanical properties of sea ice are complex and will not be discussed in detail. Unconfined compressive strength as obtained in laboratory tests depends on strain rate, crystal orientation and crystal grain shape and size. Confined compressive strength is substantially greater than unconfined strength. Tensile strength, flexural strength,shear strength, and Young's Modulus vary with the brine content, v_b, which is defined by the following equation:

$$v_b = S\ (0.532 - 49.185/T) \qquad \text{... (eq 1)}$$

where: S = ice salinity in parts per thousand (ppt)

T = ice temperature in °C

v_b = brine volume in ppt

Typical data obtained from Ref [16] are shown in figure 23. Ice strength and other physical characteristics change in a complex way with temperature, with the strength increasing with decreasing temperature, but also brittle behavior becoming more pronounced as temperatures fall below -10° C. This affects the correlation of ice strength to the size and shape of the contact area between ice and the walls of an offshore structure in a complex way. This problem was discussed by Bruen et al.[19]. Correlations between the contact area and pressures from first and multiyear floas developed in Reference [19] are shown in figure 24. Note that these curves are derived from a specific set of assumptions. The correlations may be different if any of the assumed parameters of the problem are changed.

Design forces are stipulated in Reference [15]. Watt [18] discusses two types of offshore structures which fall within the scope of this report: Wide indenters, which are wide structures with vertical walls, and cone structures. Wide indenters are defined as structures with waterline diameter to first year ice thickness ratios of 20 : 1 or more, and with sides which are either vertical or deviate from verticality by no more tha 20°. Critical global ice pressures are generally controlled by ice failure and depend on the characteristics of the ice feature exerting the pressure. The critical load on a wide indenter could be controlled by the "breakout" from an ice sheet, or the crushing into a floe in which case the strain rate would have a critical effect, or the kinetic energy of a colliding ice feature. Relatively thick ice features such as ridges could fail by flexure reather than crushing. Cone structures can limit the ice load by causing the colliding ice feature to ride up on their side and fail in flexure, rather than by crushing. However this failure mode can only occur if the adfreeze bond between the structure and the ice is limited. This can be accomplished by special coatings or by heating the sides of the structure. Ice Pressures against conical structures can be increased by the

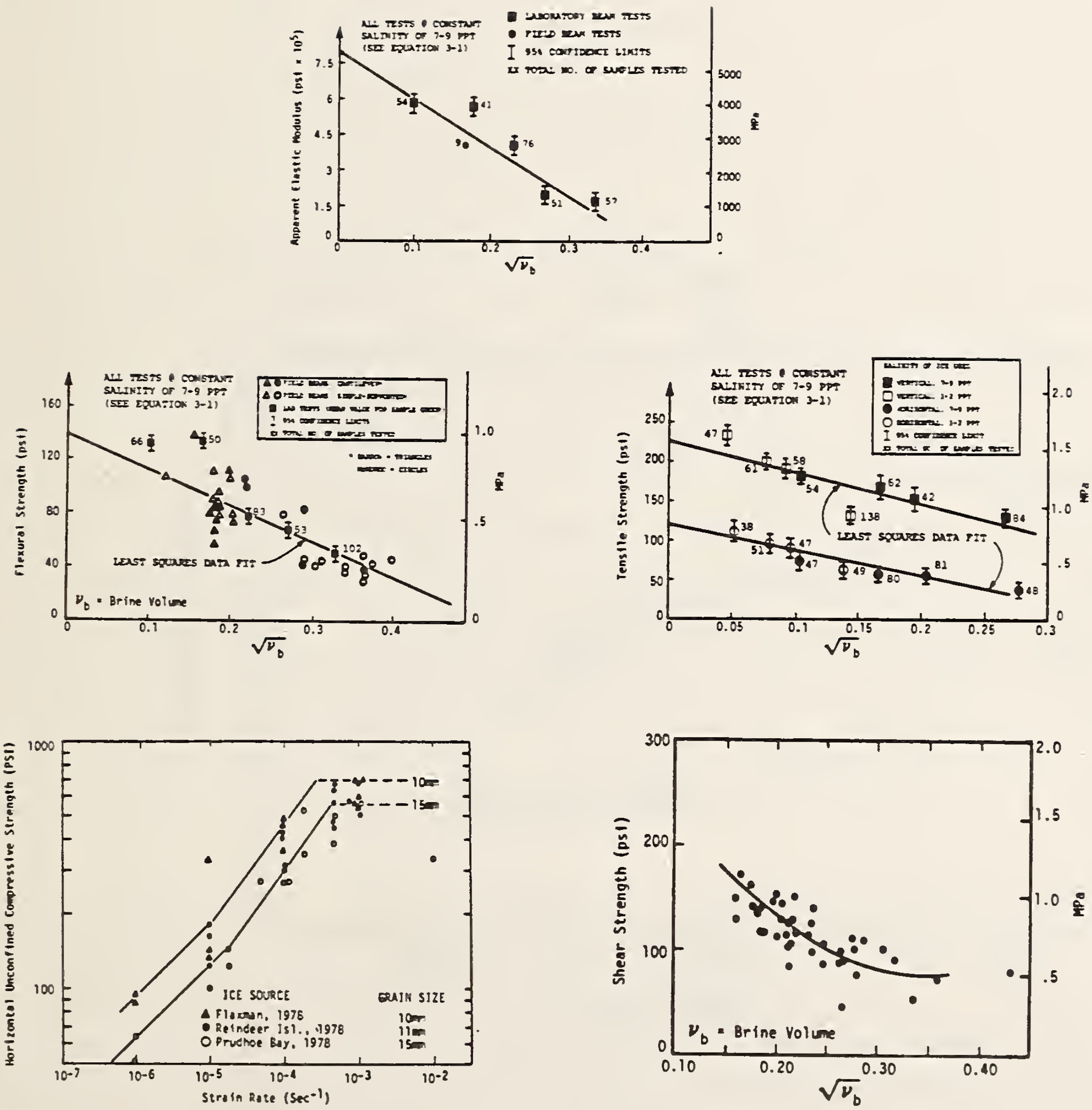


Figure 23. Typical Strenrgth Characteristics of Sea Ice [15].

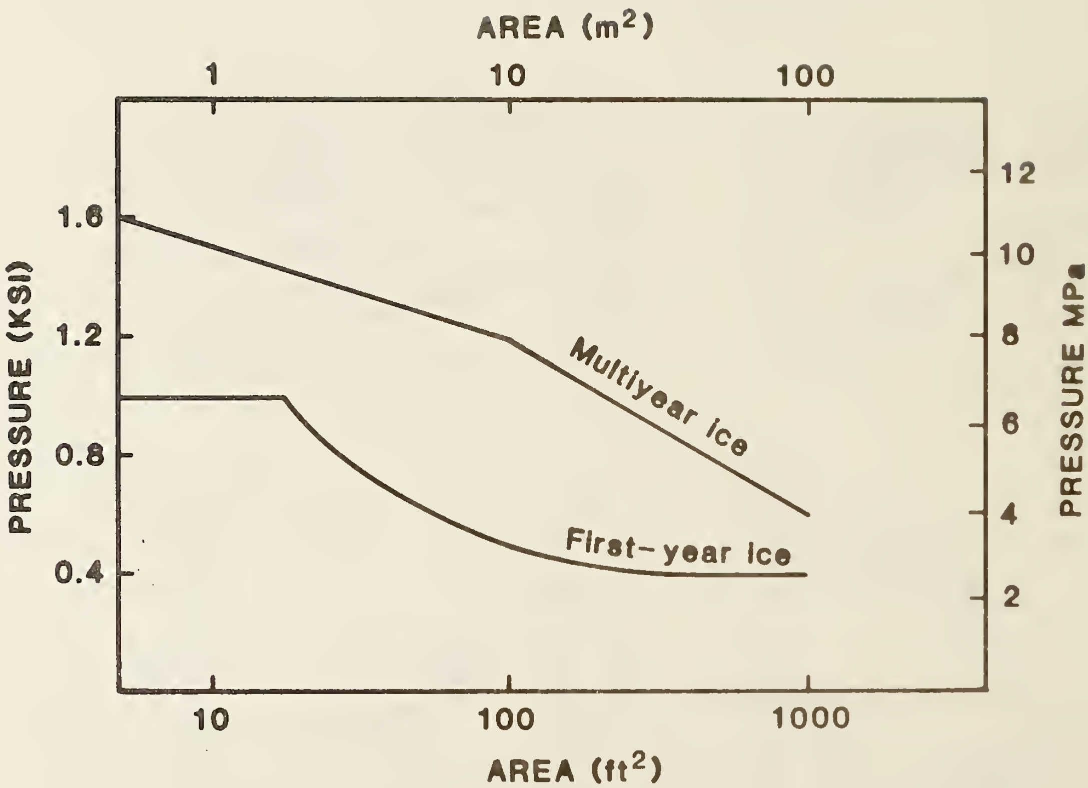


Figure 24. Correlations Between Ice Pressures and Contact Area [19].

rideup of ice against the sides of the structure.

The implications of the effect of different ice environments on global design loads are illustrated by an example presented by Bea et al. [20]. Figure 25 shows a plot of global ice forces against a large indenter as a function of the mean recurrence interval of the loads. The vertical scale is not labeled since the magnitude of the actual force would depend on the diameter of the structure. Note that the extreme event of a collision with an ice island is judged to have a low probability of ocurrence, and that actual observations at Hans Island [21] indicated that estimated loads associated with this event may be much too conservative. Figures 26 and 27, which are also taken from Ref[20], show design loads estimated for 120 to 180m wide exploration and production structures as a function of water depth. In the figures UB is an upper bound estimate and LB is a lower bound. A 25 year return interval is taken for the exploration structure in figure 26, and a 100 year return interval for the production structure in figure 27. Note that the estimated global ice loads for the upper bound estimate increase with depth. These loads are calculated for collisions with multi year floes and are sensitive to the strain rate. As the water depth increases, the velocities of the ice movement, and thus the strain rates increase.

While the global loads previously dicussed are extreme loads which control the required load capacity of foundations, It is also important to consider long term and cyclic loads which could cause cumulative displacements, and the dynamic characteristics of loads which could cause excess pore water pressures. Figures 28 to 30 are taken from Ref [21] and show a winter season ice pressure record from the Beaufort Sea, a calculated ice impact load signature, and a typical winter ice loading histogram. The cyclic loads in figure 28 primarily reflect storm wind effects on the ice canopy (low frequencies) and local ice

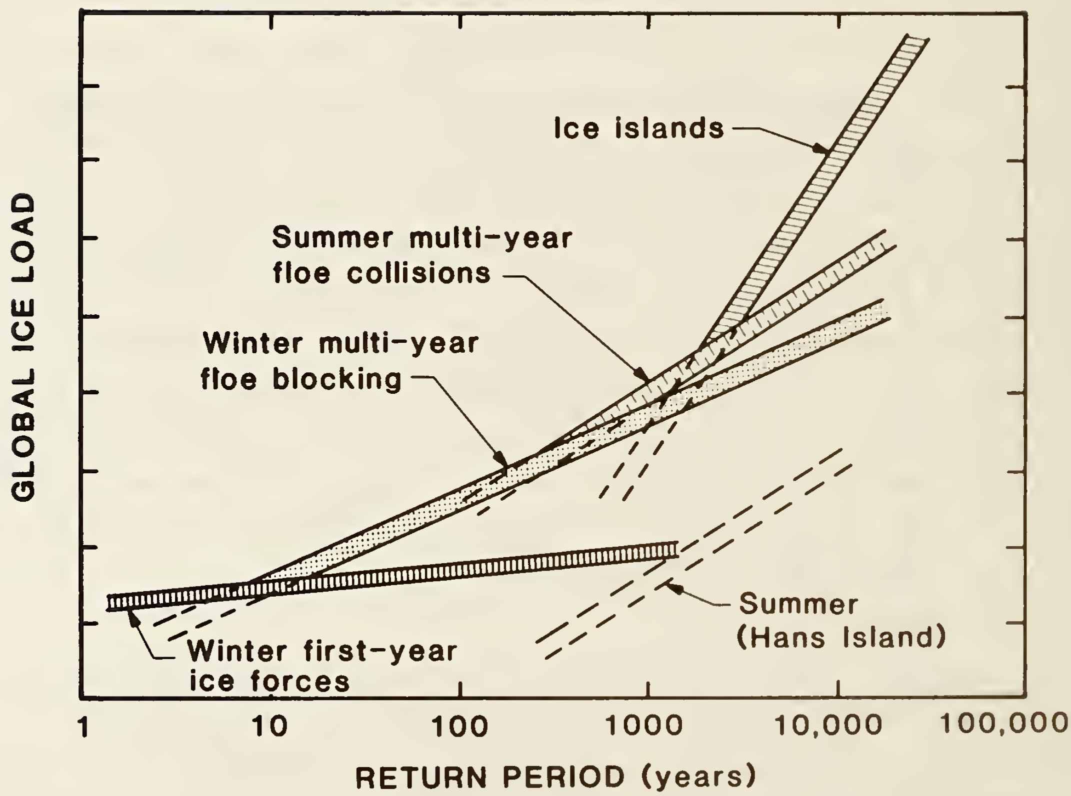


Figure 25. Global Ice Forces as a Function of Mean Recurrence Interval [20].

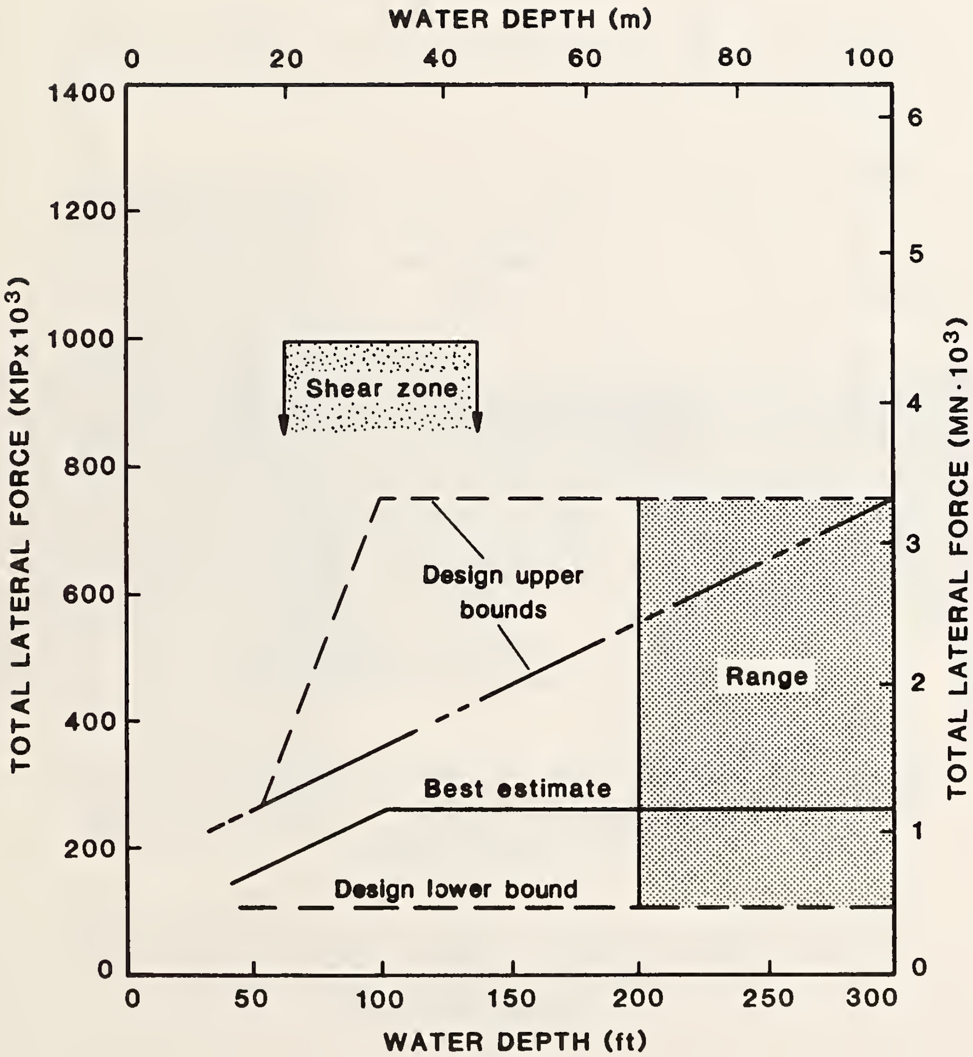


Figure 26. Estimated Global Ice Forces Against a 120-180m Wide Exploration Structure as a Function of Water Depth [20].

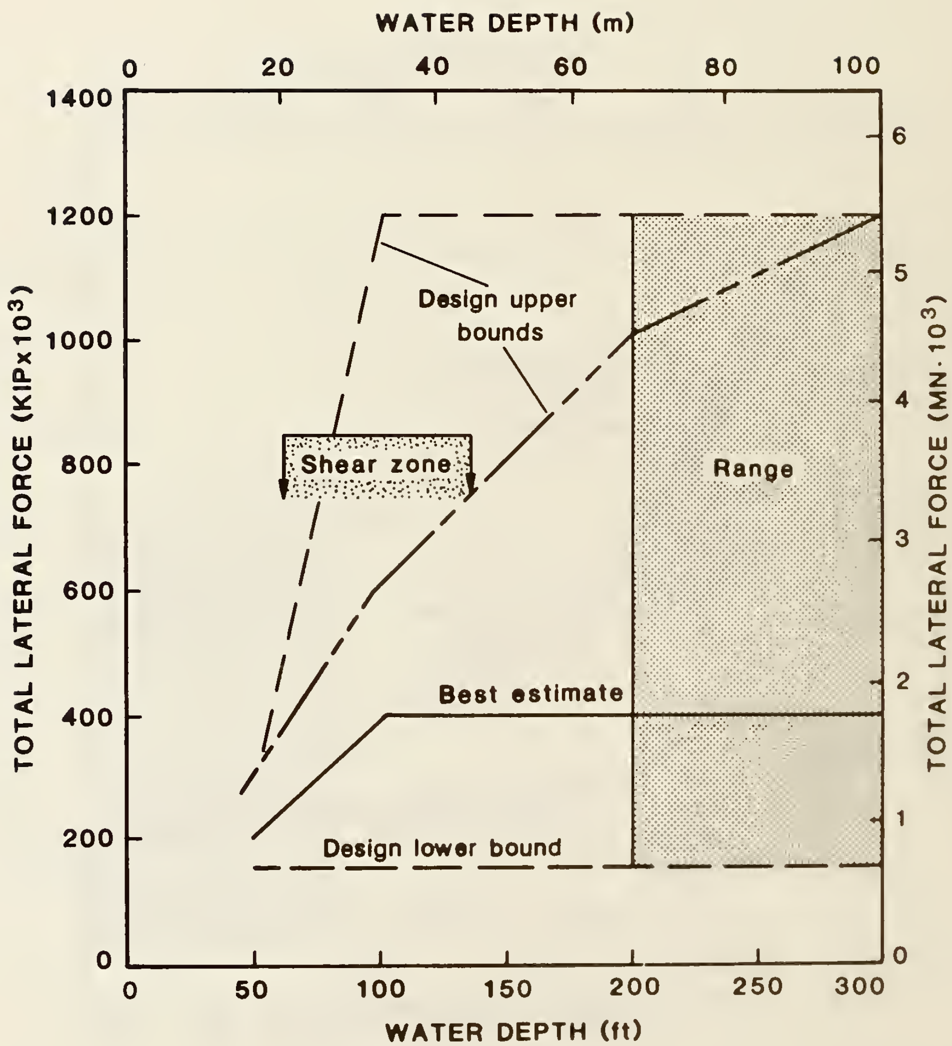


Figure 27. Estimated Global Ice Forces Against a 120-180m Wide Production Structure as a Function of Water Depth [20].

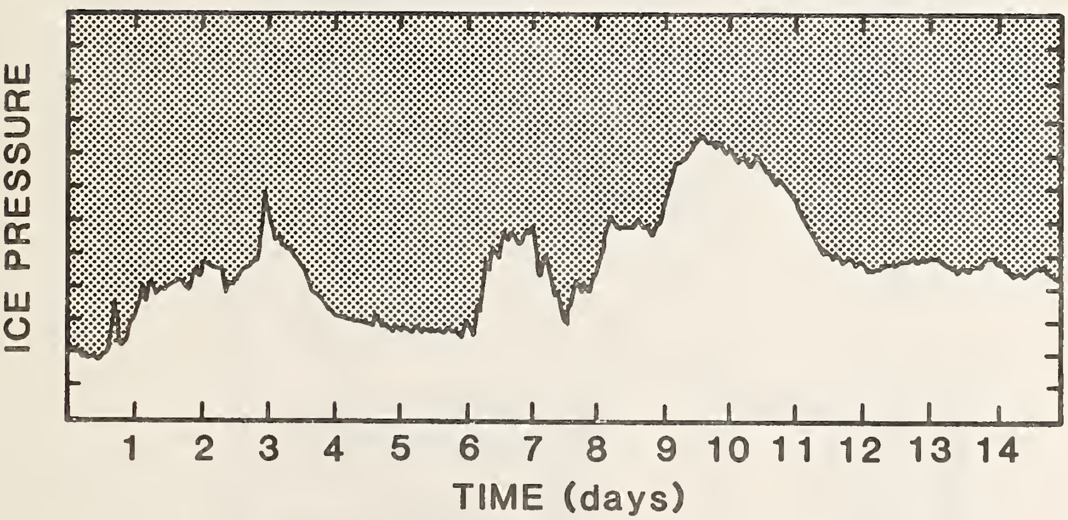


Figure 28. Winter Season Ice Pressure Record for the Canadian Beaufort Sea [21].

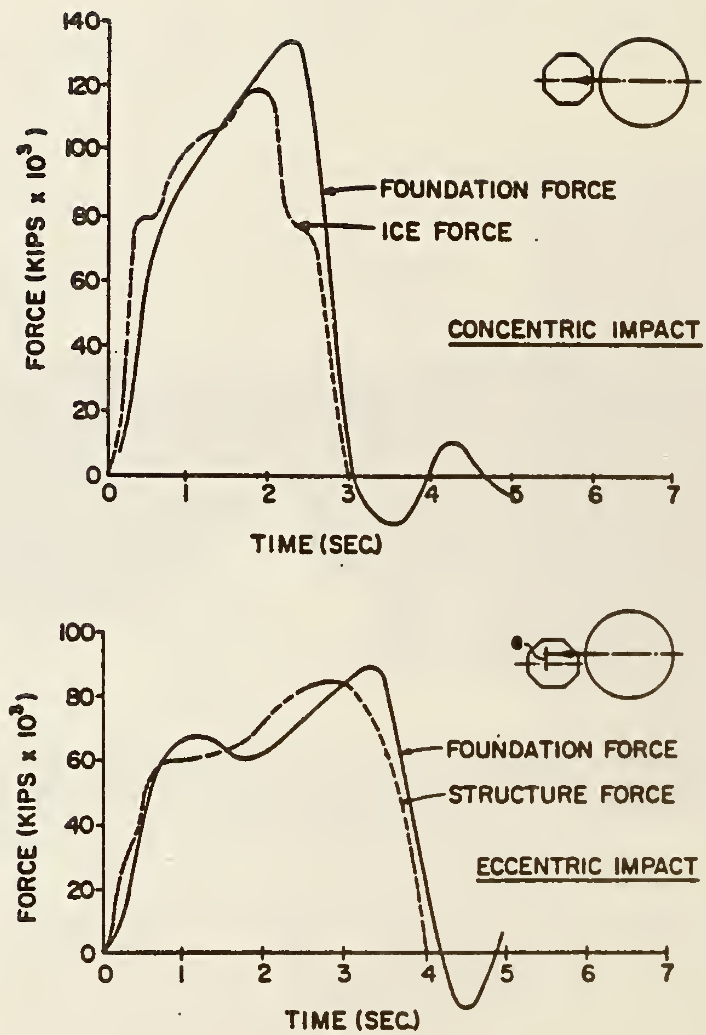


Figure 29. Calculated Force-Time Histories for Large Ice Feature Impact [21].

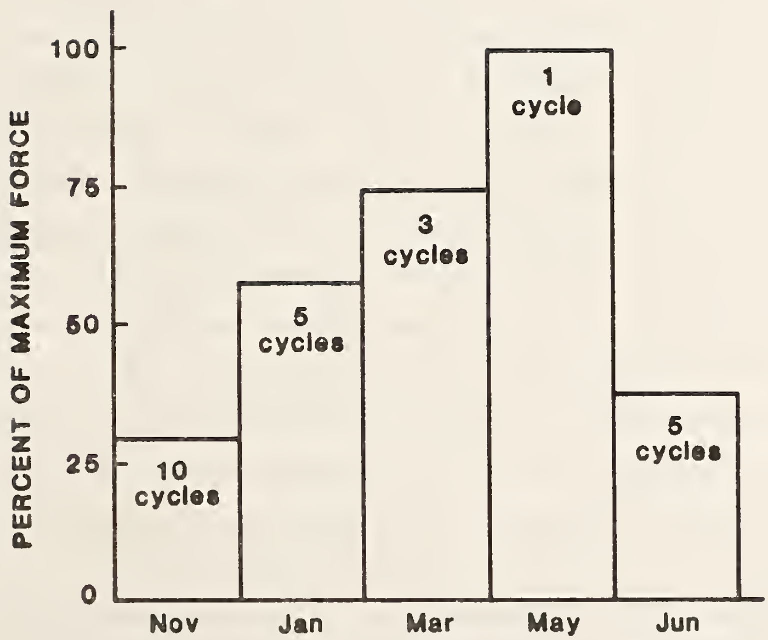


Figure 30. Typical Winter Ice Loading Histogram [21].

fracturing (high frequencies). It is important to note, that the directionality of the ice forces discussed above is not random, since it is affected by prevailing wind and current directions. If residual displacements from ice load cycles are anticipated, it is likely that these displacements will result in seasonal, or even cumulative annual lateral drifting of the structure. These lateral displacements must be considered in the design, particularly for permanent drilling and production structures. It should also be noted that the high frequency cyclic forces could lead to liquefaction or cyclic mobility of the underlying or core infill soils, particularly in the case of low density, hydraulically placed cohesionless soils.

(6) Special Environmental Loads:

Some special structural loads could be generated by the arctic environment: 1. Settlements caused by the thawing of permafrost or oil and gas removal could create voids under large, relatively stiff mat foundations, resulting in shear forces and bending moments in the mat. 2. Release of trapped or hydrated gases could in extreme cases induce uplift forces, followed by settlement. Trapped gases can also reduce the shear strength of supporting soils. 3. Soil erosion could be caused by wave and current action and by strudel scour (erosion caused by the rapid flow of melt water through openings in the ice sheet) and can result in soil removal at the fringes of mat foundations or gravel berms, causing high local stresses in the mat foundation or local stability failures. 4. Freezing could occur under a mat foundation as a result of changes in the thermal regime or as a seasonal phenomenon. Freezing will cause uplift forces, and subsequent thawing could leave voids under the mat foundation.

4. DESIGN LIMIT STATES

4.1 General

In accordance with accepted usage of the term, a "limit state" occurs when a structure fails to fulfill the functions or meet the conditions for which it is designed. In present design practice, two types of limit states are generally defined: "ultimate" limit states, which are associated with loss of life or major damage, and "serviceability" limit states which are associated with transient loss of function or minor damage. The objective of limit states design is then to assure that ultimate limit states, which are generally associated with strength, have a suitably low probability of occurrence, and that serviceability limit states, which tend to be associated with stiffness, not occur so often as to impair the functionality of the structure or unduly increase its maintenance cost.

The previously discussed approach uses two general categories of limit states, ultimate and serviceability, and generally provides more or less uniform safety margins against the ocurrence of these limit states, either by probabilistic code formates, or by more traditional design schemes (working stress or factored loads and resistances). This approach is too simplistic for the complex environment and the innovative technology associated with offshore structures in arctic regions.

By definition a limit state is a failure mode, and since each structural scheme tends to have its own failure modes, limit states are scheme dependent and can not always be generalized. Offshore technology in arctic regions is evolving rapidly and many different concepts are under consideration, some of which are designed to avoid, rather than resist loads. Furthermore, some of the loads that could be encountered during summer invasions of ice islands are of such a magnitude that it would be in many instances unfeasible to design adequate foundations. To provide adequate guidance for structural design , which would not unduly restrict the introduction of new, innovative emgineering concepts, criteria for required structural performance need to be stated in more fundamental terms.

4.2 Requirements for Safety and Serviceability

The requirements for safety and serviceability of the structure in place can be expressed as performance criteria:

1. Protection against loss of life and severe injuries.
2. Protection against major environmental damage
3. Protection against major property damage
4. Protection against minor environmental damage
5. Maintenance of efficient operation
6. Acceptable maintenance and replacement costs

Another set of criteria is needed to insure contructability in the Arctic environment. Criteria 1 to 3 would be normally lumped together and satisfied by the safety margins against the ultimate limit states. However in this instance this is not necessarily an axpedient way to address the problem. A better way to look at the problem is by stating that Criteria 1, 2 and 4 concern not only the owner of the facility, but are also a concern of government at various levels, adjacent property owners, residents in the general area, and possibly other countries. Criteria 3, 5, and 6 primarily concern the owner of the facility and could be addressed in purely economic terms. This distinction is important, because protection against loss of life and major environmental damage could conceivably be accomplished even if the structure is permitted to fail under, or designed to avoid extreme lateral forces, such as those resulting from a collision with an ice island.

The six performance criteria listed above must be stated in terms of acceptable failure probabilities. The question therefore arises, how such probabilities can be determined. Figure 31, which was presented by Whitman in the seventeenth Terzaghi lecture [22], is based on evaluated risks to structures and other engineering projects, and gives some indication of accepted risks in present engineering practice. The ratio between the cost of the loss and the accepted risk, obtained from a cost benefit analysis for Arctic offshore structures, may differ from the trend shown in the figure because of the unique problems associated with the Arctic region. Nevertheless, the figure gives some indication of what may be acceptable to our society in conjunction with the "ultimate" limit states.

In the figure, an annual failure probability of the order of 10^{-3} is shown for fixed drill rigs. However, this probability primarily addresses the rigs in the Gulf of Mexico, where the cost per drill rig is lower and the consequences of environmental damage are more manageable and less severe. For production platforms in the Arctic region, an annual probability of catastrophic failure of the order of 10^{-4} or less would probably be more in line with the financial and environmental consequences. Examination of figures 25 and 27, which give some insight into design criteria actually used indicates, that a design for the 100 year ice load cannot provide this level of reliability in locations where ice island invasions can occur, unless some mechanism is provided by which extreme loads can be either avoided or prevented from occurring. An additional factor which must be taken into consideration is the limitation imposed by the lateral load capacity of foundations which will be discussed later.

In the case where economic considerations govern, such as for instance for Criterion 3, a cost benefit analysis would be more appropriate than the imposition of some arbitrary failure probability. The methodology used in such an analysis is illustrated in figure 32, which is taken from Ref [23]. In view of the fact that the cost of an offshore production facility could

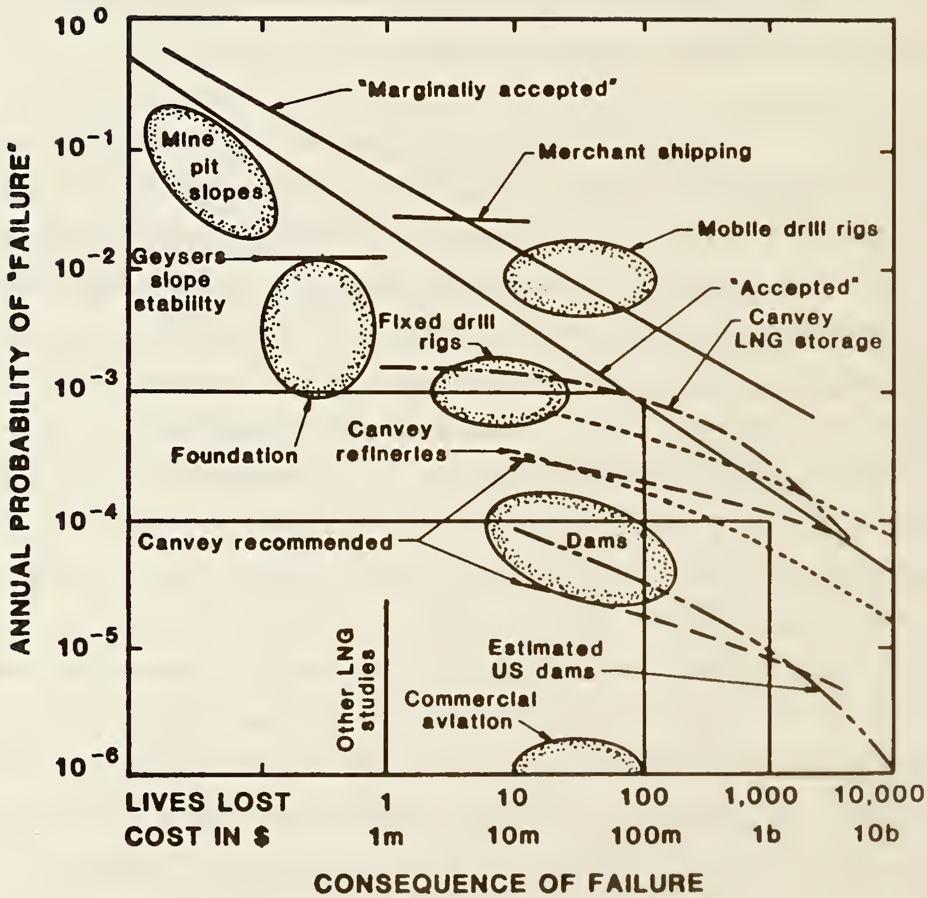


Figure 31. Risk for Selected Engineering Projects (taken from Ref.[22]).

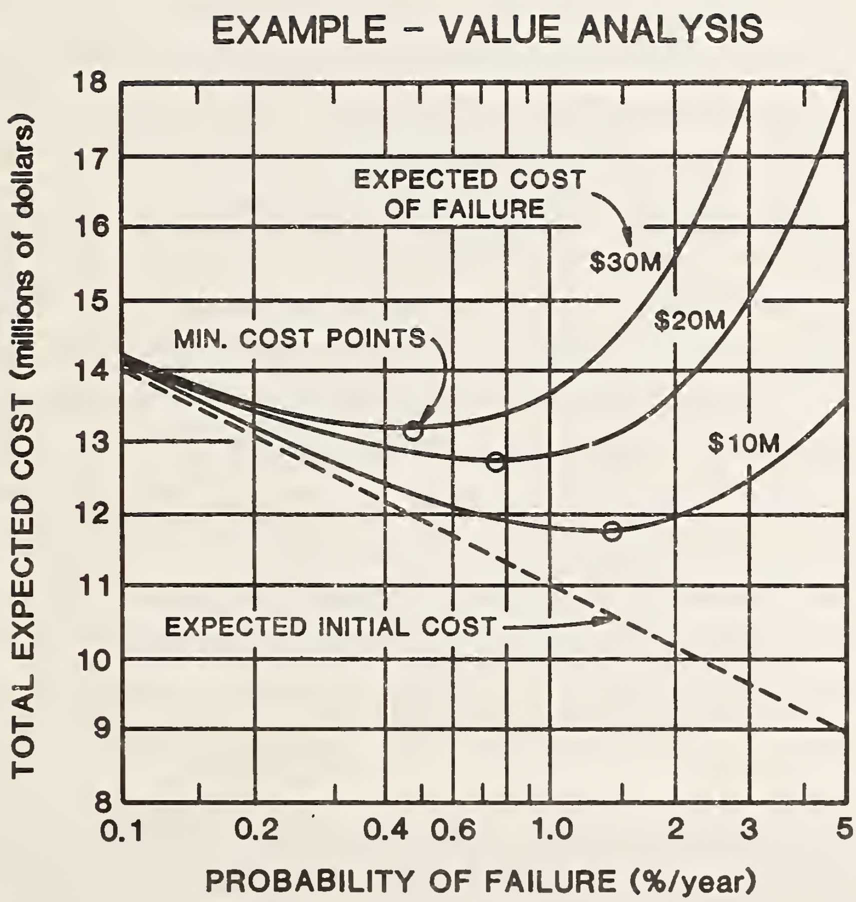


Figure 32. Probability of Failure Obtained from a Cost Benefit Analysis (taken from Ref. [23]).

exceed $ 1 billion, a cost benefit analysis may also result in a low failure probability. Thus the design capacity against lateral loads may be limited by our ability to build lateral-load resisting structures and foundations, and the design solution may have to be supplemented by strategies which would allow us to avoid or prevent exposure to extreme ice loads and the consequences associated with undesirable foundation performance.

Acceptable risks for exploration structures would be considerably higher than those for production structures, because these structures are less costly, and the environmental impact of a catastrophic failure would not be as high as in the case of production structures. The period of exposure of these structures in any one location and their useful life are also much shorter than those of production structures. The data in figure 31 indicate a failure probability in the order of 10^{-2} for existing mobile drill rigs, and that this failure probability seems to exceed even marginally accepted reliability in present engineering practice. Again it appears that for Arctic installations this failure probability would have to be lower, even if dictated by purely economical considerations. As these structures are usually designed to be mobile, it may not be very problematic to plan on avoiding extreme ice loads which tend to occur during the summer open water season. Annual failure probabilities of the order of 10^{-3} may be reasonably compatible with accepted engineering practice.

Failure probabilities associated with criteria 4, 5, and 6 would tend to be dictated by cost benefit considerations, except that in the case of criterion 4 there may be instances where even minor environmental damage may not be acceptable. The difficulties with these criteria may be in the prediction of environmental effects which would cause failures, such as cumulative displacements, erosion, local force concentrations, and possibly dynamic effects.

4.3 Structural Concepts

4.3.1 Background Information

Since the technology of Arctic drilling and production is in a stage of rapid evolution, many different concepts are under consideration or in different stages of design and construction. For instance Buslov and Krahl [24] discuss 51 new concepts.

There is a record of past successful experience with gravity structures in hostile offshore environments. Concrete lighthouses have been used in the St Lawrence Seaway and the Baltic Sea since the turn of the century, and concrete gravity platforms have been installed in the hostile waters of the North Sea since the early 1970's [25]. Past offshore exploration in the Arctic was generally confined to shallow waters and was carried out either from the ice or from artificial islands constructed from locally dredged materials.

More recently different exploration structures for use in deeper waters were built by U.S. and Canadian companies, including caisson retained islands, mobile gravity structures and floating vessels. Many other concepts for exploration and production structures are in various stages of planning and design.

4.3.2 Gravity Structures Used or Considered.

This report deals specifically with design limit states for foundations. Thus only those features of the various structural types which are relevant for foundation design are discussed. There are essentially three types of foundations associated with the structures under consideration: foundations for earth structures, for artificial islands (though the island also incorporates a variety of earth supported structures); foundations for caisson-retained earth structures; and mat foundations supporting rigid structures. Since the design limit states for these foundations do not only depend on the foundation type, but also on the geometry, flexibility, and vulnerability of the supported structure, it is necessary to discuss the characteristics of these structures in some detail.

1. **General Characteristics:** the offshore structures discussed in this report are either exploration or production structures. **Exploration structures** are either temporary in nature, such as dredged artificial islands which are used in the winter sason and later erode, or mobile structures which are re-used after completion of the exploration. Exploration structures need a working area of about 100m diameter and are often designed for loads with a 25 year mean recurrence interval (even though this philosophy may be questionable for re-useable structures [25]). **Production Structures** are normally built for a service life of 25 to 30 years and require a working area of approximately 200m diameter (unless they are multi level). Non Arctic offshore production platforms are normally designed for loads with a 100 year recurrence interval.

2. **Artificial Islands:** Most of the artificial islands in the Alaskan Beaufort have been built in water depths of about 7m. The gratest depth is 13m [26]. The materials used are mostly gravels. Construction methods include barge haul and winter haul over ice roads. On the Canadian side, there is also a considerable amount of sand pumping by dredges and bottom dumping from barges. Artificial islands become uneconomical in depths greater than 20m. Design slopes depend on the grain size of the construction material, the method of placement, and whether the slope is in the upper zone (+ 3m to top), the wave action zone, or the bed level zone (-3m and deeper). Since erosion and wave runup are major problems, sacrificial berms or other means of slope protection have to be used. In the Alaskan Beaufort, slope protection generally

consists of 2-4 cubic yard sand bags. Figure 33 which is taken from Ref.[26] shows different slope treatments used. In the Canadian Beaufort, where dredging is extensively used, slopes are flatter (typically 1:15) and erosion protection is often provided by sacrificial berms.

3. **Caisson Retained Islands (CRI):** These structures have a sand core which is contained by peripheral caissons. Some of these caisson structures are sectional, such as the Tarsiut Island structure shown in figure 34 (Ref.27), which has a sand core, contained by sectional concrete caissons and rests on a sand berm in a water depth of 21m. Others have a continuous caisson shell around a sand core, such as the Gulf Mobile Arctic Caisson (MAC) which has an annular steel caisson and is designed for water depths of 15 to 40m. This structure is discussed in Ref.[28] and is shown in figure 35. Other structures of this type are the Esso stressed steel CRI and a planned Sohio concrete CRI [29]. The concept is also under consideration for production facilities [30].

4. **Rigid Base Structures:** Under this heading many different types of offshore structures are lumped together because they rest on a rigid structural mat, which either rests on a prepared berm or is placed directly on the ocean floor. These structures have either vertical sides which are designed to resist ice pressures as wide indenters, or sloped sides which are designed to induce flexural failure in the ice. Several types of gravity structures are included in this category: (1) Water ballasted caisson structures. Included in this category is the vertical sided Super CIDS, built by Global Marine for Exxon, which is shown in figure 36 (taken from Ref.[31]). The latter structure was installed with a protective ring of grounded ice to provide protection against extreme ice forces. The BWACS cellular floating caisson is also vertical sided and was developed by Brian Watts Associates and commissioned by Zapata

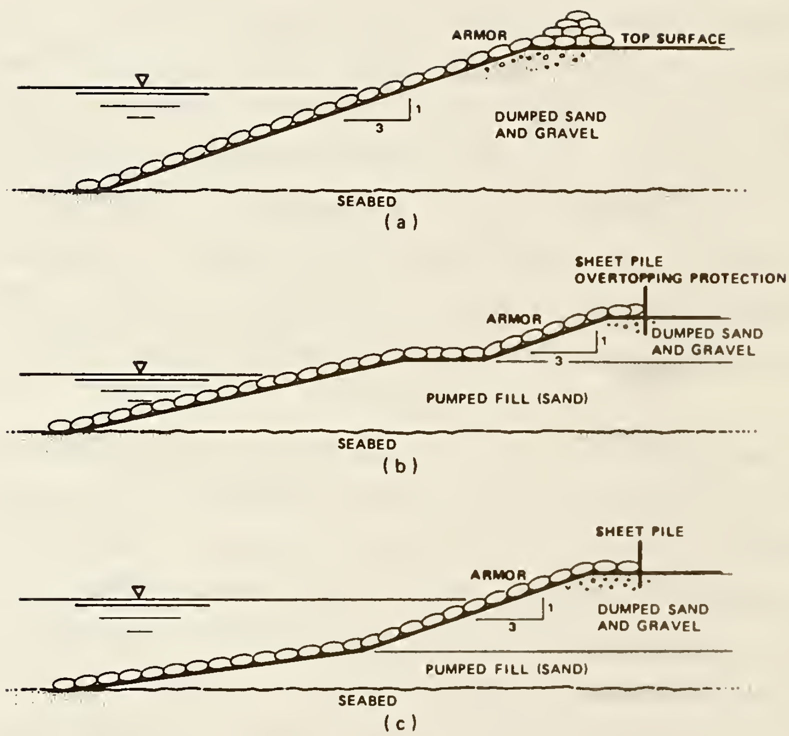


Figure 33. Typical Shallow Dredged Island Cross Sections (taken from Ref.[26])

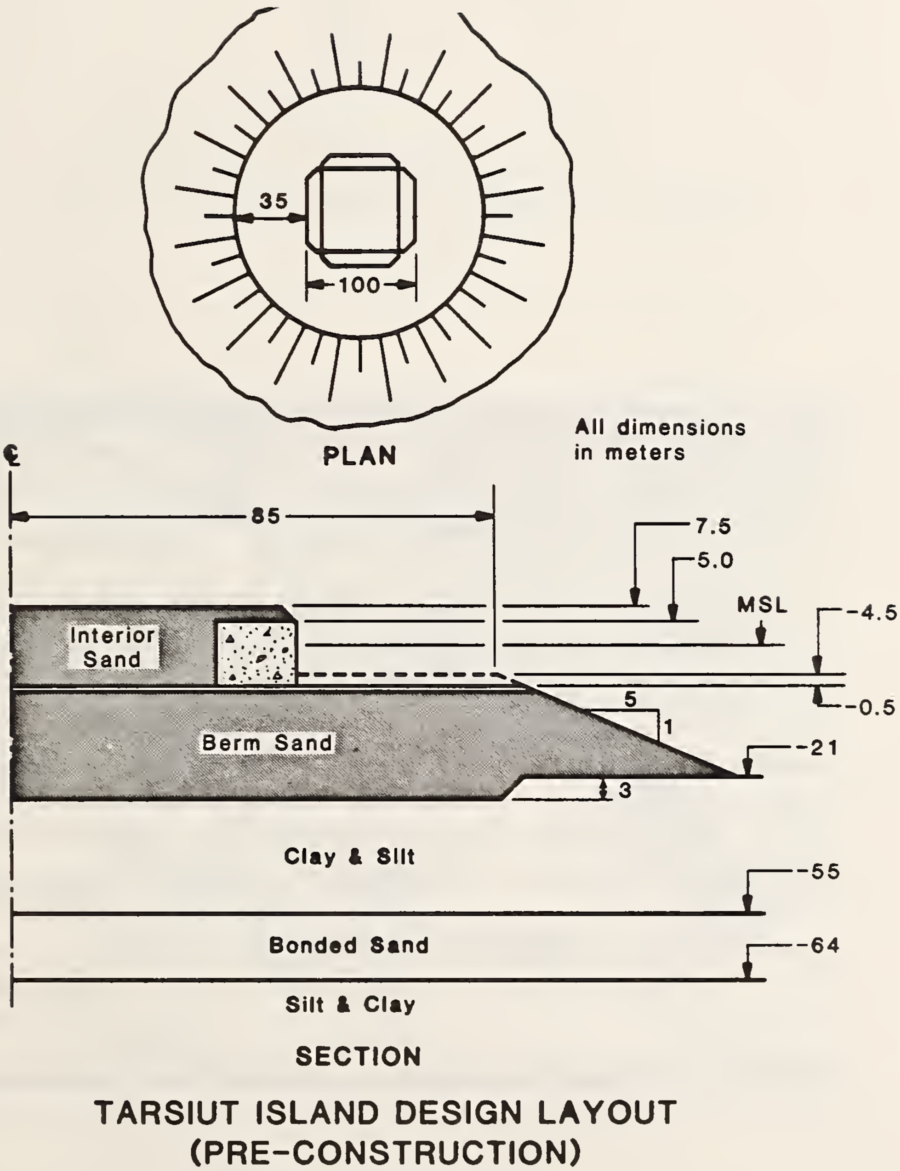


Figure 34. Tarsiut Island [27].

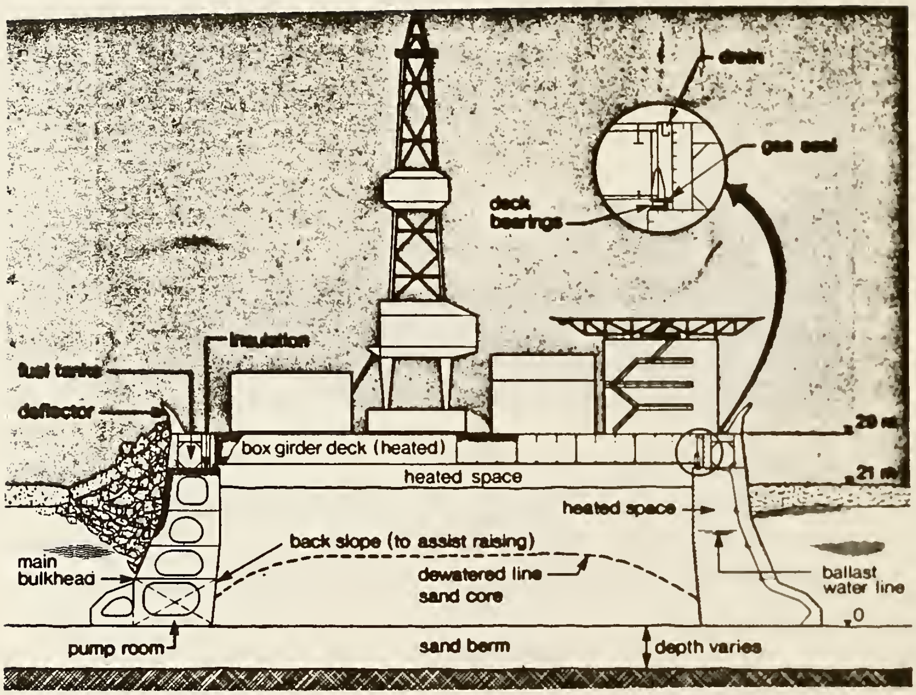


Figure 35. The Gulf Mobile Arctic Caisson (MAC)[28].

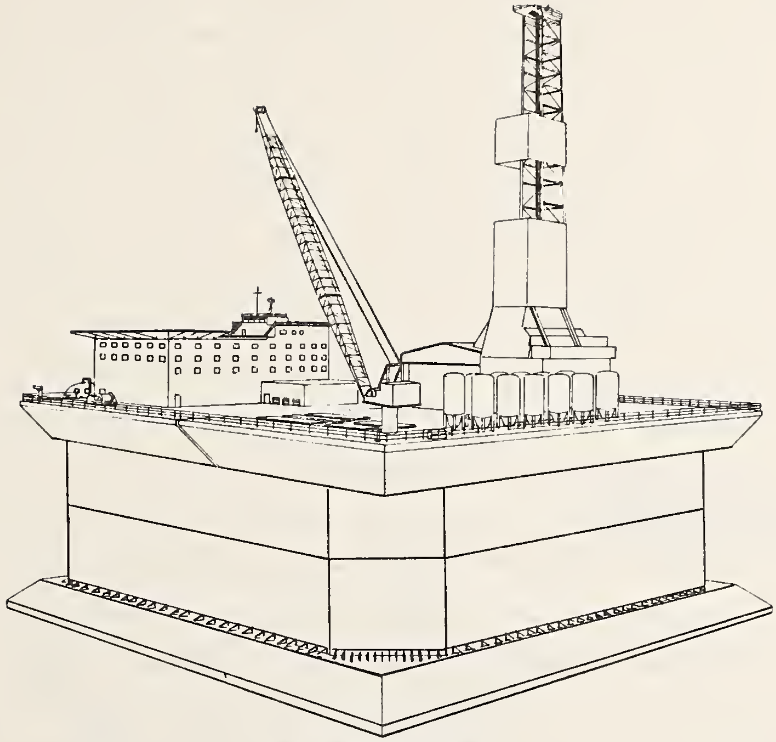


Fig. 1—Super CIDS—isometric.

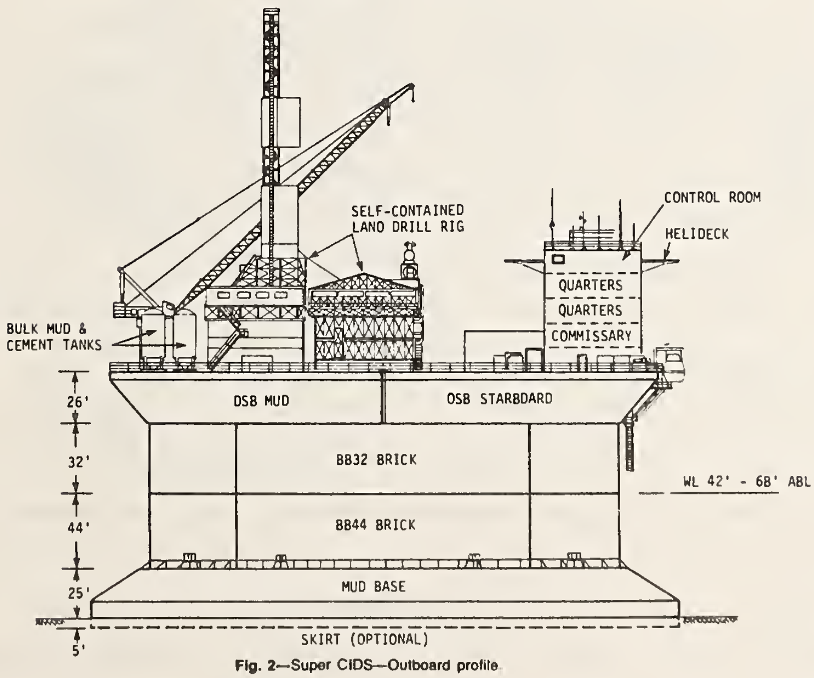


Fig. 2—Super CIDS—Outboard profile.

Figure 36. Exxon's Super CIDS [31].

Offshore Services and is shown in figure 37 (from Ref.[31]) Sloped sided water ballasted caisson structures include the Arctic Cone Exploration Structure [33] which is designed for heavy ice and for water depths from 15 to 33m. The latter structure is designed to be placed on the unprepared ocean floor. It has short skirts and a sand undergrout system that permits injection of sand into the voids under the base. This sand can then be further consolidated by ballasting. The structure is shown in figure 38. Another, "second generation" concept in conical caissons, the SONAT hybrid arctic drilling structure is discussed in Ref.[34]. The structure has a steel base to provide some foundation flexibility and a concrete mid section, and was designed with special attention to minimizing ice loads and ice runup effects.

(2) A second category of rigid based structures is equipped with a wide rigid base to maximize soil resistance and a relatively narrow stem to decrease the width exposed to ice forces. These structures are often referred to as "monopode" structures. The stem is normally equipped with an icebreaker cone. An example of such a structure is the adjustable monocone rig which was developed by Esso Resources of Canada and is shown in figure 39 [35]. Various models of these structures were designed to operate in water depths ranging from 10m to 60m. (3) Miscellaneous other structural concepts have been developed. These include: 1. The Sohio Petroleum Arctic Mobile Structure (SAMS) which is shown in figure 40 and discussed in Ref.[36]. This structure is an octagonal water ballasted concrete barge which is equipped with a system capable of inserting and retrieving 7m diameter steel pipe caissons (spuds) to augment the lateral load resisting capacity of the mat foundation. Sophisticated monitoring systems keep track of the vertical and lateral loads and deformations. It is planned to insert the spuds only if the need arises. 2. A proposed detachable monocone production structure [37], which consists of a steel cone, supported by a

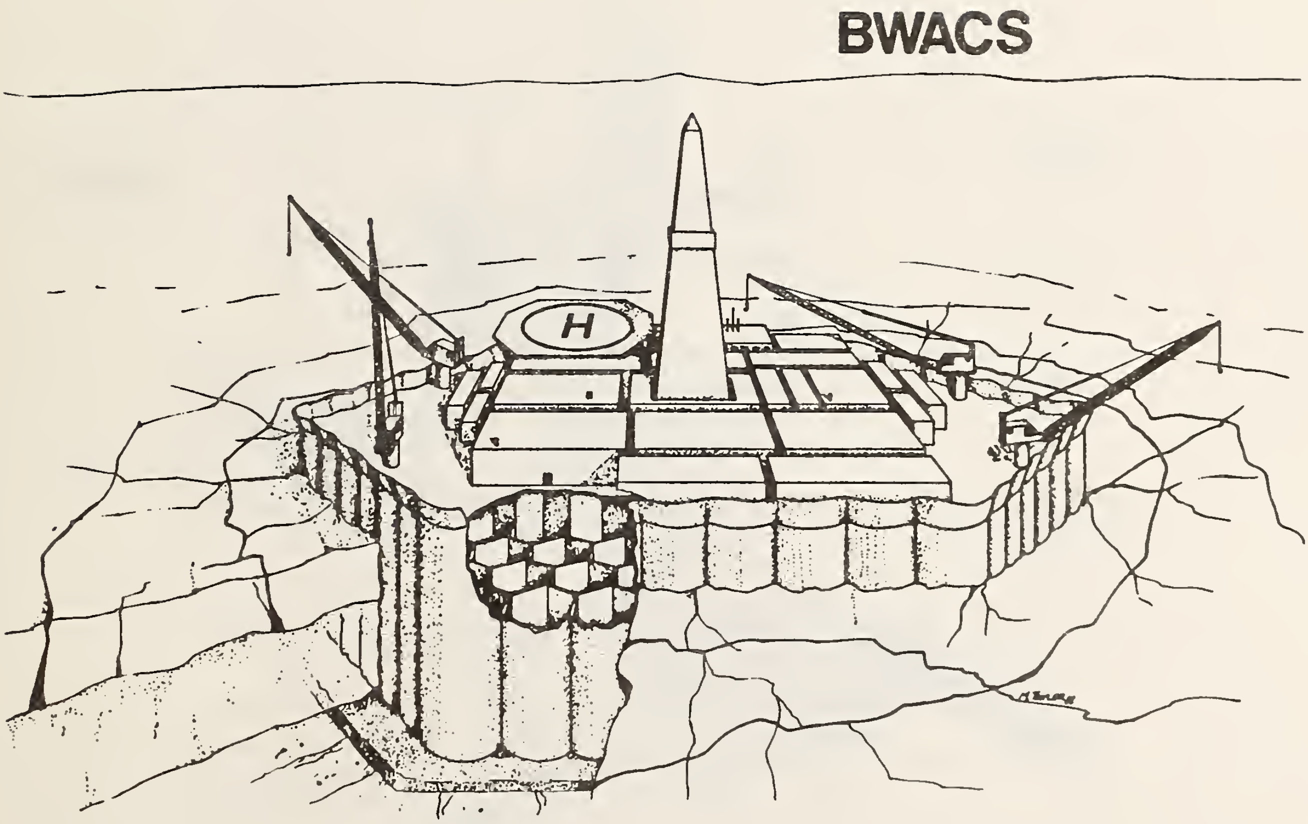


Figure 37. The BWACS Floating Caisson [31].

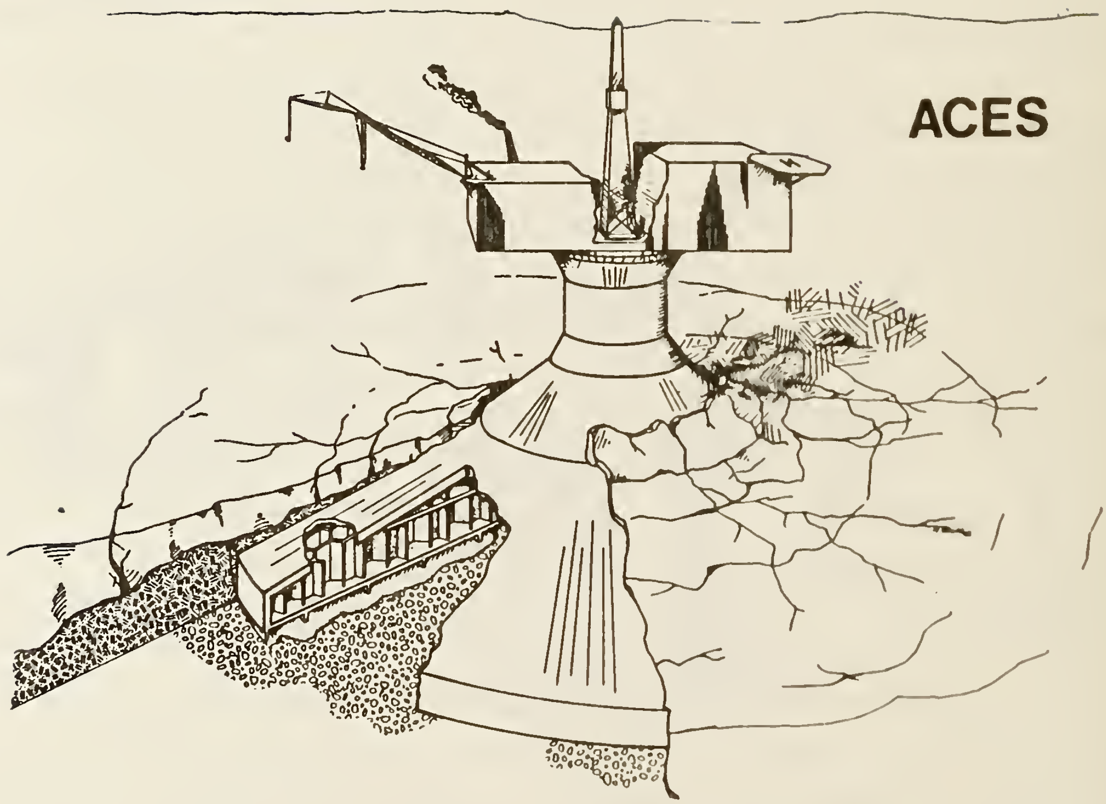


Figure 38. The Arctic Cone Exploration Structure (ACES) [33].

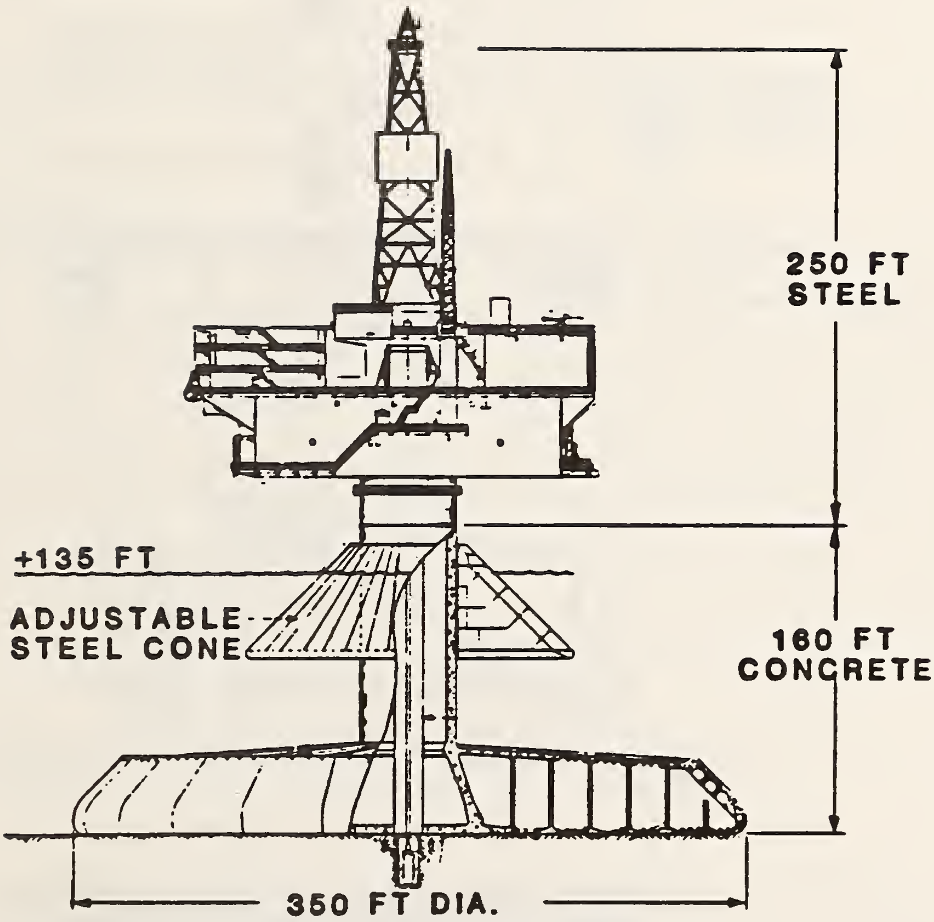


Figure 39. Adjustable Monocone Rig [35].

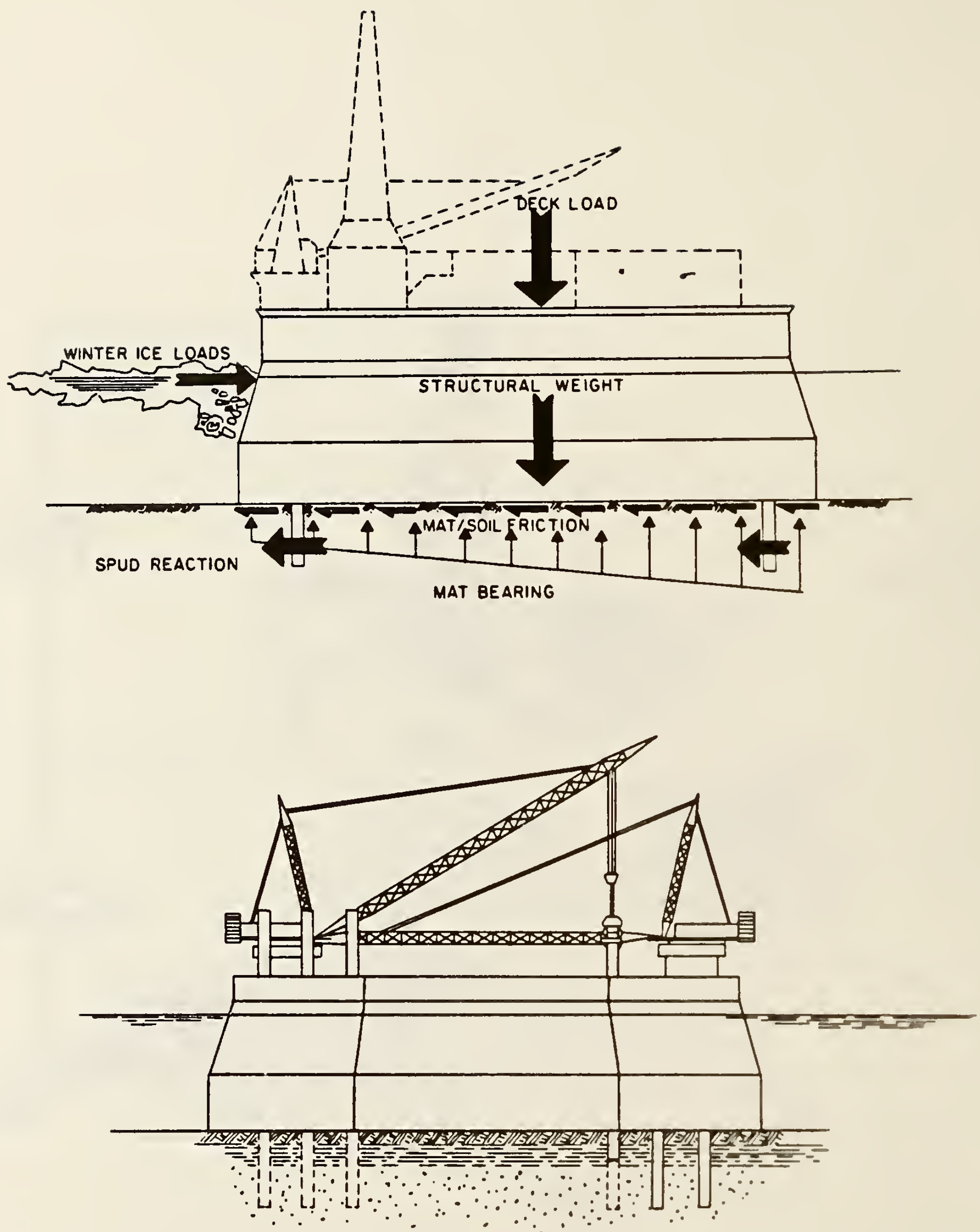


Figure 40. The Sohio Arctic Mobile Structure (SAMS) [36].

steel or concrete 130m diameter circular mat foundation. The cone is designed to disconnect from the base, so it can be removed during a rare ice event. 3. A stacked annular caisson system for yearround drilling in 4 to 20m water. The stacked sectional caissons have a center core which can be filled with sand. The sand core is dewatered to increase the shear strength of the sand. 4. Some concepts for structures capable of resisting ice island impact were also proposed [39, 40].

4.4 Design Limit States for Foundations

4.4.1 Definition of Limit States

Two categories of limit states are defined for the foundations. **Ultimate limit states** are failure modes causing major damage to the supported structure (either total loss or long term interruption of service). Included in this category are not only stability and truncation failures, but also settlements and displacements which would precipitate a major structural failure. **Serviceability limit states** are failure modes which cause short term interruption of service or damage which requires repairs or replacements which can be accomplished without replacing a major part of the structural system. Included in this category are local stability failures, settlements and displacements which require remedial measures ,and erosion.

It is necessary to distinguish between the two categories of limit states because of the much greater safety margins necessary to protect against ultimate limit states. However in practice the dividing line is not necessarily clear cut. For instance settlements caused by thawing of relict permafrost may be an ultimate limit state when no remedial measures are available to mitigate the effects on the supported structure, and a serviceability limit state when provisions are made to mitigate the settlement effects by underfilling.

4.4.2 Practical Limits to Load Resistance

Overall requirements for safety and serviceability were discussed in Section 4.2 and it was recognized that safety margins which would be consistent with present engineering practice cannot be realized because of physical limitations to the lateral load resistance of gravity structures. The limitations to the load resistance that can be provided by mat foundations are illustrated by the following example: Let us assume that it is desired to locate a cylindrical structure of diameter D on a stiff mat foundation of diameter D + 50 ft.,that the design ice load is 700 kip/ft and that the strength of the foundation is determined using a load factor of 1.3, and a resistance factor of 0.8. Then the ratio of resistance to load, R/S, would have to be 1.6. Let us further assume that the structure should be able to resist the extreme ice load of 1000 kip/ft. with a load factor of 1 and a resistance factor of 1 (R_u/S_u = 1). These assumptions are reasonably consistent with present engineering practice. Let us now assume that the soil conditions are similar to those shown in figure 20. Even in the most favorable location, the soil resistance would not exceed 2 kip/ft^2. Thus the assumption that the lateral load resistance cannot exceed 2 kip x the area of the mat foundation would not be overly conservative. The implication of this assumption is shown in figure 41, where R/S and R_u/S_u are plotted against the required diameter of the mat foundation. It can be seen that even a 600 ft. (200m) wide mat foundation could not satisfy the design criteria. But as the foundation width increases, so does the required thickness of the mat and the logistic difficulty of building, transporting and erecting the structure. There are various methods by which the soil resistance can be increased, ranging from skirts and spud piles to compaction grouting. But these methods, which will be discussed later in this section, also have economic and physical limitations. Thus various strategies were adopted to limit or avoid exposures to extreme loads, or to mitigate failure consequences.

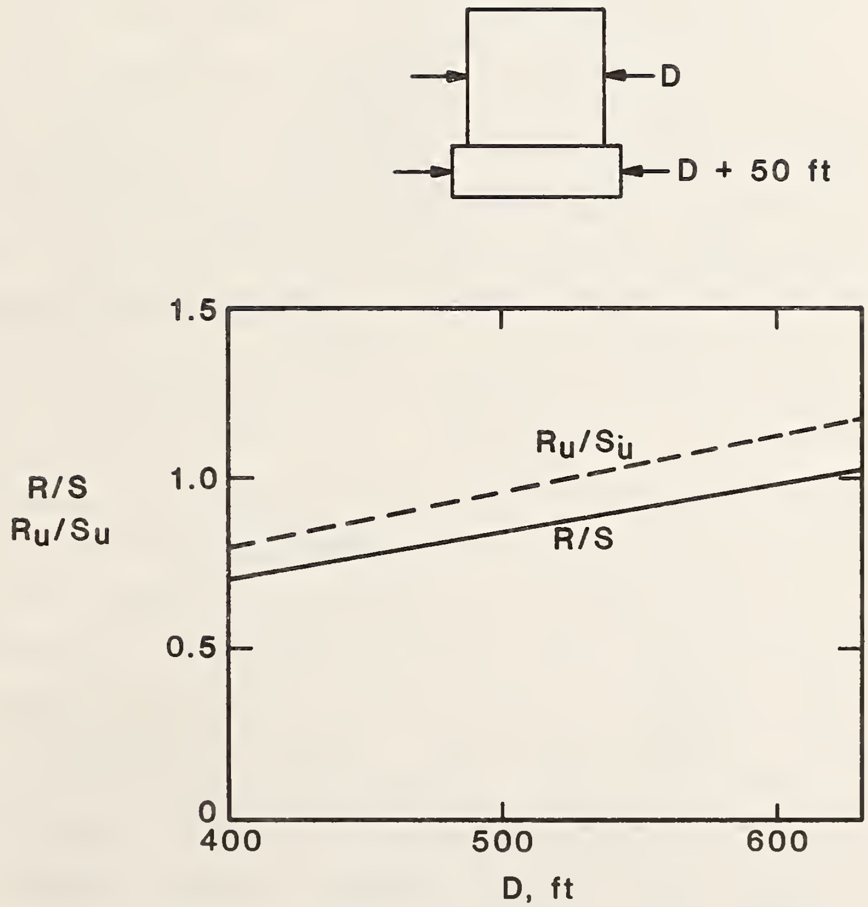


Figure 41. Load Resistance of Mat Foundation

4.4.3 Design Strategies

Possible design strategies fall into three categories:

(1) **Brute strength.** The difficulties with this approach are obvious and have been discussed.

(2) **Hazard Mitigation.** In this strategy, the performance criteria in Section 4.2 are satisfied by mitigating hazards by methods other than structural load resistance. Several such strategies have been used:
1. Personnel Protection
2. Well Protection
3. Structure Protection
4. Ice force Management

These strategies can be used separately or in combination to get the desired level of reliability. A good example of using strategies 1. and 2. is Tarsiut Island. It is realized that extreme ice forces could cause a catastrophic failure. Rather than trying to provide load resistance against these extreme forces, it was decided to mitigate the consequences of potential failures [41]. For this purpose, four alert levels were instituted, ranging from normal operations to total evacuation after a safe shutdown of the wells. An example of Strategy 4.,ice force management is the protective ring of grounded ice installed around Exxon`s Super Cids. Other measures, such as splitting dangerous ice features before they impact the structure are under consideration. Examples of 3., structure protection are the sacrificial berms in the artificial islands, and also the capability of the floating caissons to be moved from a particular location during the critical summer season. Another form of structure protection are energy absorbing devices developed to protect offshore structures from the effects of floating icebergs. After the collision, these devices can be restored to their original position [39]. Weak ductile links in the structure or yielding foundations could also protect the structure.

3. **Adaptive Controls.** These strategies involve the design of the structure to a reasonable nominal strength level, with provision for a capability to increase load resistance in the field in accordance with sensed or observed structural performance. A good example of adaptive controls is provided by the Sohio SAMS structure shown in figure 40. The structure is instrumented with sophisticated monitoring systems and it is planned to install the spuds only if ice forces are expected to exceed certain critical levels. The spuds can be installed and retracted relatively rapidly. A second adaptive system on the SAMS is associated with expected settlements due to thawing of relict permafrost. The remedial measure in this instance is sand underfilling. Both the instrumentation and the projected settlements, which are discussed in References [42] and [43], are shown in figures 42 and 43. Adaptive controls could conceivably also be used in other ways to modify foundation resistance, such as ballasting and grout injection.

4.4.4 Failure Probabilities Associated With Limit States

Design procedures for gravity structures presently used generally follow the FIP recommendations [44], which address themselves primarily to the North Sea conditions. In this document "extreme" loading conditions, to be used with ultimate limit states, have a mean recurrence interval of 50 to 100 years, and "normal" environmental conditions have a mean recurrence interval of 1 month. Neither of these provisions is applicable to ice loads in the Arctic, however a 100 year mean recurrence interval for production structures and a 25 or 50 year mean recurrence interval for exploration structures could be used if appropriate design strategies are available to deal with specified extreme ice features.

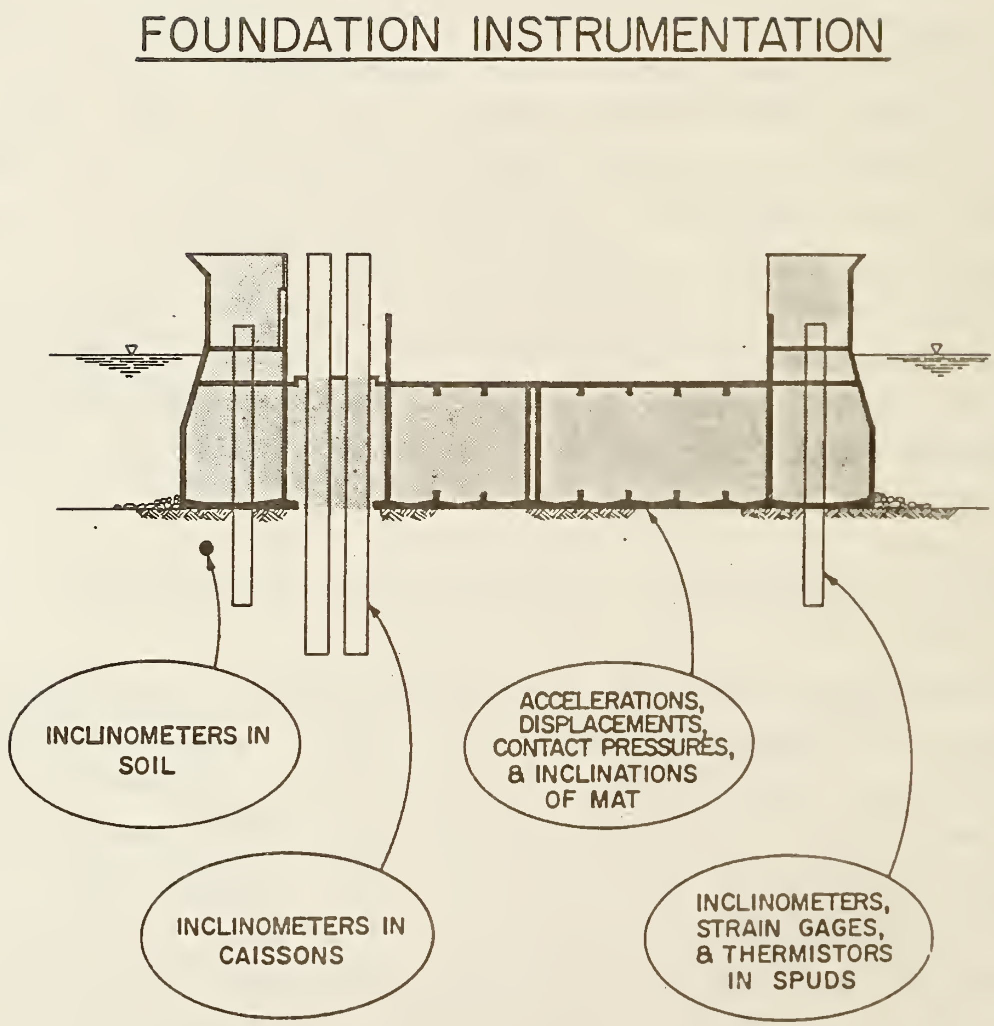


Figure 42. Instrumentation Used to Monitor the Performance of SAMS.

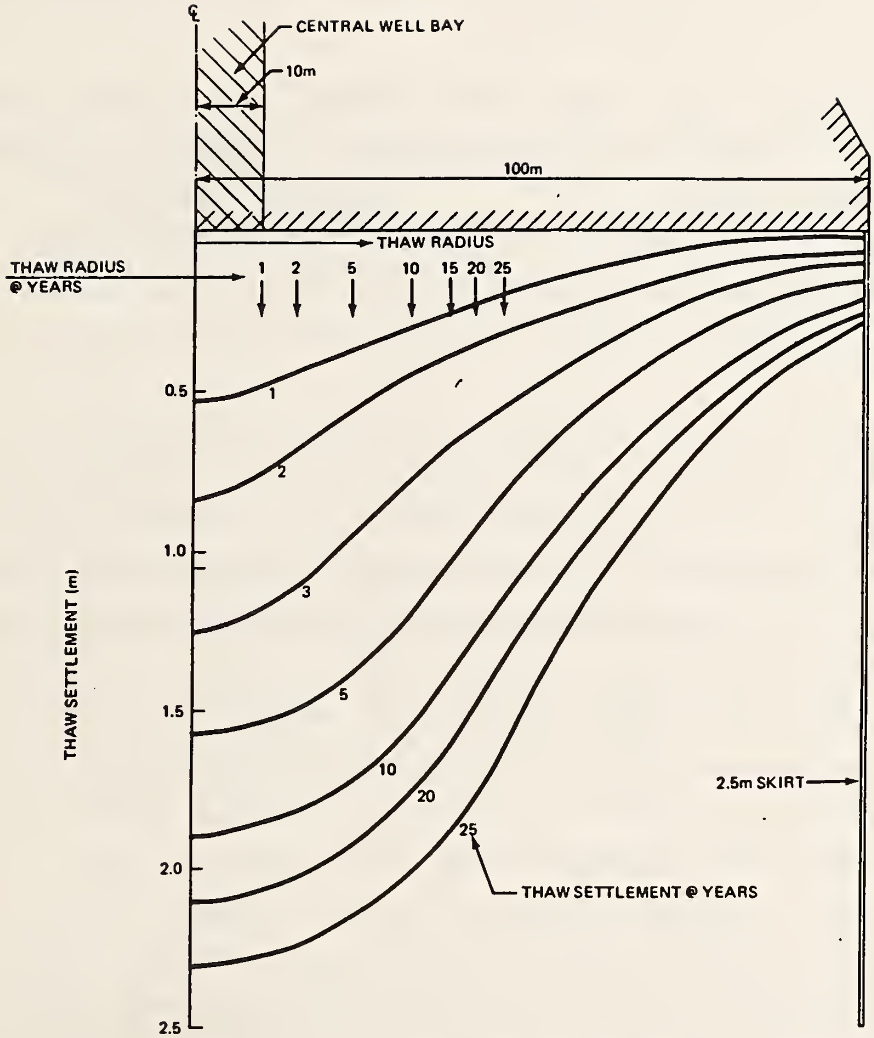


Figure 3–7

THAW SETTLEMENT PROFILES
CENTRAL WELL BAY CONFIGURATION

Figure 43. Settlement Control Planned for SAMS.

The assumption can then be made, that if the appropriate probabilistic parameters for the loadings (demands) and resistances (capabilities) can be determined, failure probabilities, conditional on the ability to implement design strategies associated with extreme ice features, can be calculated.

The following load and material factors are cited in the FIP state of the art report on North Sea gravity structures [45]:

Reference	Load Factor for Env. Load	Material Factors Cohesion or Undrained str.	Material Factors Effective Friction
FIP (1977)	1.3	1.4	1.2
DnV (1977)	1.3	1.3	1.2
NPD (1977)	1.3	1.3	1.2

These factors correspond to the European "semi probabilistic" design approach. The "material factors" correspond to the inverse of the capacity reduction factor used in U.S. practice. The cohesion and undrained strength column mainly applies to silty and clayey soils and to sandy soils subjected to dynamic loading, while the effective friction column applies to sandy soils. It can be seen that there is reasonable consensus on safety margins and that global nominal safety factors range from 1.82 for cohesive soils to 1.56 for sandy soils. Even though there is no definitive credible set of data on which assumptions with respect to the statistical characteristics of the load and resistence parameters can be based, it is possible to asses failure probabilities associated with the above mentioned global safety factors when some engineering judgment is made with respect to load and strength variabilities.

The following closed form solution can be used to calculate notional failure probability (P_f) as a function of the global safety factor, if the loading and resistance distributions are idealized as lognormally distributed variables {[46] (as modified and supplemented)}:

$$P_f = 1 - \Phi\{(\ln [R/S\ (1 + V_S^2)/(1+V_R^2)]^{1/2})\ /\ [\ln(1 - V_S^2)(1 + V_R^2)]^{1/2}\}$$
$$\simeq 1 - \Phi[\ln (R/S)/\ (V_R^2 + V_S^2)^{1/2}] \quad \text{for } V_R,\ V_S < 0.3 \quad \ldots(\text{Eq.}4.1)$$

where: $\Phi[X]$ = tabulated normal cumulative probability of the standard variate, X (expression in brackets)

R = mean value of resistance

S = mean value of load

V_R = coefficient of variation of the resistance

V_S = coefficient of variation of loading

While it is not anticipated that credible data on the statistical parameters in the above equation will be available in the forseeable future, estimated values may give us some indication of the order of magnitude of failure probabilities. For instance, the following values are calculated:

R/S	V_R	V_S	P_f
1.82	0.2	0.2	.016
1.82	0.3	0.2	.053
1.69	0.2	0.2	.03
1.56	0.1	0.2	.02

P_f in the table would be the failure probability for a time period equal to the mean recurrence interval of the design load. If it is assumed that for the North Sea a 100 year mean recurrence interval was used for determining the design loads on production structures, the calculated failure probabilities would range from 1.6×10^{-4} to 5.3×10^{-4}.

Another factor that should be taken into consideration is the conservatism inherent in traditional geotechnical engineering practice. Thus design values for undrained strength or the angle of shearing resistance are likely to be somewhat lower than the true mean values that should be used to calculate P_f.

The approach used in the preceding calculations for North Sea structures cannot be applied to offshore structures in arctic regions where the coefficient of variation of the load is of the order of 100% ($v_s \simeq 1$). In that instance the annual P_f could be approximated as the reciprocal of the mean annual recurrence interval of the load required to develop the ultimate capacity (resistance) of the structure.

The foregoing discussion is intended to illustrate how probabilistic reliability approaches can be used,as an aid in determining design loadings and load capacities. The computed notional P_f's should be interpreted as indices that are indicative of actual P_f's. They should not be confused with the true safety or reliability characteristics of a given structure. The notional P_f's have value in quantitative comparisons of design criteria alternatives [76, 77]. Extensive literature has been developed during the last 20 years on the topic of reliability based design criteria for offshore structures [78 through 83].

4.4.5 Design Limit States for Artificial Islands

Artificial islands have been constructed by three methods: Summer construction by dredging (the fill material is taken from a nearby site and deposited directly by dredges); summer construction by bottom dumping from barges; and winter construction by dump trucks supported by ice. The strength and grain size characteristics of the fill material, as well as the sideslopes of the island depend to a large extent on the construction method used.

Dredged material normally ranges from fine silty sand to gravel. Bottom dumping is only possible when $D_{50} \geq 0.28$mm where D_{50} is the grain size below which 50% of the material by weight is smaller, and when the silt content [material passing a #200 (0.075mm mesh) sieve] is less than 5% [47]. Thus a material meeting these minimum requirements would have to be poorly graded, or else the material would have to be courser. Trucked material is gravel from on shore borrow sites.

Typical sideslopes used are summarized in the following table which is taken from Ref.[26].

Material	Elevation	Placement	Side Slopes
Onshore Gravel	+ 10 ft. to top	Truck Haul	1 : 1.5
	Wave Action Zone		1:8 to 1:10
	Bed Level to -10ft		1 : 3
Offshore Sand and Gravel	+ 10 ft. to Top	Dredge	1 : 1.5
	Wave Action Zone		1:12 to 1:15
	Bed Level to -10ft		1 : 8
Offshore Sand and Silt	+ 10 ft. to Top	Dredge	1 : 3
	Wave Action Zone		1 : 20
	Bed level to -10ft		1:12 tO 1:15

Note that slopes are dictated not only by strength and stability requirement but also by construction feasibility.
Six generic categories of limit states need to be considered: Major stability failure; local stability failure; excessive displacements; erosion; wave overtopping; and runup of ice.
Whether these limit states are ultimate or serviceability would entirely depend on the severity of the consequences of failure and should not be prescribed a priori.

Stability Failures: Two types of loading can cause stability failures: gravity loads, and ice push. Stability failures under gravity loads will tend to occur toward the end of construction, when the maximum gravity load is applied while the foundation soil may not have had sufficient time to consolidate. The problem occurs when the soils supporting the island are cohesive. Failure could occur at the mudline where bottom sediments tend to be the weakest, or at some depth below the mudline because consolidation at that depth will require more time. To analyze this problem, soil strength as a function of time as well as depth must be considered. Because of the peculiarities of frozen fills, slope stability problems can also arise in steep slopes during winter fill construction [48].

Potential stability failure slip planes associated with ice push are shown in figure 44 which is taken from Ref.[26]. The broken line shows the depth of freezing. The frozen cap of the island is likely to act as a rigid body, thus facilitating a major stability failure of this type. The slip plane could also be located at some depth below the mudline. Watt [48] points out that stability problems should be analyzed as 3-D problems so that account is taken of the island geometry. Slip surface 2 in figure 44 is a truncation failure through the granular fill of the island, which has relatively low density and therefore is sensitive to even small cyclic loads such as those induced by wave action. These cyclic loads can cause excess pore water pressures and thus reduce the effective stress, and consequently the shear strength on the slip plane. This possibility should be cosidered if ice forces during the open water season could cause a threat. A parametric study by Kontras et al.[49] in which various assumptions were made with respect to the location of the bottom of the freeze front indicates that the failure plain is likely to be located at the freeze front, even if it extends into the bottom sediments. During the first winter, local, rather than global ice failures are likely to occur (slip plane 1 in figure 44) since sea ice is stronger than the unfrozen fill. In subsequent easons the global stability failures pose a credible threat, even though localized failure mechanisms may reduce the overall ice forces.

Figure 44. Potential Failure Modes of Fill Islands (Taken from Ref.[26]).

Another failure mode not shown in figure 44 is liquefaction. This type of failure could occur in seismically active zones. It also could be precipitated by other dynamic or rapidly applied loads, However the latter possibility seems remote. Artificial islands and other hydraulically placed soils are very vulnerable to earthquake induced liquefaction because the soils are granular and have low relative densities.

Excessive Diplacements: Excessive settlements could occur as a result of the weight of the fill, oil or gas withdrawal, or thawing of relict permafrost. Damage caused by settlements would be to structures and well casings. Settlements could also lower the top elevation of the island and increase its vulnerability to truncation failures, wave topping and ice runup. Excessive lateral displacements could also occur since the frozen upper portion of the island could act like a rigid body. Mitigation of displacement effects would be difficult, however flexibility could be provided to accomodate reasonable amounts of displacement. Horizontal displacements may be very difficult to predict.

Erosion: Erosion could be caused by wave and current action, ice gouging and strudel or other scour. Slope protection was discussed in Section 4.3.2 and was shown in figure 33. Erosion is a serviceability limit state, but it could precipitate stability failures if unchecked. Erosion, particularly during the summer season, is likely to require periodic maintenance work. Experience with 4 cubic yard sandbags at Seal Island [50] indicates that erosion protection is expensive, and the bags tend to be damaged by ice. Other forms of protection, such as concrete armor units (i.e. DOLOS) have been suggested but not tried. In dredged islands, large sacrificial berms are sometimes used instead of slope protection.

Ice Runup: Ice runup is not strictly a foundation limit state. Structural type protection normally provided is illustrated in figure 33. Another method of protection may be construction of sacrificial berms, which cause formation of rubble fields away from the island. Such grounded rubble piles may also act to reduce ice forces, but this effect is offset by an increase of the width of the area exposed to ice forces [25]. If unchecked, ice could completely overrun an artificial island.

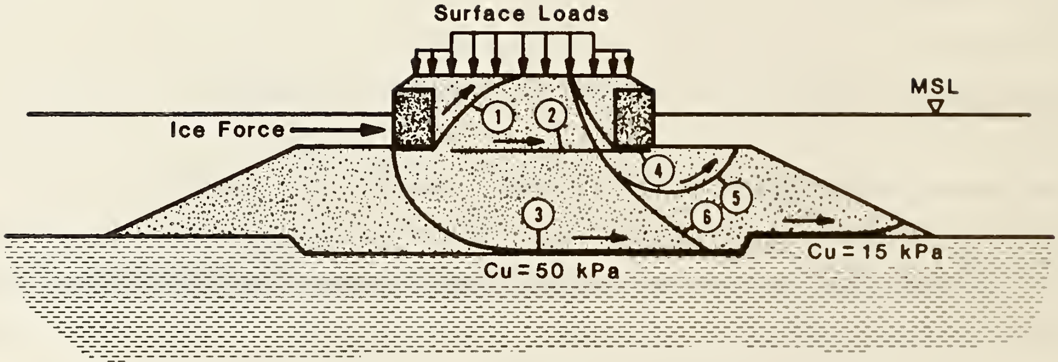
Wave Overtopping: Wave overtopping, and also spray may be a problem. Present designs seem to accept a 5% overtopping rate for exploration facilities and a 2% rate for production facilities. These criteria would require 25 to 30 ft freeboards (from data presented in [48]).

4.4.6 Design Limit States for Caisson Retained Islands

Caisson retained islands are normally placed on a sand or gravel berm, but they could conceivably be placed directly on the ocean floor if conditions are suitable. They have a hydraulically placed sand core which is partially dewatered to increase the effective stresses at the base of the caisson. Their stability depends to a large degree on the strength properties of the supporting sand berm and the enclosed sand core.

Seven categories of limit states are identified: Major stability failure; local stability failure; structural failure of the caissons; excessive displacements; erosion; ice runup; and excessive construction loads. Some of these limit states are dependent on the strength of the supporting berm.

Stability Failures: Figure 45 shows potential stability failure modes for Tarsuit Island, identified in Ref.[27]. Other possible failure modes for similar structures would be a bearing capacity failure under the caisson, possibly aided by seepage pressures caused by the partial dewatering of the core, and a deeper seated sliding surface if the foundation soil under the berm is



FAILURE MECHANISMS

1. Passive failure, single caisson
2. Decapitation
3. Beaded failure, entire structure
4. Active failure, single caisson
5. Rotational failure, single caisson
6. Beaded failure, single caisson

Figure 45. Failure Modes Identified for Tarsiut Island (taken from Ref.[27]

weak. The accuracy of an assessment of the safety margins against these failure modes would depend on the accuracy with which the shear strength of the supporting soils can be estimated. The shear strength of the berm and the core depends to a large extent on the grain size and method of placement of these materials. The purpose of the berm subcut shown in figure 45 was to interface with stronger materials, and also to provide a shear key for the berm. This is predicated on the assumption that the berm is stronger than the supporting deposits. Some additional failure modes were identified in Ref.[18] and are shown in figure 46. These include active slope failures behind the caisson precipitated by tensile forces associated with an ice breakout, and structural caisson failures associated with the uneven distribution of ice forces acting on the caissons.

Ring type caisson structures would tend to act differently from segmented caissons. Active or passive stability failures at the caisson would have to be associated with an overall structural failure of the caisson, and truncation failures could occur even when the sand core is not frozen. These ring type structures could also experience a major bearing capacity failure, aided by seepage forces or precipitated by erosion of the berm. As in the case of artificial islands, effects of cyclic loading in the shear strength along slip planes needs to be considered.

Liquefaction is another failure mode which could occur during seismic events. The consequence of liquefaction would be spreading of the supporting berm and a buoyancy type bearing failure of the caisson, causing major settlement and possibly rotational tilting. Dredged or bottom dumped sand berms would be extremely vulnerable to liquefaction, unless the construction procedure includes compaction or compaction grouting.

Structural Failure: A structural failure would only be a foundation limit state if the structure relies on the support derived

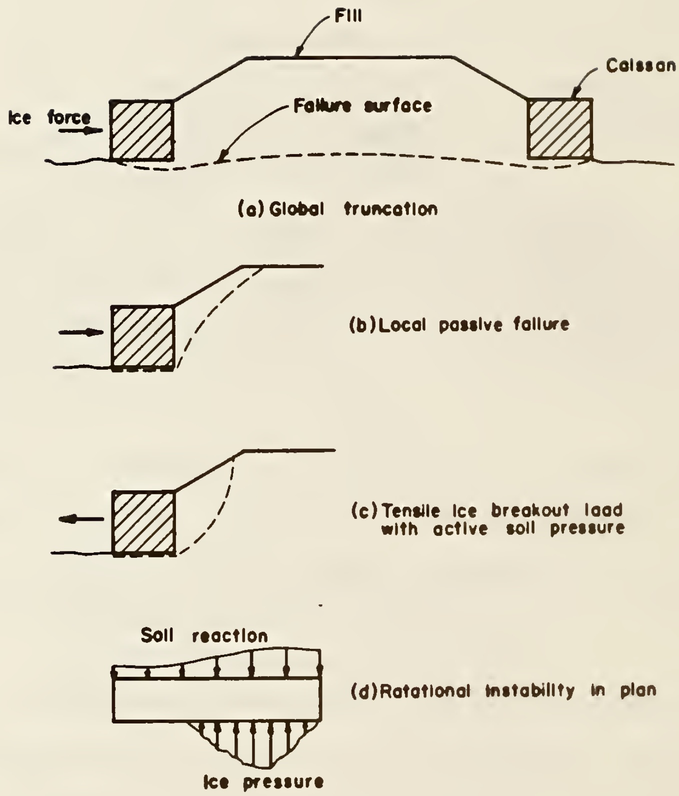


Figure 46. Failure modes for caisson retained islands identified in Ref.[18].

from the sand core or the supporting foundation soil, particularly when the pressure on the structure is non uniform. The amount of support derived from the soil, in turn, would depend on the method of placement and compaction.

Excessive Displacements: Potential causes of cumulative vertical and lateral displacements were already discussed. Cumulative lateral displacements would be more likely in the case of ring type caisson structures, which can translate as a rigid body.

Erosion: In general CRI's are designed so that the caissons are placed in the wave action zone. However the shelf of the sand berm could be subjected to gouging by grounded ice, associated with rubble piles which are likely to form in the vicinity of the island. Scour effects at the base of the caissons and on the sand berm slope may also be significant. The type and effects of erosion likely to occur would depend on the depth of the berm below mean sea level.

Ice Runup: Ice runup is more likely to occur when the sides of the caisson structure are sloped. The consequence could be overtopping of the caissons.

Excessive Construction Loads: Two loading conditions may be critical: lateral loads and uplift on the caissons associated with the placement of the hydraulic core; and pressures on the underside of the caissons during installation (up to 1500 kPa 48]).

4.4.7 Design Limit States for Rigid Mat Foundations

Included in rigid mat foundations are those for the floating caissons and for monopod structures. Even though these mat foundations are called rigid, attempts have actually been made to build in some structural "flexibility" (or ductility) by using steel rather than concrete for the mat [34]. The foundation could be placed on a prepared berm, or directly on the ocean floor with a minimum of preparation. In some instances it is planned to place the structures directly on the ocean floor and subsequently backfill any voids with sand [33]. In North Sea gravity structures concrete grout, rather than sand has been used to fill these voids. In the Arctic it is preferred to have sand fill, so that it will be possible at some future time to fill voids created by vertical settlements. Mat foundations usually incorporate skirts to enhance load capacity and facilitate underfilling or grouting and dowels for accurate positioning. They can be kept vertical by differential ballasting, and the space below the mat is vented to permit control of porewater pressures. The underside of mats can be level or can consist of dome shaped surfaces to improve structural efficiency.

Many of the structures with rigid mat foundations can be repositioned. This capability not only enables the operator to avoid extreme and rare loading conditions, but perhaps more importantly permits correction in positioning. This may enable the operator to accomodate major lateral displacements. The rigid foundation also permits remedial actions for settlements, by filling the voids created under the mat.

The load capacity of mat foundations can be augmented by ballasting, grout injection, spud piles and skirts. Short skirts (about 7m long) can also be used to ensure full development of the soil shear resistance near the soil-foundation interface.

Six categories of limit states are identified: Major stability failure; structural failure of the mats; failure of skirts during penetration; local failure at skirts by piping; excessive displacement or tilt; and scour. The failure modes associated with these limit states depend to a large degree on the geometry of the structure. Some of the critical conditions occur during construction.

Stability Failures: Different types of local and global stability failures, identified for North Sea gravity structures [45] are shown in figure 47. For Arctic structures, the ratio of horizontal to vertical loads is much larger. Thus sliding failures are more likely to occur than are bearing capacity failures. However, for structures with sloped sides, the ice forces have a substantial vertical load component and bearing capacity failures are possible. Similarly, when the water depth is great the overturning moment is substantial and can lead to rotational sliding. The shear resistance along these potential sliding surfaces depends on many parameters and will be discussed in the next section. The presence of ice gouges can reduce sliding resistance because of incomplete contact between the structure and the soil and non-uniformity in the density and shear strength of the upper soil layer.

Structural Failure of the Mat: While structural failure of the mat is a structural design limit state which is outside the scope of this report, it is precipitated by excessive soil pressures. The critical load could occur during construction, after set-down and before underfilling or grouting. However, critical contact stresses may also develop at the ridges of ice gouges. The effect

of ice gouges in sandy soils (which produce the most critical contact stresses) is illustrated in figure 48. Figure 49 shows some measured data on contact pressures on domed foundation sections for very dense sands in the North Sea [51].

Failure of Skirts During Penetration: Skirt penetration resistance has been considerable in North Sea structures and could conceivably cause a structural failure if the structural design is inadequate. Thus it is important to determine the maximum possible skirt resistance that the soil could develop.

Local failure at skirts by piping: While the structure is being

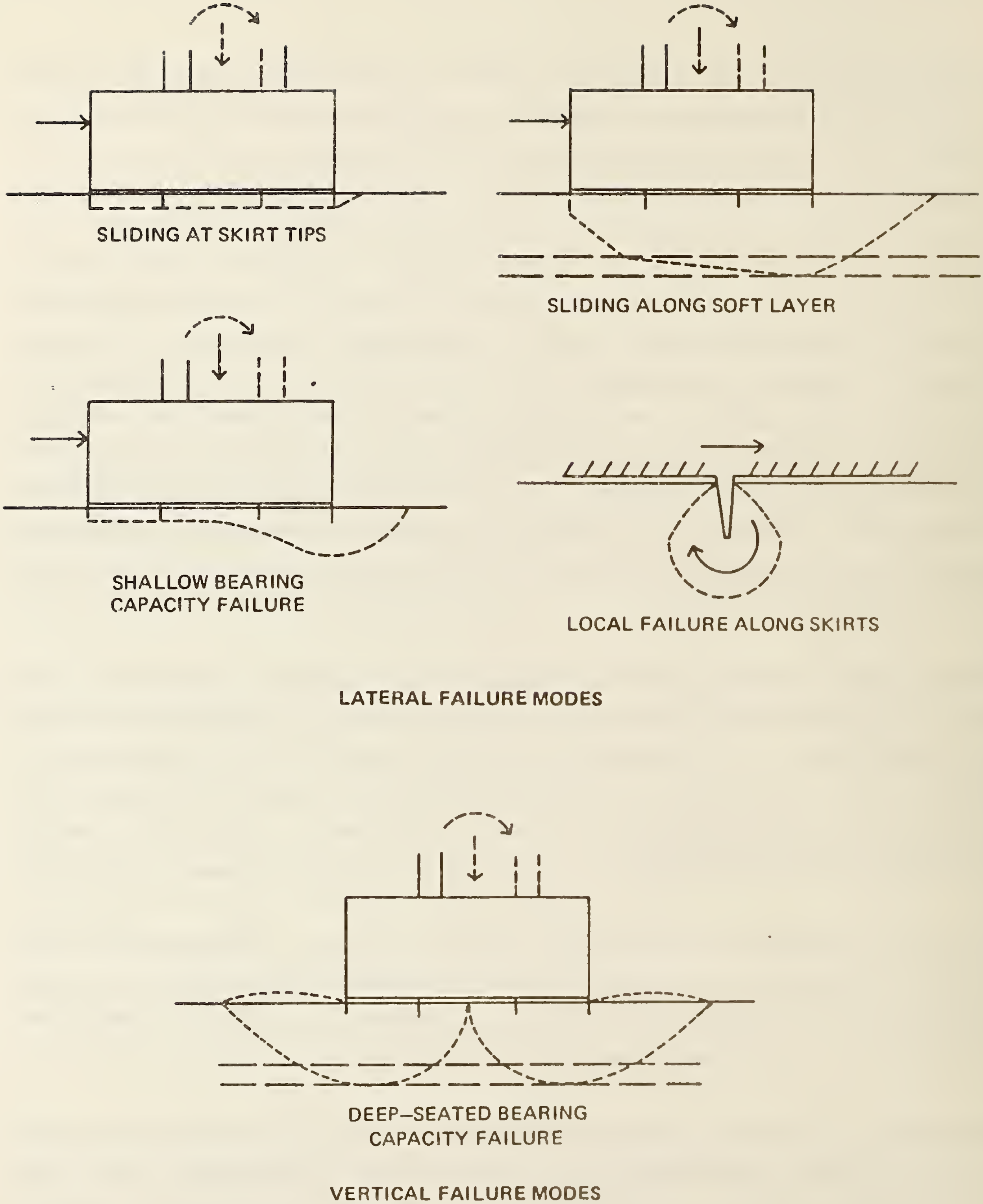
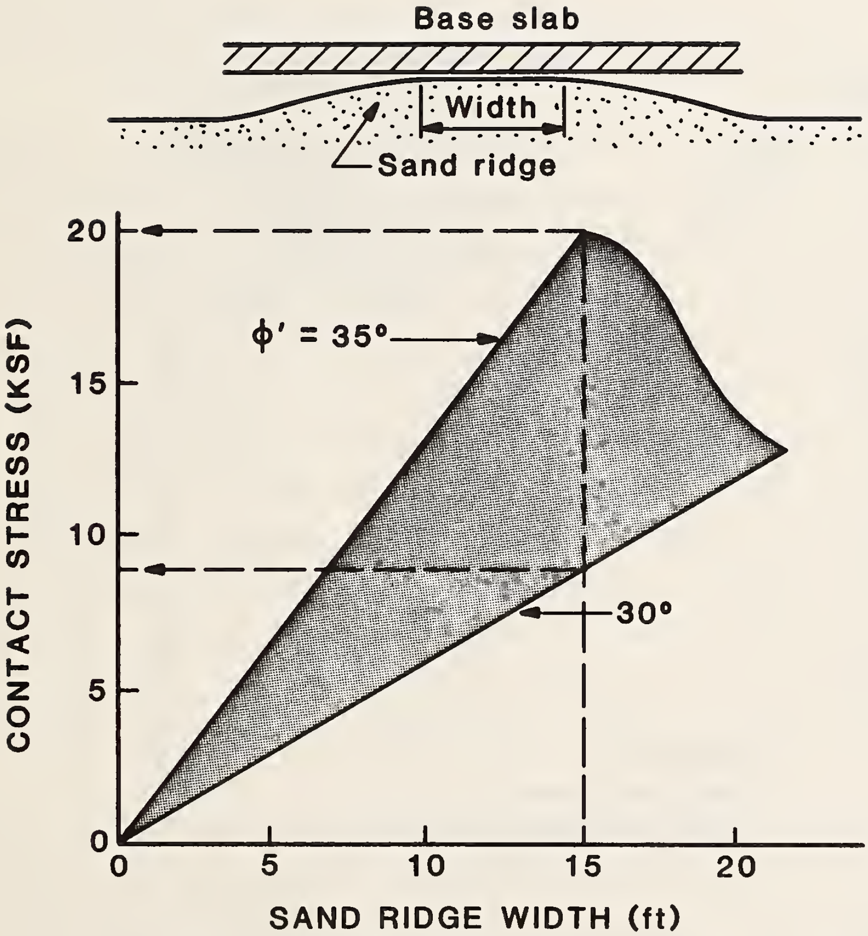


Figure 47. Failure Modes of Gravity Structures (from Ref.[45])



$$\frac{Q}{A} = \frac{1}{2}\gamma' B N_\gamma$$

Figure. 48. Contact Pressures at the Ridges of Ice Gouges.

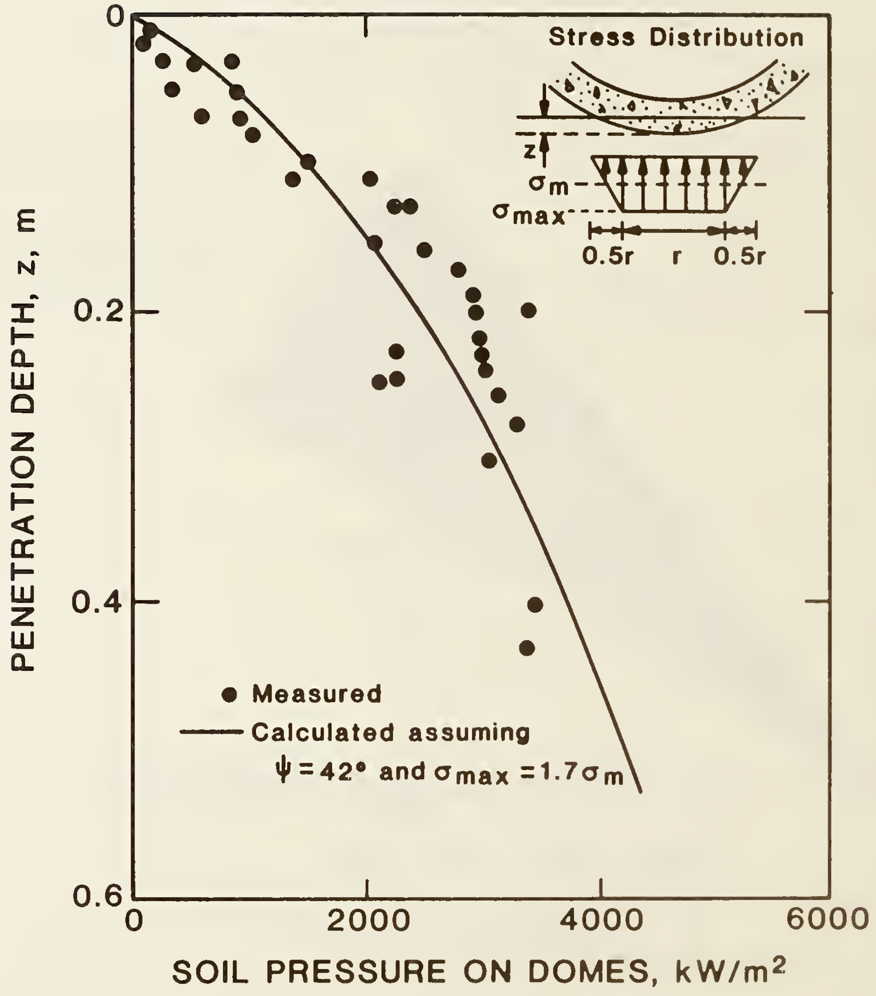


Figure 49. Contact pressures on Mat Foundations in the North Sea (from Ref.[51])

lowered and the skirts are penetrating some distance into the soil, high hydrostatic pressures develop under the foundation. This could cause a blowout around the skirts which could erode the area around the skirt and also transport considerable quantities of loose soil under the mat. Such failures occurred in the North Sea and prompted designers to provide for venting or pressure regulation of the space enclosed by the skirts. Such pressure regulation has also been used to control the vertical position of the platform during installation [51].

Excessive Displacements: Excessive static displacements or rotations can occur by the cumulative effect of ice loads, but also as a result of dislocation caused by a single ice impact. The determination of these displacements requires a realistic estimate of the number, magnitude and directionality of significant load cycles as a function of time, where a significant load cycle is associated with a load large enough to cause a permanent residual displacement. This assessment is difficult to make. For sandy soils, the threshold shear strain for a permanent displacement is about 10^{-2}%. Excessive dynamic deflections could conceivably occur in deep water as a result of wave and wind forces, however it is unlikely that these forces would have a significant effect on structures which are designed for the much

larger ice forces. Tolerable lateral displacements and settlements must be defined in terms of the tolerance of the structural and mechanical systems. These systems, in turn, should be designed to permit reasonable amounts of displacements, or to permit adjustments for displacements. For instance Andersen et al.[50] determined vertical settlements which would cause yielding of the well casings to be of the order of 100 - 150mm. Displacements under cyclic loads, rather than global stability failures could under certain conditions govern the design.

Scour: Scour could occur at the edges of mat foundations or on the slopes of sand berms. In general scour is not likely to be a problem in great water depths, but it could occur as a result of currents.

4.4.8 Design Limit States for Sand and Gravel Berms.

Sand or gravel berms are either dredged or dumped from barges. Dredged material can be fine (silty sand) but the angle of repose would be very flat. Dumped material has to be courser, as noted in Section 4.4.5. The angle of repose of dredged material placed under water is 1:15 to 1:20, however the material can be placed between bonds (dikes prepared by other placing methods) which have a steeper slope angle [47].

Sand berms or gravel pads that are constructed at a location prior to the placement of an Arctic gravity structure have been developed and applied in the Canadian Beaufort Sea. They have been found economically attractive for the following reasons:

1. A relatively long open water season (about 100 days)
2. An abundance of nearby good borrow material (coarse grained sands).
3. Several high capacity dredge systems are mobilized, capable of dredging, transporting, and placing fill.
4. Proposed exploratory drilling sites could be determined one or two years in advance of drilling.
5. Soil conditions are conductive to placing berms.
6. Berms have the advantage of acting as ice defense beaches that can ground large ice features and develop protective rubble pads around the platform.
7. Berms were a part of the evolution of offshore systems from gravel islands to caisson retained islands.

In the U.S. part of the Beaufort this site preparation has limited applications for the following reasons:

1. A short open water season (generally less than 30 days).

2. Good nearby borrow material is not abundant.
3. No high capacity dredge systems are mobilized, and their availability is restricted by legislation.
4. Development leases are such that at this stage it is unusual that an exploratory drilling site can be determined far enough in advance to allow construction of a berm (over several working seasons) without significant disruption of exploratory drilling. Berms at several candidate sites would be too expensive.

The limit states for berms were discussed in previous sections. However, the question arises, whether it is possible to densify the hydraulically placed material, which has a very low relative density, and therefore is vulnerable to liquefaction and excess pore water pressure buildup by even small cyclic loads, and generally develops low shear strengths. Soil improvement methods will be discussed in Section 4.5, but at this point it should be noted that they are both time consuming and expensive. Some options for soil compaction are discussed below.

Silts and clays can be generally compacted by surcharge, provided that their consolidation time is reasonably short. However sands and gravels can not be significantly densified by surcharging. Thus some mechanical compaction method must be used. Three methods which are reasonably successful on shore are vibroflotation, use of explosive charges, and dynamic compaction. Vibroflotation is quite expensive and is generally performed to a depth of $\pm$ 23m. Equipment for working under water has apparently not been developed. Blasting seems reasonably effective, but may be difficult and environmentally objectionable under water. A comparison of blasting and vibroflotation in an offshore project is made in Ref.[53]. Dynamic compaction seems more promising and appropriate. The latter method has been successfully used offshore [54,55,56]. It seems that it is necessary to provide a rock blanket of about 1-1.5m thickness on top of the fill. Drop height of the weight under water is 10m, effective compaction was achieved to a depth of about 5-6m. Weights used varied from 13 to 40 metric tons. The weights should be fluted to minimize water resistance. In situ monitoring to determine the density achieved, such as cone, dilatometer, or standard penetration tests would be necessary. Whether the logistics of transporting and lifting the heavy weights, building a rock blanket on top of the fill, and performing the in situ tests would justify the potential benefits derived from the compaction has not been determined. Gravel islands could conceivably be effectively compacted from a curface above the water line, which would eliminate the need for a rock blanket, increase the efficiency of the operation, and permit conventional in situ monitoring.

4.5 Resistance to Loads and Displacements

4.5.1 General

It has been noted in Sections 2 and 3 that there is considerable variation and uncertainty in subsurface and loading conditions. Specifically, ice loads will vary from location to location. In addition, there are smaller cyclic or variable loads caused by wind forces on the ice canopy ,multiple ice impacts or wave forces which may have a significant effect on the foundation resistance and displacements and which at this point in time cannot be very accurately characterized. Soils are also quite variable, in part because they were and still are partially frozen. Lenses or layers of frozen soil will not consolidate together with the adjacent unfrozen soil, and after they thaw they form pockets of soft soil or dicontinuities in soil profiles. Presence of past permafrost and many cycles of freezing and thawing probably also account for the irregular overconsolidation pattern shown in figure 17, which makes it difficult to accurately characterize strength properties at a shallow depth.

Analytical procedures and models by which foundation resistance associated with the previously discussed stability limit states can be calculated are generally available. However considerable uncertainty is associated with the prediction of cumulative residual displacements associated with smaller load cycles and with the measurement of the soil strength and stiffness parameters used to define the constitutive laws associated with analytical models.

4.5.2 Characterization of Soils

Soil properties that generally need to be defined are: shear strength (static, cyclic, residual, steady state); compressibility; permeability (coefficient of consolidation); maximum past pressure (overconsolidation ratio); stiffness (shear modulus); liquefaction potential; sensitivity (in clays); grain size distribution; Atterberg limits; natural moisture content; void ratio; and density. In addition we need information on soil fabric, anisotropy, fissures, and trapped gas and permafrost lenses which may cause us to misinterpret laboratory test results.

To measure these soil properties we have two types of methods at our disposal: in situ measurements and laboratory testing. In situ measurements have the advantage of measuring soil properties in place, without the disturbances caused by sampling and release of confining pressures, and without the scale effect associated with small laboratory samples. On the other hand, most of the in situ tests also disturb the soil, and our in situ testing capabilities are very limited since we cannot controll the boundary conditions which dictate the state of stress and failure mode.

In Situ Soil Exploration: methods most commonly used off shore are: geophysical exploration; static cone penetration (CPT)

tests; vane shear tests; pressuremeter tests; and gamma logging tests. Other teste which could yield useful results are standard penetration (SPT) tests, dilatometer tests, self boring pressuremeter tests, and plate loading tests.
Geophysical tests generally provide information on the velocity of compression and shear wave propagation and on soil stratification. They also permit us to locate pockets of trapped gas and relict permafrost. The compression wave propagation velocity in saturated soil is generally controlled by the elastic properties of the pore water and therefore provides little information on the soil skeleton. The shear wave propagation velocity, however, provides information on the shear modulus associated with very small strains (of the order of 10^{-7}), G_{max}, which in turn has been correlated with shear moduli at larger strains for various soils (for instance [57]). Thus it can be said that geophysical measurements provide information on the stiffness of the undisturbed soil deposits. This information could be particularly useful in the determination of the susceptibility of granular soils to excess pore water pressure buidup under cyclic loading.

Cone penetrometer (CPT) tests expand a cylindrical cavity in the soil as the penetrometer advances. Tip and side resistance are measured separately. The test gives excellent information on the stratification of the deposits. Even though the penetration resistance, particularly in granular soils, is a complex function of strength and deformation properties, very useful information for preliminary design purposes can be derived. Application of the test in North Sea foundation practice is summarized in Ref.[58]. Undrained shear strength of clays can be reasonably estimated, particularly if the plasticity index and the overconsolidation ratio (OCR) are known. The angle of shearing resistance of sands can also be reasonably estimated if the OCR is known. However, the latter parameter may be difficult to determine, unless intervening clay layers are present for which the OCR can be determined in odometer tests. Schmertmann

[59] also developed correlations for determining the relative density of sands. The CPT does not retrieve a soil sample which is a major drawback. However, an indication of the grain size of the material can be derived from the "friction Ratio" (the ratio between the side and tip resistance). CPT test results can also be used to assess liquefaction potential [60], however a reasonably reliable assessment would require information on D_{50}. This information can then be used to estimate the the SPT blowcount which would be obtained. The SPT blowcount, in turn, has been correlated with liquefaction potential on the basis of extensive data [61]. CPT-SPT correlations are shown in figure 50. Figure 51 shows correlations between SPT blowcount and liquefaction potential. CPT tests have also been used very successfully to predict soil resistance to skirt penetration and pile capacity. Prediction of skirt penetration resistance is illustrated in figure 52 from Ref.[58]. Similarly, resistance to spud pile penetration could be predicted.

Vane shear tests give useful information on the undrained shear strength of clays and cohesive soils that were in an undrained mode during shear. Extensive experience has been gained with this test.

Pressure meter tests have been extensively used and correlations with soil strength and stiffness were established. These tests provide more reliable information on in situ confining pressures than other in situ tests. However confining pressures in sandy soils are difficult to obtain because of the sensitivity of the measurements to disturbances. In the latter case, self boring type pressure meters would have a better chance of yielding sucessful measurements.

Gamma logging tests are used in offshore exploration to log soil stratification through casings. However attemps to use the soil emission for density logging were not very successful.

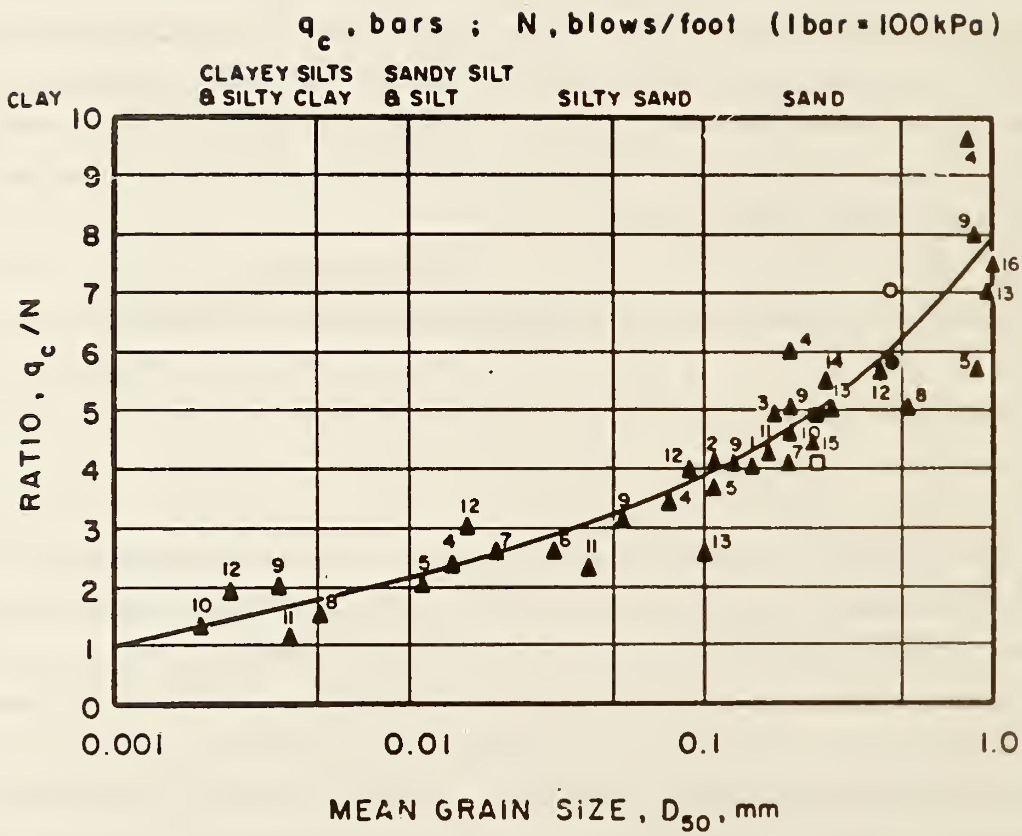


Figure 50. Data on CPT-SPT Correlations (from Ref.[60])

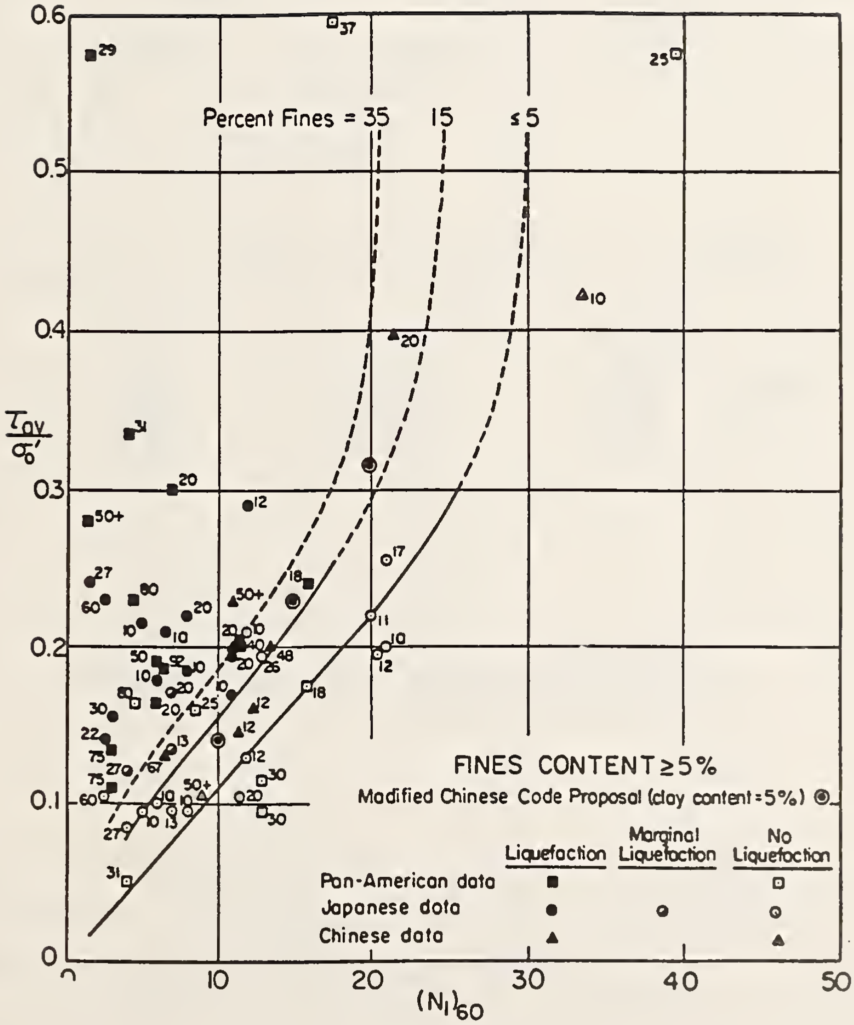


Figure 51. Data on Correlations Between SPT Blow Count and Liquefaction Potential of Sands and Silty Sands

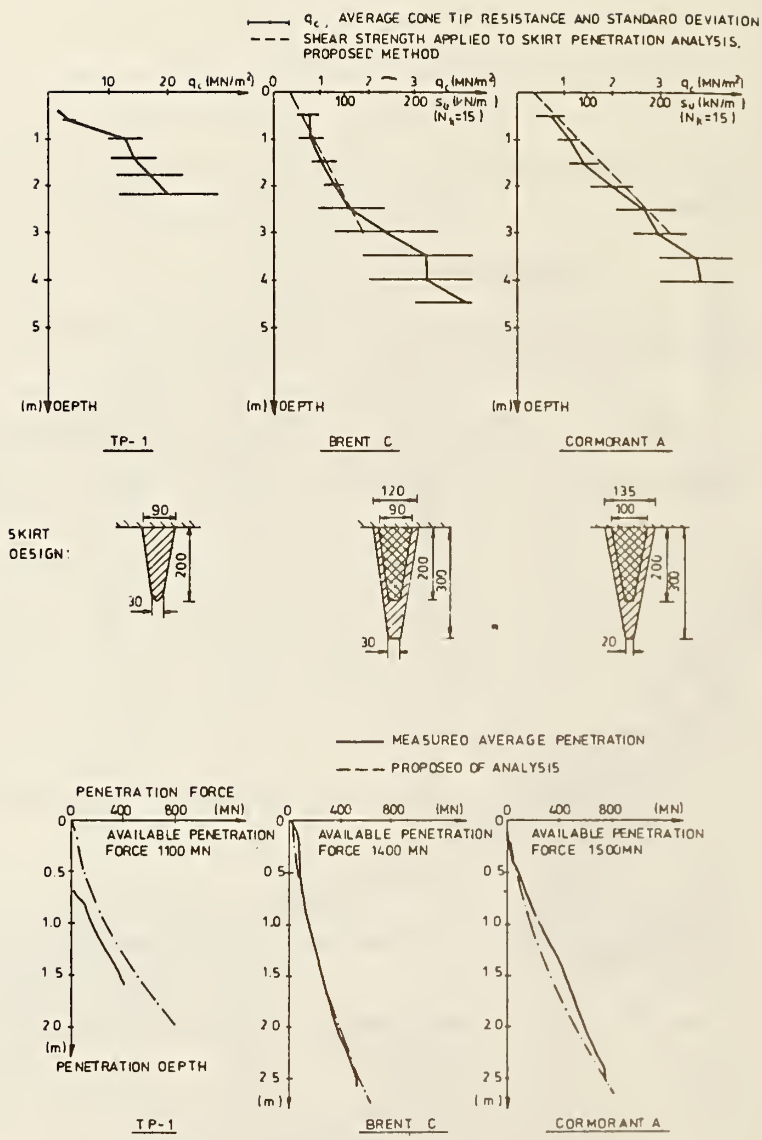


Figure 52. Predicted and Back Calculated Skirt Resistance (from ref.[48]).

Standard penetration tests are well known and need not be discussed. Except in the case of assessing liquefaction potential, the CPT yields more reliable and repeatable results than the SPT. However the SPT will retrieve disturbed soil samples which yield much valuable information.The SPT is generally more reliable in granular soils.

Dilatometer measurements are of relatively recent origin but they show promise in measurements related to soil stiffness and in situ confining pressures. The test is much more inexpensive than the pressuremeter test and is lately extensively used in the monitoring of dynamic compaction.

Plate loading tests have been used in the North Sea to a minor extent, primarily to predict contact pressures. The plate used had a 300mm diameter, thus requiring a substantial downward thrust. Another plate loading test that could be used is the screw plate test. Large plate test may be of interest, since it has been determined as a result of a great number of tests with a 865mm diameter plate, that calculations based on laboratory test results for clays tend to underestimate stiffness (settlements) and overestimate strength [62]. While these conclusions may not apply to the particular soils investigated, insitu plate loading tests could help calibrate information obtained from laboratory or other in situ tests.

Laboratory tests: Laboratory tests permit us to apply controlled boundary conditions and separate the many variables involved in soil strength and stiffness by changing one parameter at a time. However they also have severe drawbacks. these include:

1. Scale effects. Not only do we have to upscale the resuls by several orders of magnitude as we go from a laboratory sized sample to the full scale problem, but there is also a size effect on strength and stiffness, particularly when the soils are fissured.

2. Sample disturbance. It is difficult, and in many instances impossible, to take undisturbed samples. Not only is there a distortion from the center to the periphery of the sample, but we also remove the confining pressures in the process of sampling (and as we pointed out, these confining pressures are very difficult, and frequently impossible, to determine). Undisturbed samples of cohesionless soils are very difficult to obtain, even though some freezing techniques were recently successfully applied.
3. Boundary conditions. While we attempt to create a determinate state of stress in the soil volume under test, there are stress and pore water pressure gradients within the sample. The problem is particularly severe in the simple shear test, where stress gradients develop at the end plates.
4. Anisotropy and direction of principal stresses. Soils are generally consolidated under gravity load and thus are not isotropic. While in the simple shear test the directions of the principal stresses resemble those in the field, this is normally not the case in the triaxial test. Soil strength may change as we rotate the axis of the principal stresses.
5. Load history simulation. Soil strength is stress history dependent. However it is virtually impossible to exactly simulate the actual loading sequence anticipated in the field.

Sampling: The difficulties in obtaining undisturbed samples have been already discussed. In the offshore, samples are obtained by driving or jacking. Drive samples are generally disturbed. Frequently samples are also disturbed by internal gas pressures. In order to insure the quality of offshore samples it is desirable to use X ray techniques to examine the samples while the exploration is in progress.

Determination of shear strength: Generally the undrained shear strength has to be determined, however the point in time after the application of the full vertical loads when the critical shear stresses are expected to occur may be an important consideration when the tests are conducted. Generally five different types of test are used: unconfined compression tests; Undrained, consolidated triaxiar shear tests, which can be compression tests (the major principal stress is increased while the minor principal stress [cofining pressure] is held constant), extension tests (the major principal stress is held constant while the confining pressure is decreased), or torsional shear tests (the vertical and horizontal stresses are held constant while torsion is applied); and undrained consolidated direct simple shear tests. Unconfined compression tests are generally performed for clays and can give a good indication of undrained shear strength if good undisturbed samples are available. Figure 53 which is taken from Ref.[51] shows the location on a hypotetical slip plane for which triaxial compression, direct simple shear, and triaxial extension tests are applicable. The torsional triaxial test could be used instead of the direct simple shear (DSS) because of the similarity between the state of stress and the direction of applied stresses. For failures generated by a large horizontal load and a relatively moderate vertical load the state of stress would be most closely approximated by direct simple shear or torsional triaxial tests.

The effect of the deformation modes represented by the above discussed tests on the shear strength is illustrated by figure 54, which is taken from Ref.[63] and was obtained for Santa Barbara silt, a soil somewhat similar to the Beaufort silt. Note that triaxial compression produces the largest shear strength, triaxial extension the smalles, and simple shear falls in between.

Another important parameter that must be considered in these tests is the effect of the consolidation pressure applied in a drained condition before the undrained shear test is performed. This consolidation pressure corresponds to the prevailing

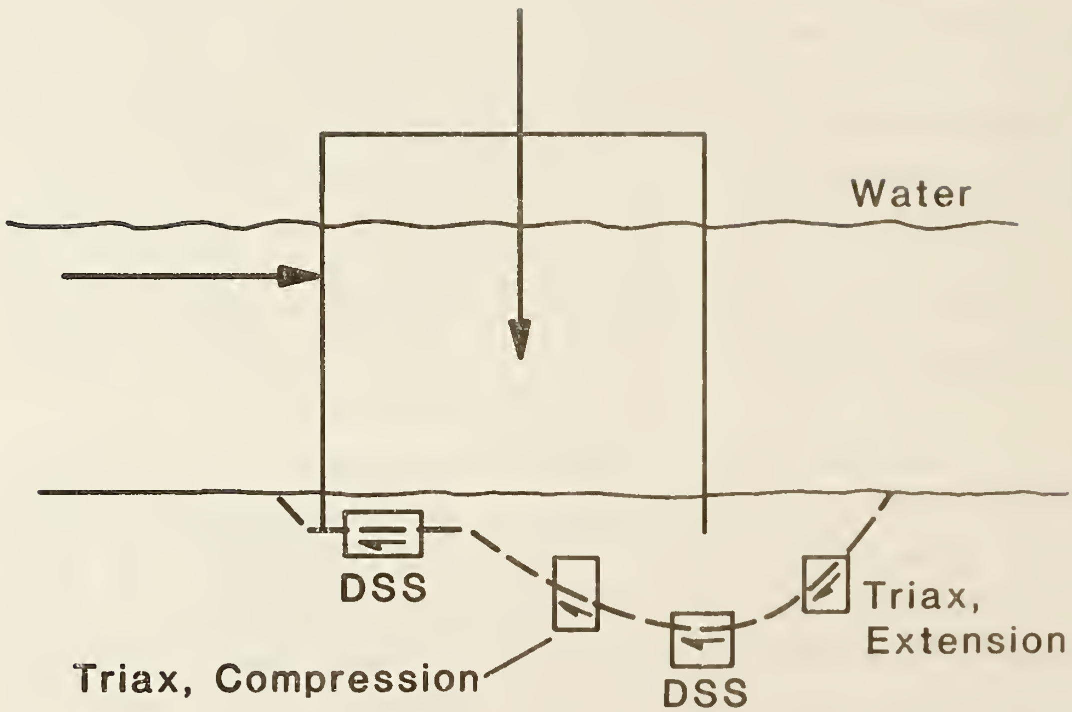


Figure 53. Deformation Modes associated With Shear Failures (taken from Ref.[51]).

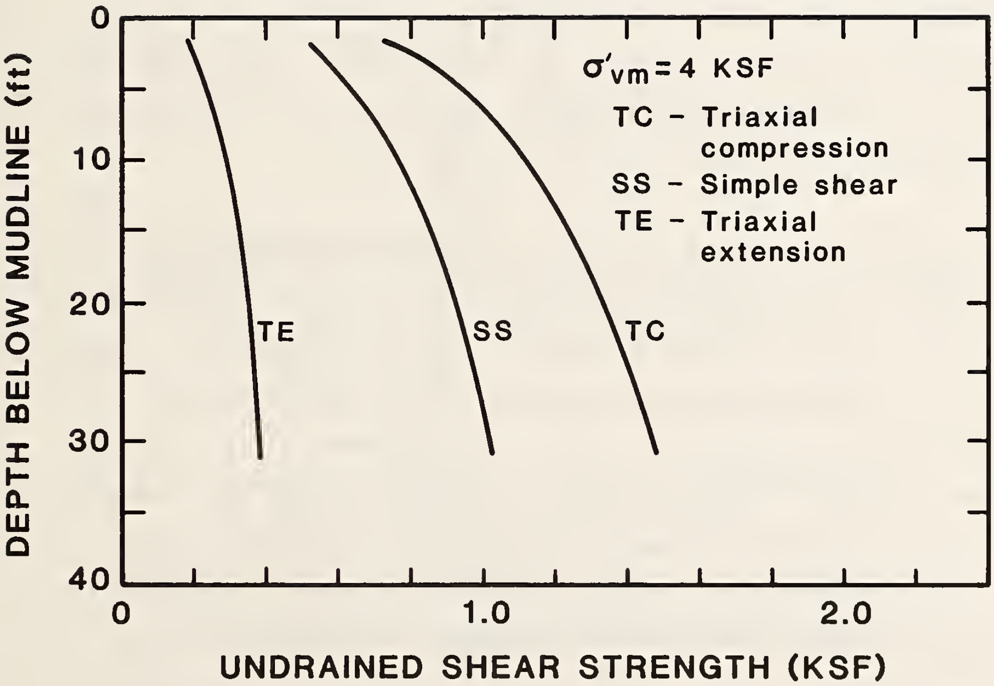


Figure 54. Effect of Deformation Modes on Shear Strength (from Ref.[63])

confining (or overburden) pressure in the field, and the additional pressure exerted by the structure. Wether or not this additional pressure is applied in the test will depend on whether the structure had time to consolidate before the critical horizontal load occurred. The effect of this parameter is illustrated in figure 55 from Ref.[63].

A third parameter affecting shear strength is the OCR, or the apparent maximum past pressure. The magnitude of this effect in Santa Barbara silts is shown in figure 56 (also from Ref.[63]). This effect is of interest because the Beaufort offshore deposits are heavily overconsolidated at a shallow depth. Since the pattern of overconsolidation is rather irregular, and overconsolidated soils frequently have a tendency to form fissures and loose some of their shear strength when horizontal confining pressures are reduced [64], great care must be exercized when the shear strength of these soils is evaluated.

Effect of Cyclic Loading on Shear Strength: When relatively small cyclic loads are applied to soils in an undrained condition (or within a time frame where pressure dissipation by drainage does not have a significant effect) they will cause a buildup of excess porewater pressures. This phenomenon will occur, even if the soils dilate under large shear strains. Somewhat of an explanation for this porewater pressure buildup can be derived from an examination of figure 15. Even soil A, which is dilative under large axial strain exhibits an initial increase of excess porewater pressure (u). As excess porewater pressures increase, effective stresses, and thus shear strength, decrease. In relatively pervious soils, the buildup of porewater pressures is frequently checked by drainage and the pressures dissipate soon after the cyclic load ceases. In soils of low permeability, however, the excess porewater pressures may affect the shear strength for a long time after the cyclic loads were applied. An extreme example of the potential effect of cyclic loads on the

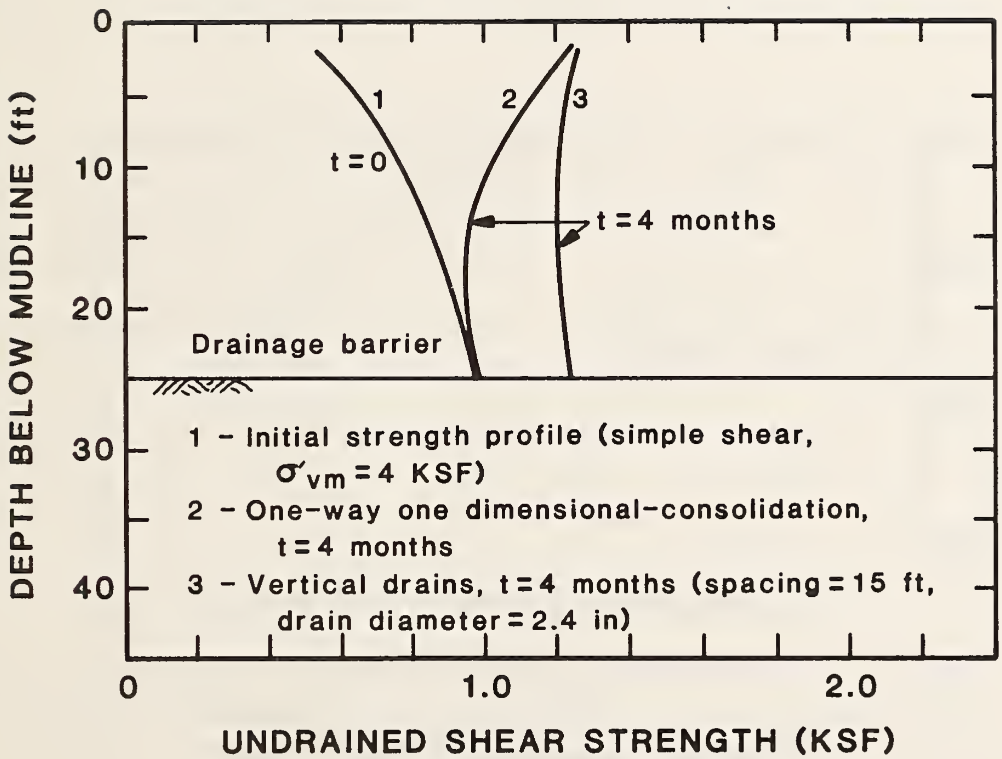


Figure 55. Effect of Consolidation Under the Weight of the Structure on Shear Strength (from Ref.[63]).

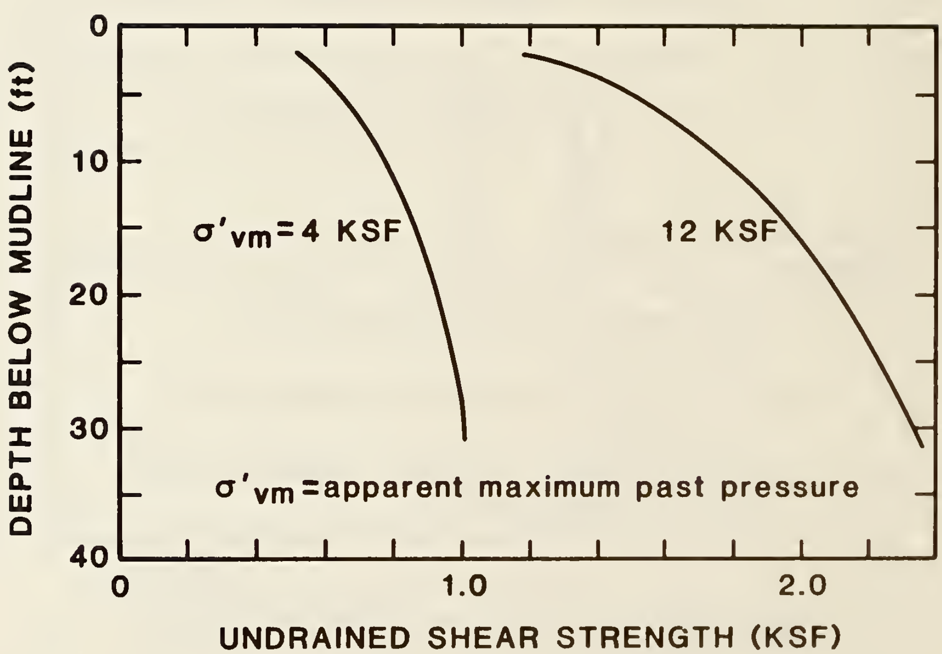


Figure 56 Effect of Apparent Overconsolidation on Shear Strength Profiles (from Ref.[63])

static shear strength is shown in figure 57, from Ref.[49]. The data in the figure were obtained from large diameter plate tests. This effect is more severe in overconsolidated soils than in normally consolidated soils. The effect is very significant in North Sea structures, where it is generally assumed that the maximum wave is going to occurr at the end of a 6 hour storm, and the 6 hour storm is then simulated by a cyclic loading sequence, whose effect on the failure load is evaluated. In the Beaufort Sea the effect of cyclic loads is probably less pronounced, since the cyclic loads are much smaller in relation to the maximum load than in the North Sea loading. However we cannot afford to diregard this parameter, particularly for soils of low permeability.

Another case that needs consideration is cyclic shear strength (the number of cycles to failure under a given stress (or strain) level). A typical set of data for North Sea clays is shown in figure 58 from Ref.[49]. It can be seen from the figure that overconsolidated soils are more sensitive to strength reduction by cyclic loading than normally consolidated soils. This undrained cyclic load effect should not be confused with the case where there is sufficient time for the excess pore water pressures to dissipate. In the latter case, prior load cycles are likely to increase, rather than decrease the shear strength.

Determination of Compressibility: Compressibility can be determined by odometer tests or drained triaxial tests, which do not need to be further discussed. However, in the Beaufort Sea, a careful evaluation needs to be made whether settlements could be affected by enclosed permafrost or entrapped gases. Predictions based on odometer tests are reasonably reliable with respect to magnitude of settlements, however the estimates of the time element invoved in consolidation settlements is usually not very reliable. Most soils are ortotropic with respect to their permeability because of the manner of their deposition and pre

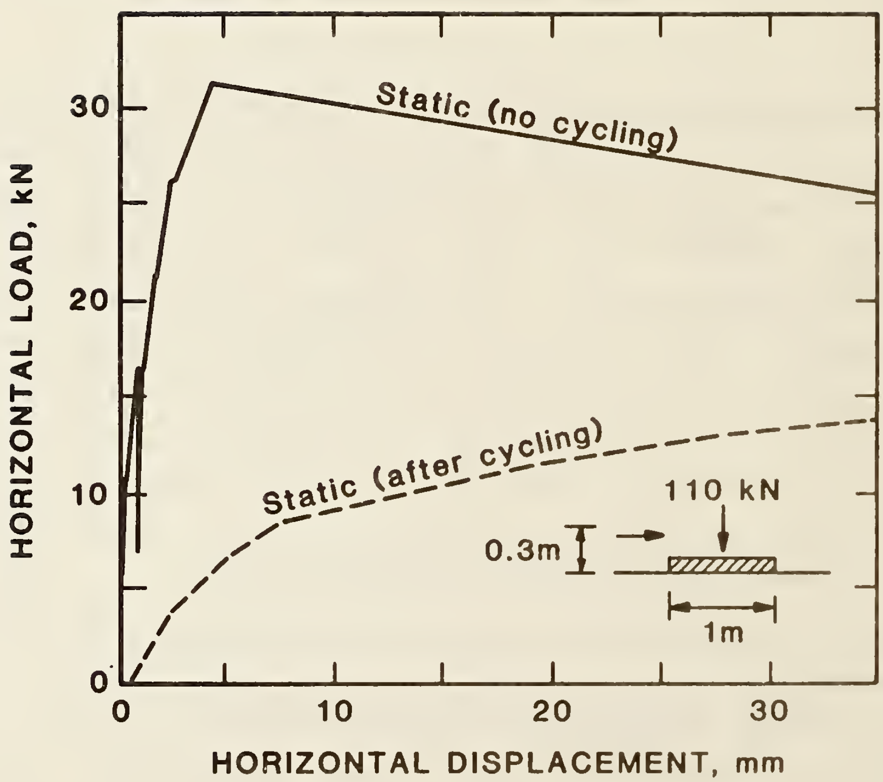


Figure 57. Effect of Cyclic Loads on Monotonic Shear Strength in Undrained Condition (from Ref.[51])

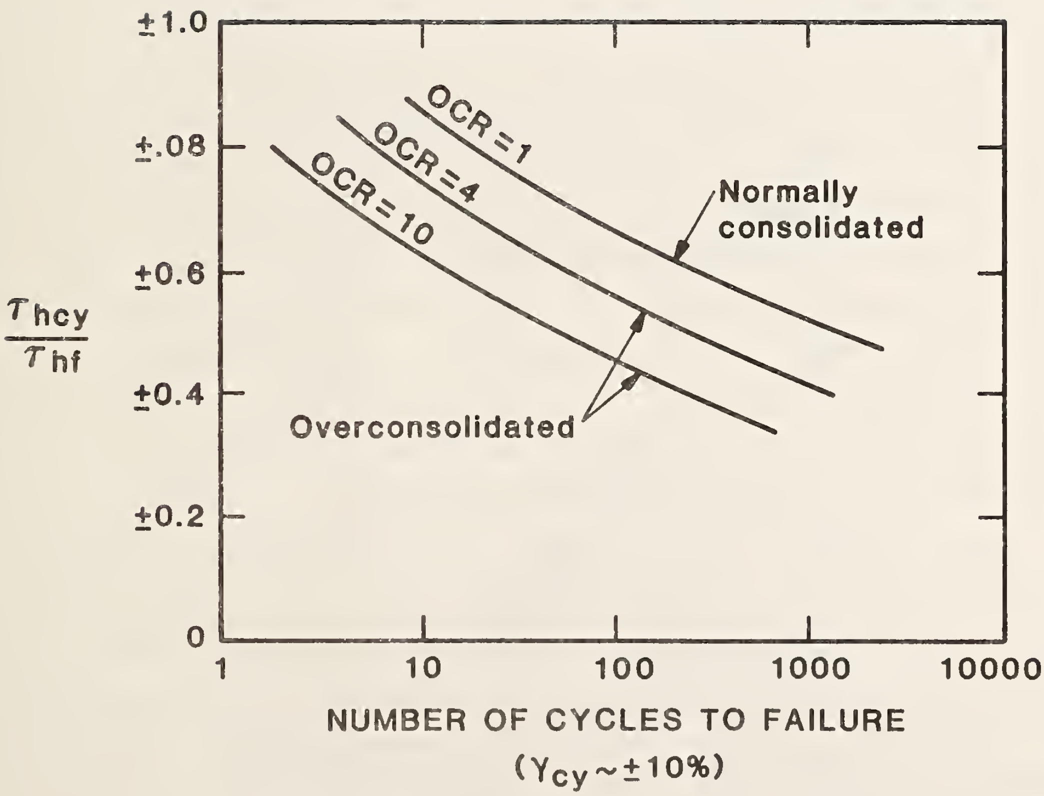


Figure 58. Undraind Cyclic Shear Strength of North Sea Clays (from Ref.[51]).

loading, and permeability in the horizontal direction tends to be much greater than that in the vertical direction. This fact leads to a tendency to overestimate the settlement time, which is an error on the conservative side. Analytical methods for considering anisotropy and the geometry of boundary conditions are available (for instance [65]). As previously noted, it is important to estimate the consolidation time because of its effect on the shear strength of the supporting soil.

Liquefaction, Cyclic mobility and Steady State Strength: Liquefaction is a matter of concern when dealing with sands and silts. However as previously noted, cyclic strength may be a matter of concern with most soils. Seismic loads are the primary concern, but dynamic loads associated with multiple ice impacts may also have to be considered. As previously noted, excess porewater pressures will buid up in most saturated soils subjected to undrained cyclic loading. When the cyclic loads continue for a sufficient number of cycles and are repeated rapidly enough the effective confining pressure will approach zero, and soils with no cohesive strength will experience initial liquefaction. In that stage soils lose their shear strength and act like heavy liquids. Whether or not this condition can lead to major stability failures depends on whether the soils will dilate when subjected to further shear deformation. In general, structures on loose sands or silty sands will experience catastrophic failures when initial liquefaction occurs, while structures on dense soils will experience limited displacements. Some indication of the effect of liquefaction on sands of various relative densities can be derived from figure 59 which shows shake table data presented by Seed [66]. It can be seen that sands with relative densities below 40% liquify and cause catastrophic failures and sands with relative densities of less than 60% are subject to large shear deformations. Whether or not initial liquefaction is likely to occur as a result of a given cyclic load can be determined from a set of cyclic triaxial tests such

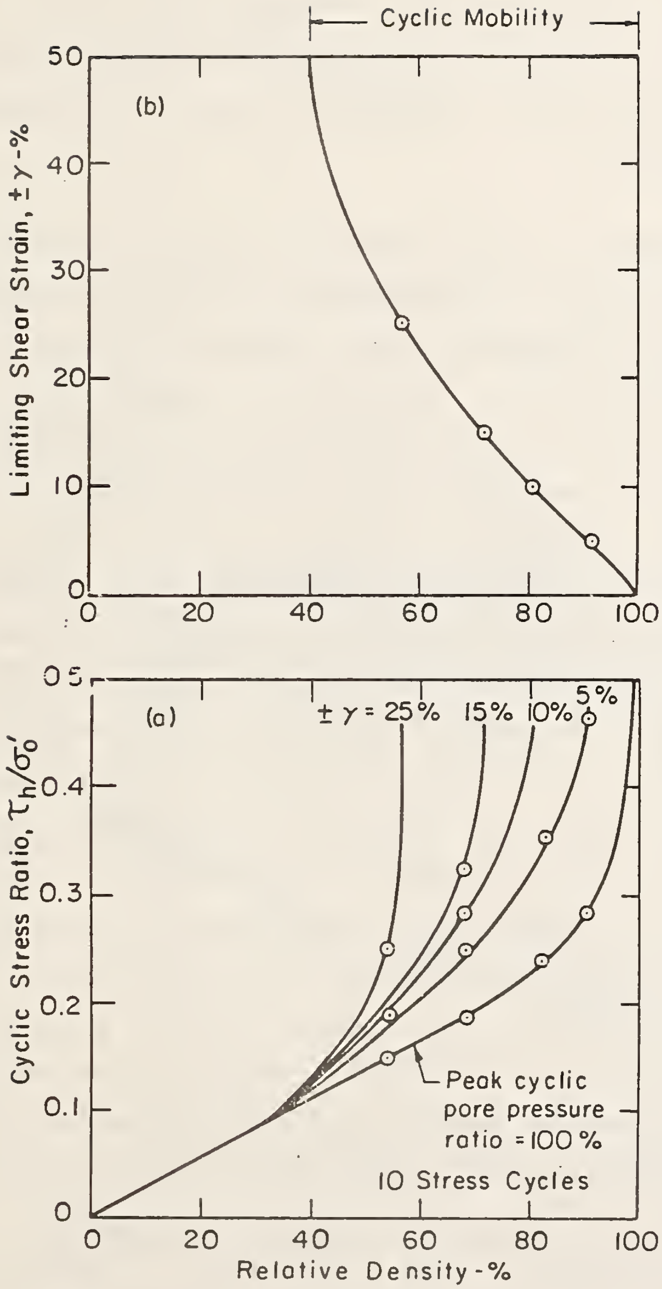


Figure 59. Shake Table Data on Cyclic Mobility (taken from Ref.[66]).

as those shown in figure 60 which is taken from Ref.[67]. The trouble is that it is very difficult to obtain undisturbed samples of granular soil, and there is no assurance that reconstituted laboratory samples are representative of field conditions. Thus at the present time empirical curves such as the one shown in figure 51 are most frequently used to estimate susceptibility to liquefaction.

There are two concepts which should receive consideration when evaluating susceptibility to liquefaction: The threshold strain, and the steady state strength. The threshold strain concept is based on data obtaind for a variety of sands, but has not yet been extended to silts. It is based on the hypothesis that there must be intergranular sliding if the volumetric strain which causes pore water pressure buidup is to occur. It has been observed experimentally and shown theoretically that below the threshold strain of 10^{-2} % (0.0001) no porewater pressure buildup in quartz sands is possible. Thus when it can be shown that this strain will not be exceeded there is no need to worry about liquefaction. Whether or not this strain is likely to be exceeded can be determied from an analysis using shear modulus values derived on the basis of geophysical measurements. The concept can probably also be extrapolated to predict porewater pressure buidup above the threshold strain, but not enough research has been done to accomplish this end. Figure 61 shows threshold strain data [68]. Figure 62 shows trends of porewater pressure buidup above the threshold strain from experimental data [67]. The threshold strain increases with an increase in the OCR [68a].

Following the hypothesis that porewater pressure buildup is primarily related to the shear modulus, some attempts have been made to correlate shear wave velocity measurements with liquefaction potential [69]. The results of this work are shown in figure 63. Unfortunately there are not enough data at this time. The use of geophysical measurements for liquefaction

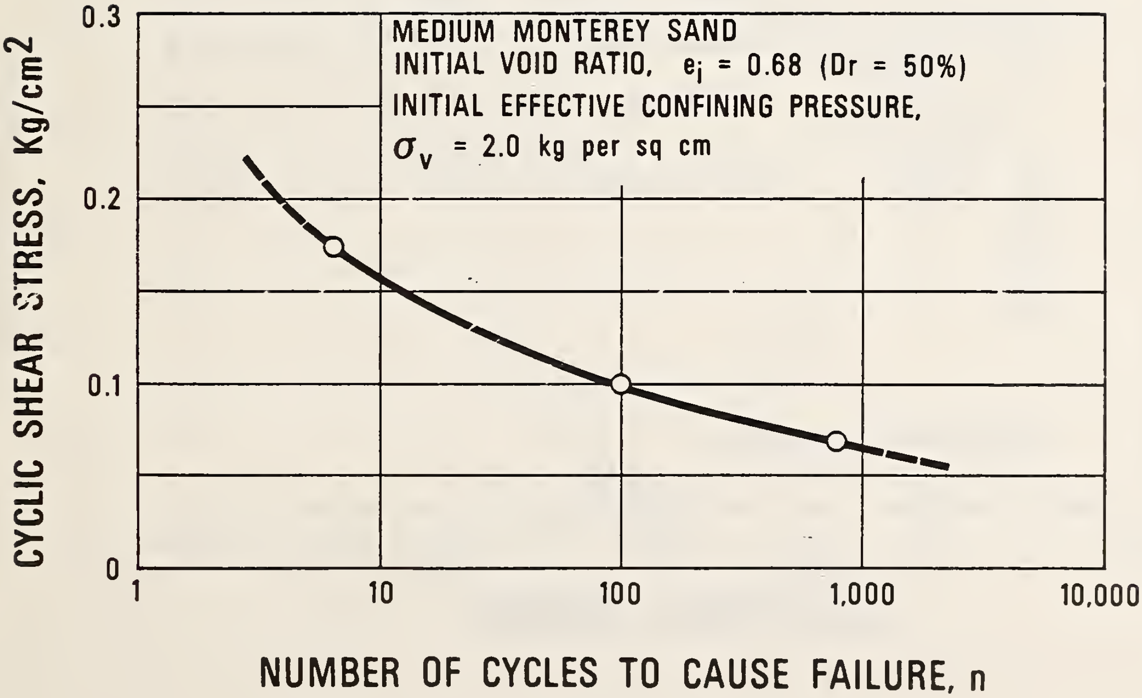


Figure 60. Stress Controlled Cyclic Triaxial Tests of Reconstituted Sand Sample (taken from Ref.[67]).

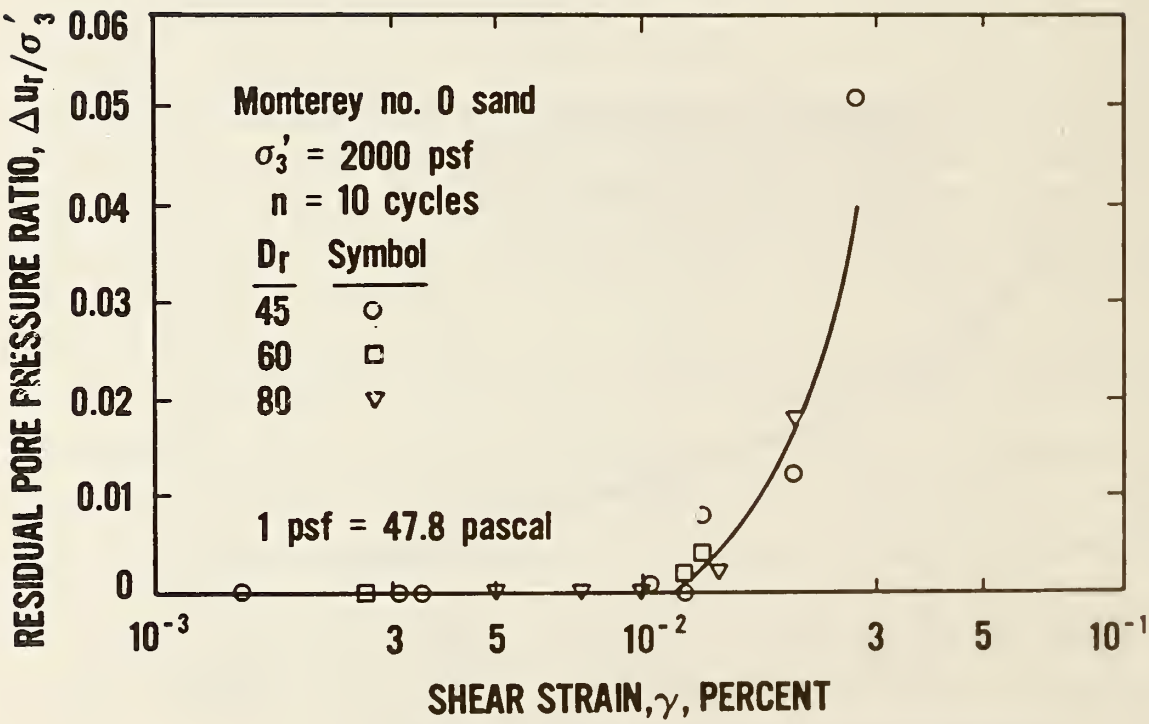


Figure 61. Observed Threshold Strain (taken from Ref.[68])

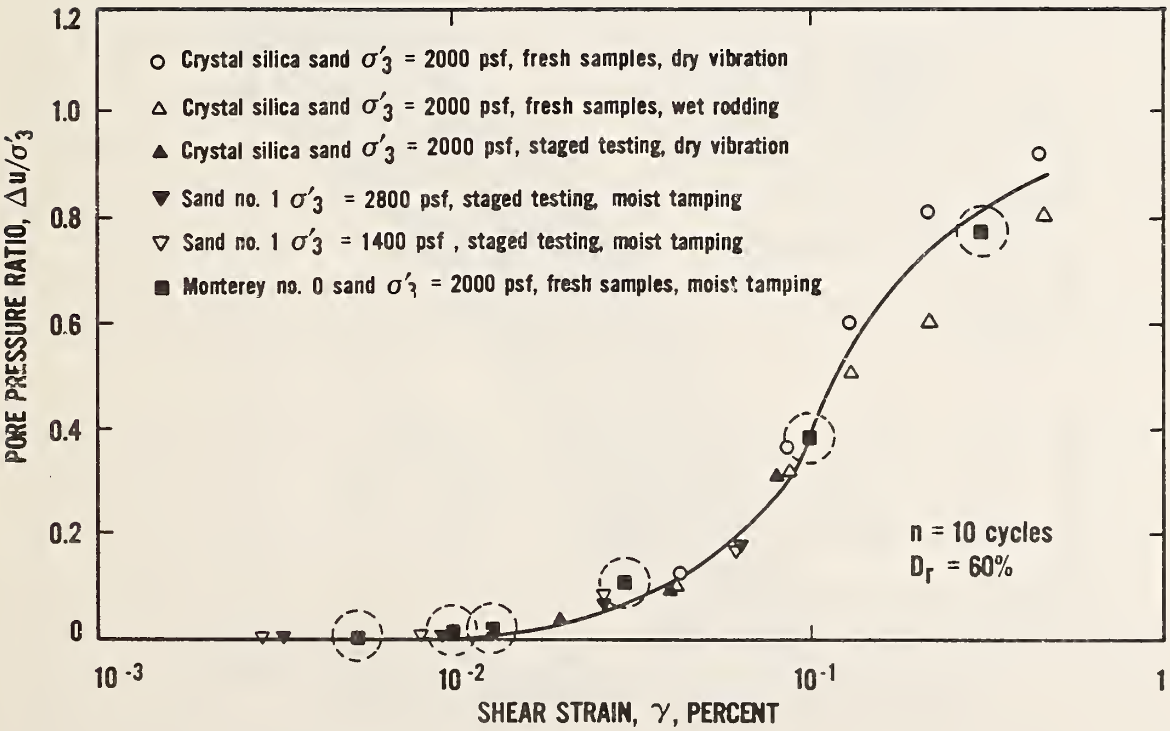


Figure 62. Correlations Between Cyclic Strain and Pore Water Pressure Buildup (from Ref.[67]).

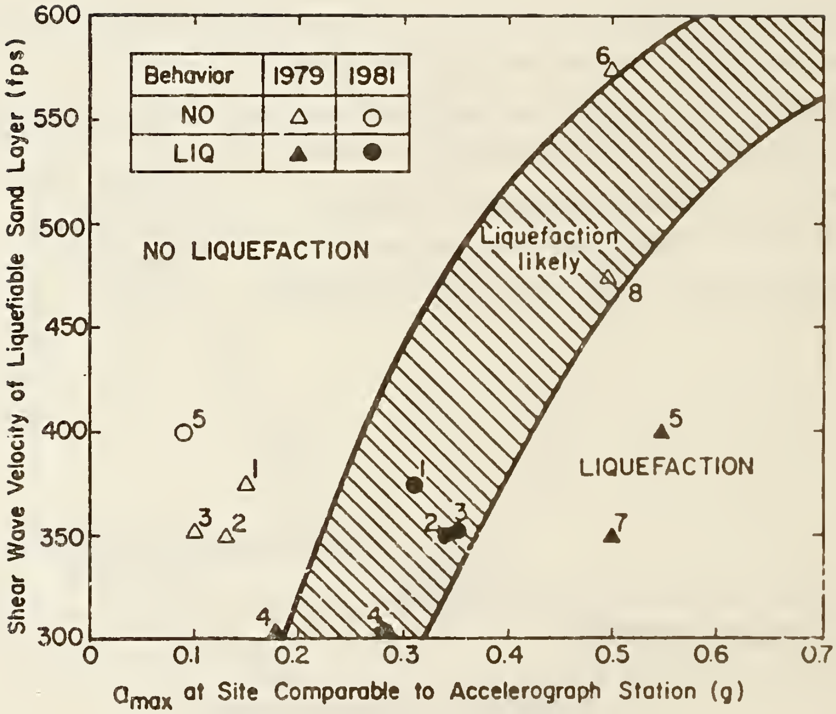


Figure 63. Correlation Between Shearwave Velocity and Liquefaction Potential of Sands (from Ref.[69]).

eveluation is an attractive option, particularly in the preliminary stages of a project.

The steady state strength concept [70] is based on the observation that saturated soils, subjected to large shear deformations, will reach a stage of ideal platic flow, where further shear deformation is not associated with any volume change. In this stage the effective confining pressures remain constant. The steady state strength is a unique function of density and grain size characteristics, and independent of soil fabric or initial confining pressures. A typical steady state strength curve is shown in figure 64. The steady state strength could be approached from the left side of the curve in figure 64 by volume expansion if the material is dense and the initial confining pressures are relatively small, or from the right side of the curve by volume contraction if the material is loose and the initial confining pressures are relatively high. If the shear strength required to support the structure is smaller than the steady state strength, a catastrophic failure cannot develop. Conversely, if the required shear strength is higher than the steady state strength a catastrophic failure will result from initial liquefaction.

The steady state strength approach is attractive because the parameters of the problem, steady state strength and in situ density, can be evaluated. The steady state strength line can be determined in the laboratory from disturbed samples (even though the authors recommend to also test undisturbed samples).
In situ density must be measured very accurately. The reason for this requirement is that the steady state line, particularly for soils with subrounded or rounded grains, is rather flat (a small change in density is associated with a large change in steady state strength). The steady state strength approach is also an effective tool for evaluating the resistance of a foundation after large shear displacements occurred and therefore could be used to estimate displacements caused by large ice features.

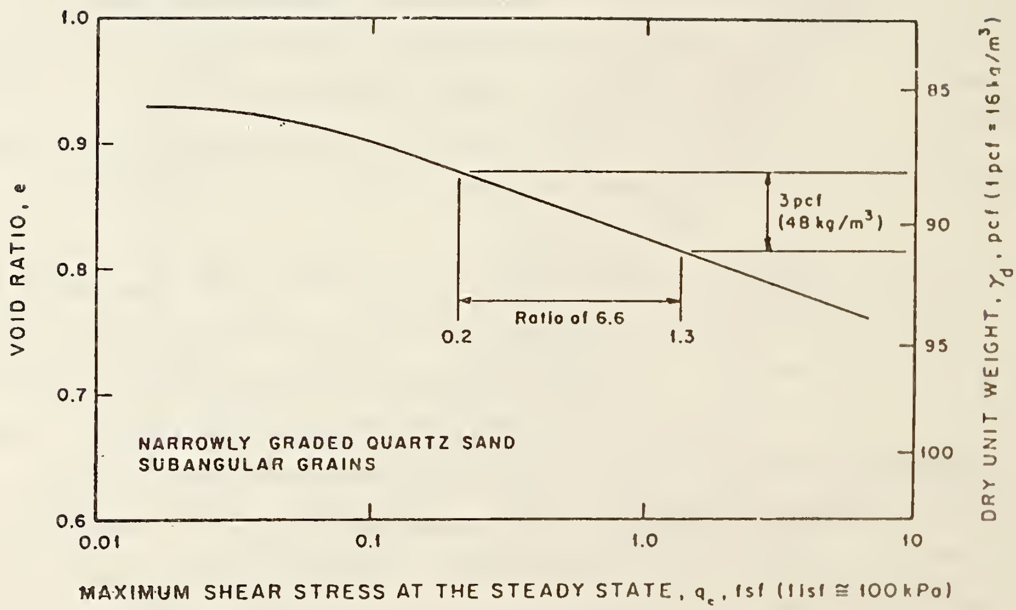


Figure 64. Steady State Strength (taken from Ref.[70])

Some analytical models where developed which are designed to predict pore water pressure buildup and cyclic mobility. One such model developed by Arulanandan et al.[71] is correlated with electrical in situ measurements. Whether these measurements can bu used offshore has not been established.

Field and laboratory tests associated with the previously dicussed methods of liquefaction evaluation would be SPT and CPT tests for empirical approach, stress controlled triaxial tests of reconstituted samples for the traditional approach, and shear wave velocity measurements for the threshold strain approach. For the steady state strength approach very accurate in situ density measurements are needed (piston samples are recommended). To establish the steady state line for the material, shear strains have to be very large. This can be in some instances accomplished by triaxial tests with special provisions for sample expansion near the end platens. Larger strains can be achieved in torsional triaxil tests and in simple shear tests. For clays laboratory vane shear tests may have to be performed.

The State Parameter: The concept of the steady state strength has been used more recently in conjunction with the stability problem associated with the placement of hydraulic fills for sand berms (dredged or bottom dumped). Large excess hydrostatic pressures develop in these fills [72], leading sometimes to stability failures. The concept of the steady state strength is ideally suited to deal with this problem. If the sand exhibits contractive behavior when subjected to shear stress, a shear failure can be precipitated as excess porewater pressures increase. Conversely, if the sand expands under shear stress the fill will be stable. Expansive sands will be to the left of the steady state line in figure 64, while contractive sands will be to the right of the line. The position of the in situ material with respect to the steady state line is defined by two parameters: the void ratio, and the in situ state of stress (confining pressures).

A single parameter has been proposed by Been et al.[73] to define this in situ state of the sand, the "state parameter". The state parameter, Ψ, was defined by the authors as the difference between the void ratio of the in situ sand and the void ratio the same sand, subjected to the same confining pressures, expressed in terms of the first stress invariant, $I_1 = [(\sigma_1 + \sigma_2 + \sigma_3)/3]$, would have to have in a steady state of deformation. This definition is illustrated in figure 65. Ψ would be positive if the in situ sand is contractive, and negative if it is expansive. Thus a sand with a negative state parameter would be inherently stable, and a sand with a positive parameter inherently unstable, regardless of its relative density. Therefore the state parameter describes the in situ state of the sand in a more fundamental way than the relative density.

In reference [73] the state parameter is correlated with several sand characteristics. An example is shown in figure 66. As previously noted, the utility of the concept of steady state strength goes beyond the liquefaction problem. In terms of measurement of soil properties, definition of the state parameter would require determination of in situ confining pressure. It is suggested that another parameter, namely the ratio of the in situ confining pressure to the confining pressure at the same void ratio at the steady state of deformation would be very useful for design purposes.

Work now in progress [72] includes study of the use of the CPT test in the definition of the state parameter. A proposal at the end of this report to develop an instrument which can combine the CPT measurement with a measurement of the in situ confining pressure could solve this problem.

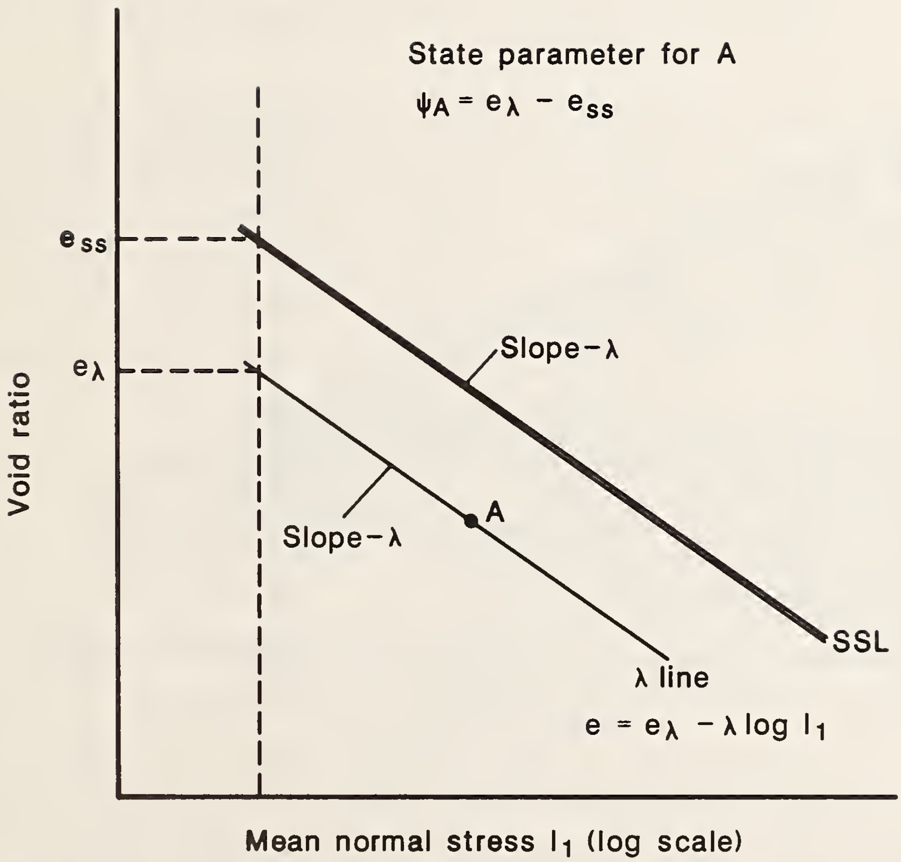


Figure 65. Definition of the State Parameter (from Ref.[73])

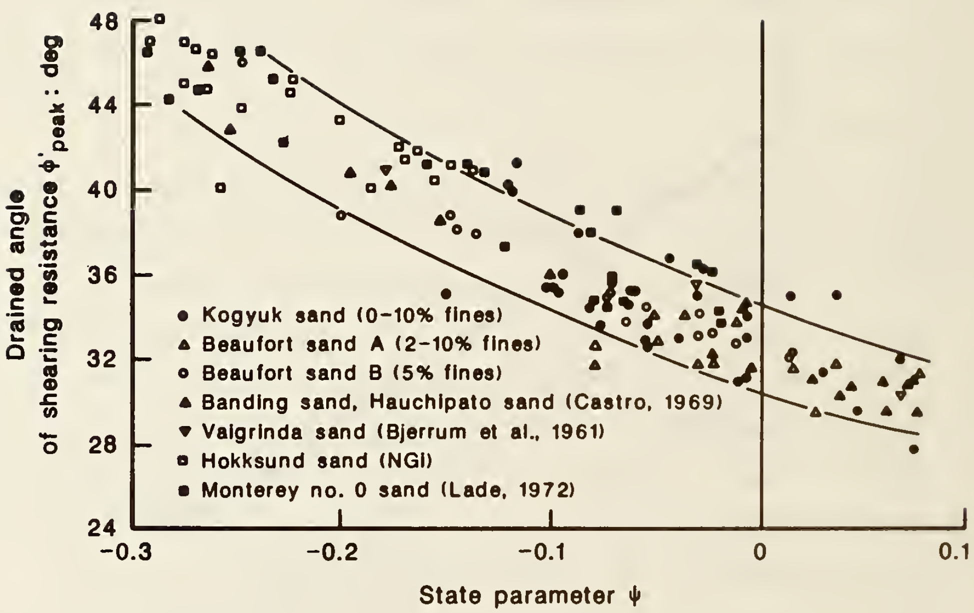


Figure 66. Correlation Between the State Parameter and the Angle of Shearing Resistance (from Ref.[73]).

Determination of stiffness parameters: Soil stiffness is generally affected by several parameters. These include: density; confining pressure; magnitude of strain; rate of strain; and OCR, and there is a considerable body of knowledge by which various in situ measurements can be correlated with shear modulus. In the field, geophysical measurements can be used to determine G_{max} as previously dicussed. In the laboratory, triaxial, as well as compressive and torsional resonant column tests can be used, provided reasonably undisturbed samples are available. Stiffness data are important for mathematical modeling of static and dynamic response to loads.

Use of Index Properties: In many instances, when there is already substantial information on the soil deposit, index properties can be effectively used to predict strength and stiffness parameters. For instance values of s_u/p', where s_u is the undrained shear strength and p' is the effective overburden pressure have been correlated with the plasticity index of normally consolidated clays, and and the s_u/p' value for normally consolidated clays, in turn, can be adjusted for the overconsolidation ratio. At least this latter adjustment may have to be used even when the shear strength is determined from laboratory tests because of the erratic variation of overconsolidation ratios in shallow Beaufort offshore deposits (the test values may have to reduced to correspond to some reasonable OCR value). The use of index properties may become very effective when calibrated against a body of available data, and may be used to detect test results which substantially deviate from the mean values of available data.

4.5.3 Augmentation of Soil Resistance

As noted in Section 4.4.2, there are practical limitations to the load resistance of gravity mat foundations. The load capacity can sometimes be increased by modifications of the foundation or the supporting soil. It should however be realized, that the plan dimension of these foundations is very large, and that strengthening of the soil to a moderate depth may sometimes merely change the location of the slip plane without a substantial change in the load resistance. There are three types of methods by which the load resistance can be augmented:

(1) Compaction or consolidation to increse density.

(2) Grouting, admixture or freezing, to change the material characteristics.

(3) Addition of load resisting elements such as spud piles and skirts.

Increase of Soil Density: Cohesive soils are can generally be densified by surcharging and consolidation. The time element required for the consolidation settlement may be a problem if critical loads could occur soon after installation of the structure. Consolidation settlements can be increased by vertical sand or wick drains. Granular soils generally can not be substantially compacted by surcharging. Mechanical means of compacting sands and gravels were discussed in Section 4.4.8.

Compaction Grouting, admixtures and freezing: These methods will generally increase the soil strength by at least an order of magnitude. They tend to be expensive, but they may be effective when a relatively shallow layer of weak soil is underlain by much stronger soil. The feasibility and type of grouting which can be effectively used depends on the soil type and permeability. A newly developed mixing procedure [74] has also been used successfully for seabed strengthening and is being tried in the Arctic. In that latter procedure, cement is mixed in place with in situ soils. An interesting possibility would be the use of a grouted soil layar instead of a pre fabricated mat. This possibility has apparently not yet been explored.

Soil freezing is also being explored. This method could rely on a passive system, using heat exchange with the environment, an active system which would require energy input, or a combination of both. On a much smaller scale, freezing has been successfully used in the foundations of the Trans Alaska Pipeline.

4.6 Reliability of Predictions

Any analytical model can only be as accurate as the parameters used to characterize the soil. There is presently no organized body of information to assess the accuracy of our in situ and laboratory tests. Thus this estimate must be left to the judgment of the engineer. In general, the confidence in our estimates can be much improved when the strength and stiffness parameters are determined by one method and then verified by other methods. The topic of gravity foundation reliability has been addressed in a recent presentation [75].

The use of partial safety factors, like those in European practice, may be reasonable, since it permits the engineer to assign a numerical value to the partial safety factor for resistance which corresponds to his confidence in the results of in situ and laboratory measurements. In conjunction with this approach, a target value could be assigned to the safety index (the argument in equation 4.1). A value of 2.3 would not be unreasonable, when used in conjunction with the 100 and 25 year mean recurrence intervals for design loads, presently used for production, and exploration facilities, respectively.

5. SUMMARY

Subsurface Conditions:

Subsurface conditions in the Alaskan Beaufort and Eastern Chukchi areas pose complex and unique problems. These include the presence of relict permafrost, shallow gas deposits, ice gouges, and erratic patterns of shear strength and overconsolidation. There are some areas of instability and some seismically active areas. Sand and gravel resources are relatively scarce.

Surface deposits are predominantly overconsolidated silty clays and clayey silts of Holocene origin. These are underlain by Pleistocene deposits of sands, silts and clays which are frequently fully or partially ice bonded. Pockets of soft soils, and irregular overconsolidation patterns are common.

Loads:

Ice loads develop large horizontal thrusts. Design wind speeds with a 50 year mean recurrence interval are between 90 and 100 miles per hour. Maximum wave heights are about 10m and storm surges rise to about 3m above mean sea level. Currents and strudel scours can cause erosion problems. Seismicity is limited to a small area in the vicinity of Barter Island.

Ice loads are are much greater than any other environmental loads and controll the design of foundations. However, potential effects of other environmental loads should be taken into account. Rare and exceptional ice loads (e.g. due to ice islands) are of a magnitude that cannot be resisted by feasible gravity structures on mat foundations. These loads must be avoided, or their effects must be mitigated by means other than load resistance. Global ice lods used for foundation design increase with water depth and depend on the exposed width of the structure and the slope of its sides.

Foundation Types:

Foundation types considered in this report are foundations for sand and gravel islands, foundations for caisson retained islands which are rigid circumferential structures with a sand core, resting on sand or gravel berms, and rigid mat foundations for gravity structures which are usually placed on the ocean floor with relatively little preparation, but could be placed on prepared berms. The type of structures and foundation used is to a large extent dictated by contruction feasibility during the short open water season of the U.S. Beaufort Sea (about 30 days).

Design Limit States:

When failure probabilities, associated with design limit states are considered, account must be taken of the fact that rare and exceptional ice loads, which can not be realistically resisted by gravity structures, have probabilities of occurrence which may not be acceptable for property damage or extensive environmental damage, and would probably also not be acceptable for the total loss of the installation. Thus strategies must be considered to avoid such ice loads or mitigate their consequences.

Design limit states are identified for the various types of foundations. Whether these limit states (which are failure modes) should be defined as ultimate or seviceability limit states should not be determined in accordance with rigid design rules, since this determination depends on the consequences of the failure mode associated with the limit state. Many offshore structures have the capability to mitigate the consequences of certain types of limit states, such as excessive displacements or local instability.

Load resistance:

While analytical models to determine foundation response to loads are generally available, the accuracy of the prediction is only as good as our ability to determine the relevant soil and loading parameters. Accurate characterization of soils is difficult because of the many parameters which influence the strength and stiffness of soils in a given state of stress. These parameters are discussed in this report. In particular, it is difficult to determine in situ geostatic stresses and to obtain undisturbed samples of granular soils for laboratory testing.

Prediction of cummulative displacements and degradation in soil strength and stiffness resulting from seasonal cyclic ice loads are problems that warrant significant attention.

Various methods of soil strengthening, including consolidation, mechanical compaction, grouting, admixtures and freezing are briefly discussed. It is noted thet compacion to a shallow depth may not significantly improve load capacity because of the large dimension of the foundation. The possibility of replacing rigid mat foundation by a layer of soil which is strengthened by grouting or mixing with cement is of interest.

6. NEEDED STUDIES AND RESEARCH

(1). Our capability of measuring soil properties in situ should be improved, with particular emphasis on in situ confining pressures, strength and stiffness.

(2). It is recommended to conduct combined analytical and experimental studies to improve our ability to predict horizontal displacements under sustained ice loads and cycles of seasonal ice loads.

(3). Generic information should be generated on the density and strength characteristics of hydraulically placed granular material used in berms, as a function of placement method and grain size characteristics.

(4). Density and strength characteristics of sand underfill placed after seating of the structure should be studied.

(3). More generic information should be generated on the geotechnical properties and characteristics of Beaufort shelf deposits. This information should include cyclic and steady state strength, effect of cyclic loads on static strength, effects of overconsolidation, and correlations between in situ and laboratory data.

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Design limit states for offshore gravity structures in the Alaskan Beaufort and Eastern Chukchi continental shelves are discussed. The report contains a description of geological conditions, design loads, and type of structures used. Three foundation types are considered: foundations for artificial islands; foundations for caisson retained island with sand corfs; and rigid foundations for various types of gravity structures which are positioned on the ocean floor with a minimum of preparation. Design limit states for these foundations are identified and the required reliability against the occurrence of these limit states is discussed. Our ability to determine foundation resistance is assessed.

12. KEY WORDS *(Six to twelve entries; alphabetical order; capitalize only proper names; and separate key words by semicolons)*

Artificial islands; geotechnical engineering; ice forces; mat foundations; ocean engineering; offshore platforms; oil production; sand and gravel berms; soil exploration; soil testing.

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