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Strength of Offshore Gravel Islands To Resist Ice Loads

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ABSTRACT

One of the prime concerns in designing offshore gravel islands in ice-covered waters such as in the U.S. and Canadian Beaufort Seas is the ability of the island to resist the imposed design ice loads. In presenting this paper, the goal is to describe a methodology to aid in the preliminary assessment of the overall strength of a gravel island to resist a design ice load for a set of prescribed design and environmental parameters. Conversely, the described methodology can be used to aid in developing a preliminary island design in terms of its size, height and geometry to ensure the island has sufficient strength to resist the design ice loads.

More specifically, the purpose of this paper is twofold: (1) to describe a methodology for establishing the strength of offshore gravel islands to resist imposed ice design loads; and (2) to use this methodology to illustrate how the performance of gravel islands to resist the imposed ice loads varies with the major design and environmental parameters.

In summary the results of the analysis presented in this paper indicate that, from the standpoint of resisting ice loads, gravel islands will continue to be technically feasible offshore drilling platforms for both exploration and production in the U.S. and Canadian Beaufort Seas as the oil industry moves into deeper and more harsh areas.

INTRODUCTION

With the recent offshore lease sales in the Diaper Field and the future scheduled lease sales, interest in investigating various drilling platform concepts for exploration and production in the U.S. and Canadian Beaufort Sea areas has continued. Proposed concepts have ranged from gravel islands, caisson retained islands, and mobile structures to moored floating platforms. To date gravel islands have been used as the primary man-made exploration drilling platform in the U.S. and Canadian Beaufort Seas. As interest has expanded to areas farther from shore, deeper water and less protected areas, the question arises as to whether or not gravel islands will continue to be viable and will they have sufficient strength to resist the imposed design ice loads.

In designing gravel islands for the Beaufort Sea it is important to note that other major concerns must be considered in addition to the island strength to resist ice loads. These other concerns include: ice ride-up/pile-up potential; wave run-up and overtopping; seabed scour due to ice, waves and currents around the structure; and slope protection to prevent island erosion by waves and ice. Of these, however, ice loads tends to be one of the major concerns and is the focus of this paper.

GRAVEL ISLAND STRENGTH DETERMINATION

Overview

In determining the overall gravel island strength to resist the design ice load, three failure modes were considered: failure at the freeze front, failure at the ice loading plane, and failure at the gravel fill/silt interface which may be at the seafloor or lower if surface silts are dredged before construction. For the range of soil material properties investigated (see Table 1) the governing failure mode was primarily a function of the depth of freeze as depicted in Figure 1. For freeze depths from the island surface less than the island height above the ice loading plane, failure was determined to occur at the ice loading plane through the unfrozen gravel fill. When the freeze depth extended below the ice loading plane, but not to the seabed silt layer, failure was determined to occur at the frozen-unfrozen interface in the gravel fill. Similarly when the freeze front extended through the gravel fill into the seabed silt layer, failure was determined to occur at the frozen-unfrozen interface in the silt layer.

The above failure modes were determined with the assumption that a frozen soil is considerably stronger than the same soil in its unfrozen state. While this has been proven to be the case for soils containing fresh water, significantly less information and data is available on the frozen strength of soils containing salt water. If, as postulated, frozen gravel fill and silts containing salt water are also considerably

References and illustrations at end of paper.

stronger than the gravel fill and silts in the unfrozen state, then the failure modes will occur as described above. If, on the other hand the frozen soil strengths are not considerably stronger, consideration must be given to investigating failure through the frozen gravel fill or silt layers.

The shearing resistance of an unfrozen soil closely follows Coulomb's equation [1]:

$$s = c + \sigma \tan\phi \quad [1]$$

where

s = Shear strength of the soil,

c = Cohesion of the soil,

σ = Normal stress,

ϕ = Angle of internal friction of the soil.

On an effective stress basis, Coulomb's equation can be expressed by [2]:

$$s_{eff} = c' + \overline{\sigma} \tan\phi' \quad [2]$$

where

s_{eff} = Effective shear strength of the soil,

$\overline{\sigma}$ = Effective normal stress,

c' = Cohesion on an effective stress basis,

ϕ' = Angle of internal friction on an effective stress basis.

If the voids or a portion of the voids in the soil are filled with a fluid under a pore water pressure u_w, then one portion of the normal stress (σ) is carried by the soil and the other portion is carried by the fluid. Expressed mathematically, the effective normal stress $\overline{\sigma}$ is equal to the difference between σ and u_w:

$$\overline{\sigma} = \sigma - u_w \quad [3]$$

Combining Equations [2] and [3] yields:

$$s_{eff} = c' + (\sigma - u_w) \tan\phi' \quad [4]$$

Using Equation [4], the total sliding resistance (S_{total}) of a gravel structure is equal to the product of the shearing strength (s_{eff}) along the failure plane and the area of the failure plane (A_{fail}).

$$S_{total} = s_{eff} \cdot A_{fail} \quad [5]$$

An overview of the procedure used to determine the overall strength of a gravel island to resist ice loads is depicted in Figure 2.

From Equations [4] and [5], it is seen that the sliding resistance is dependent upon the following five independent variables:

- Area of the shear plane (A_{fail}),
- Cohesion of the soil (c'),
- Overburden pressure (σ),
- Pore water pressure (u_w),
- Internal friction angle of the soil (ϕ').

For a cohesionless soil (c'=0), Equation [4] indicates that if the pore water pressure were equal to the overburden pressure, the soil would possess no shearing strength and hence no sliding resistance. On the other hand, if the soil has a zero internal angle of friction (ϕ'=0), neither the pore water pressure nor the overburden pressure are important in computing sliding resistance. Thus in order to calculate sliding resistance, it is important to have an accurate knowledge of both c' and ϕ' for the gravel island fill and the soils underlying the gravel structure.

If the soil in question is cohesionless and if the shear stength of frozen soil is considerably greater than that of unfrozen soil, then the shear failure plane will occur along the failure planes noted in Figure 1. Therefore in order to compute the sliding resistance, knowledge of the depth of the freeze front is necessary to evaluate the overburden pressure. In calculating the depth of freeze, information must be known about the soil thermal properties, and pore water content.

In the following sections, the problem is analyzed in steps. First the analysis of pore water pressure, which was aimed at determining the importance of this variable in calculating sliding resistance, is described. Next the thermal analysis aimed at determining the total overburden pressure at any given time during the life of the project is described. Thirdly, the soil strength properties (c' and ϕ') used in the sliding resistance computations are discussed.

Pore Water Pressure

The layer of silt below the proposed construction site is assumed to be a compressible substance in hydraulic equilibrium, and that drainage of water from this silt layer obeys Darcy's Law. When the first load of gravel is placed on this layer, water does not immediately drain from the pores and, as a result, a sudden excess hydrostatic pressure equal in magnitude to the surcharge exists in the soil. This excess hydrostatic pressure initially negates the added shear strength in a frictional soil due to the surcharge. With time, the excess pore water pressure decreases as the silts are compressed and water drains out. The time required for the excess water pressure to disappear is governed by the following equation [1]:

$$T_V = \frac{C_V}{H^2} t \quad [6]$$

where

T_V = Time factor (non-dimensional),

C_V = Coefficient of consolidation (ft^2/day),

t = Time in days,

H = Thickness of silt layer for one sided drainage or 1/2 silt layer thickness for two sided drainage (ft).

When T_V equals 1, approximately 90% of the drainage has occurred [1]. Therefore, the time required for 90% drainage is:

$$T_{90} = \frac{H^2}{C_V} \quad [7]$$

Based on data from DeJong [3], silts in the Canadian Beaufort have C_V values in the range of 3.5 to 20 ft^2/ day. Harding-Lawson [4] suggest C_V values of about 1.4 ft^2/day as being typical of the silts in the Prudhoe Bay area. Assuming the thickness of the silt layer of 5 to 20 ft, typical values of t_{90} given from Equation [7] are presented in Table 2. In summary, using these typical values of C_V and depth of the silt layer, the general conclusion obtained is that settlement will occur rapidly and therefore shear strengths of the soil will reach their maximum values under the surcharge very quickly. Because of this rapid settlement, long term excess hydrostatic pressures need not be considered in the sliding resistance calculations, provided the silt depth and particular value of C_V are within the range analyzed. If the silt depth and the value of C_V are not within the range analyzed then consideration of the pore water pressure must be included in determining the strength of the island to resist the ice loads.

Assumed Soil and Water Properties

Table 1 lists the range of soil and water properties used in this paper. Thermal conductivities of gravel were given by Harding-Lawson [4]. Thermal conductivities of silt and fine gravel were taken from Kersten's [5] data. Thermal properties of sea ice and water were obtained from Doronin and Kheisin [6] and Ono [7]. Using Andersland and Anderson's approach [8], specific heat capacities for silts, gravel fill, and fine gravel were computed by:

$$C_{u,f} = \gamma_d (C_s + f \frac{w}{100} C_{w_{u,f}}) \quad \text{. . . . . [8]}$$

where

$C_{u,f}$ = Volumetric heat capacity in unfrozen, frozen state,

γ_d = Dry density of the soil,

C_s = Unit weight heat capacity of the soil = 0.18 BTU/lb-°F,

f = 1.0 for unfrozen soil and 0.5 for frozen soil,

w = Water content of soil in percent,

$Cw_{u,f}$ = Unit weight heat capacity of water in unfrozen, frozen state.

Dry densities and water content were obtained from Harding-Lawson [4].

Freeze Front Determination

One and two dimensional, finite element models developed by Bafus [9,10] were used to conduct this analysis. These models solve the one and two dimensional heat transfer equation with arbitrary boundary conditions, variable thermal properties, and phase change. The model was specifically developed to handle the problem of determining freeze front propagation in gravel islands. The model has been validated by comparing its results with known analytical solutions to the problems involving freeze front propagation (Newmann equation) and known analytical solutions to two dimensional heat transfer problems not involving phase change. These comparisons showed that the model gives quite accurate estimates.

The air temperature (shown in Figure 3) used in the model was based on the historical temperatures for North Slope, Alaska. To investigate the effect of snow cover, several modified temperature distributions shown in Figure 3 were also considered. These modified temperature distributions were based on the following:

$$T_n \begin{cases} T_F - n(T_F - T_1) & \text{(October - May)} \\ T_1 & \text{(June - September)} \end{cases} \quad [9]$$

where

T_n = Average monthly temperature for a given value of n,

T_F = Freezing temperature (28.8°F),

T_1 = Average monthly temperature for North Slope, Alaska (n=1),

n = Temperature factor ($\leq$ 1.0).

Based on the data presented by Bafus [9,10], a value of n equal to 0.6 corresponds to a snow cover of approximately 1 to 1.5 ft. Using the model a typical time history of the freeze front propagation into a 600 ft diameter island in 30 ft of water with a height of 20 ft above the water surface is shown in Figure 4.

Soil Strength Properties

As pointed out earlier, the unfrozen strength of the silts and gravels are critical in determining the sliding resistance of the gravel island. Usually soil tests are made under conditions representative of load conditions in the field. Frequently this load condition is difficult to determine beforehand and hence the engineer's judgement must be used to determine the most appropriate type of soil strength tests.

As noted above the silt layers will quickly compress and drain once the surcharge gravel is in place. Triaxial, consolidated, undrained tests at a minimum should be used to determine strengths. During such tests, the pore water should be allowed to drain from the soil specimen during the application of the initial consolidation or chamber pressure. After equilibrium conditions are reached, the drain valve is closed and the specimen is located at a constant strain rate until failure. Strain rates on the order of 1% per min (1.67 x 10^{-4} sec^{-1}) are commonly used. Because strain rates associated with the design ice loads are two to three orders of magnitude less than the above strain rate and the silt appears to have good drainage characteristics, it would appear that the silt should be tested with the drain valve left open and at much lower strain rates. For the purposes of this paper the following soil strength properties were assumed for the unfrozen state where failure is assumed to occur:

	COHESION (c')	FRICTION ANGLE (φ')
Gravel Fill		
Above water surface:	0	30-40
Below water surface:	0	30-40
Seabed Silt Layer :	0	30-40
Gravel Silt Layer :	0	30-40

ASSESSMENT OF ICE DESIGN LOADS

In establishing the ice design loads there are two approaches one can choose: ICE FAILURE APPROACH and the LIMITED DRIVING FORCE APPROACH. Using the ICE FAILURE APPROACH, one assumes a sufficient driving force exists to move the ice and the gravel structures has sufficient resistance capability. In this approach the ice design load is defined as the force the gravel island structure must exert on the ice in order to fail the design ice feature. The LIMITED DRIVING FORCE APPROACH [11] removes the assumption that a sufficient driving force exists to move the ice and assesses the maximum possible driving force which can be generated on an ice field of a given fetch by the design environmental conditions of winds and currents. This maximum driving force is then compared to the ice failure load. The ice design load is then defined as the minimum of the maximum driving force and the ice failure load. As one can imagine, the establishment of a design ice feature and the associated design ice load which the gravel island must withstand is very site specific. For this reason, rather than select a single design ice load upon which to evaluate the performance of the gravel island, the following three ice design load levels were considered:

DESIGN ICE LOAD LEVELS CONSIDERED

300 kips/ft
600 kips/ft
1000 kips/ft

Using these levels, the total design ice load a particular gravel island must withstand is determined by multiplying the ice load level by the waterline diameter of the island.

RESULTS

Using the methodology outlined above, the following parametric variations, listed in Table 3, were conducted in order to examine how the strength of a gravel island varies with the major island design and environmental parameters:

Island Geometry	Figure Number
∘ Island Size (Diameter)	5
∘ Island Height	6
∘ Island Side Slope	7
Gravel Fill	
∘ Internal Angle of Friction	8
∘ Initial Temperature	9
∘ Water Content	10
∘ Freezing Temperature	11
∘ Dry Density	12
Environmental Parameters	
∘ Water Depth	13
∘ Ground Surface Temperature	14

Based on the results presented in these figures the following observations were made:

∘ Island Size: As one would expect, increasing the top surface diameter of the island is the major design parameter to increasing the capability of the island to resist ice loads. After the first year, an 800 ft diameter island can withstand 3 times the ice load that a 400 ft diameter island can withstand. For the first year and into the second, the freeze front has not penetrated below the ice loading plane. Beyond the second year, the freeze front extends below the ice loading plane, but by 5 years has not penetrated the silt layer.

∘ Island Height: Island height plays a significant role in the island strength to resist ice loads for the first two or three years; however beyond that period, the island height plays a much less significant role.

∘ Island Side Slope: In a similar manner to island diameter, island side slope can significantly increase the island strength by increasing the failure surface area over which the shear strength can act.

∘ Internal Angle of Friction: As noted in Equation [4] for a cohesionless soil, the internal angle of friction influences the island strength proportionally to the tangent of the angle. Therefore gravel fill with an internal angle of friction of 40° provides an increased island strength of approximately 45% over that provided by gravel fill with an internal angle of friction of 30°.

∘ Gravel Fill Temperature: The initial gravel fill temperature has an influence on the island strength for the first year or two due to its effect on retarding the propagation of the freeze front during the first year. Beyond this period, the influence continues to diminish with a decreasing time lag in the associated freeze depth penetration.

∘ Water Content: The percent water content has a significant influence in the gravel island strength with the lower water content gravel generating a higher island strength. The increase is due to the increased freeze front penetration.

∘ Freezing Temperature: One of the major unknowns in determining the freeze front is the potential for a depressed freezing point temperature at the frozen/unfrozen interface due to increased salt concentration. The concern is that as the freezing point temperature is depressed at the freeze front, the penetration of the freeze front could be significantly retarded from the depths used in the above gravel island strength calculations. In order to assess this impact, the freezing point temperature of the gravel was varied from the base value of 28.8°F down to 20°F. It should be noted that in making this variation, it was assumed the freezing point temperature was uniform over the entire depth rather than making an assumption as to the percent salt concentration increase with depth and time. With this conservation assumption, it was found that the freezing point temperature has a significant influence on the freeze depth and the corresponding island strengths. For a 20°F freezing point temperature, the freeze front did not penetrate below the ice loading plane until into the fourth year and after five years the strength was approximately 80% of the strength calculated assuming a 28.8°F freezing point temperature.

∘ Gravel Fill Dry Density: Assuming cohesionless soils, the island strength varies porportionally with the gravel fill dry density. Thus for the range of dry densities considered the island strength varies from approximately 92-104% of the base dry density strength value.

∘ Water Depth: For water sufficiently deep so that the freeze front does not penetrate into the silt layer, no variation in island strength occurs with water depth. If the water is shallow enough so that the freeze front can penetrate into the silt layer, the strength is reduced due to the lower dry density and higher water content of the silt layer. This trend could be reversed if the silt layer is relatively thin and the freeze front could penetrate into a gravel layer below the silts.

∘ Ground Surface Temperature: To assess the influence of an insulating snow cover on the island strength, the freezing temperature factor (n) in Equation [9] was varied from 1.0 (base case) to 0.4 to effectively increase the gravel island surface temperature. For n equal to 0.6 which corresponds to approximately 1 to 1.5 ft of snow, the freeze front was retarded from penetrating below the ice loading plane to the third year. However, after five years, island strength was reduced only by between 10-15% from the base case (no snow).

CONCLUSIONS

Based on the results presented in the previous sections the following conclusions were drawn:

- From the standpoint of resisting ice loads, gravel islands will continue to be a viable offshore drilling platform for both exploration and protection in the U.S. and Canadian Beaufort Seas. As illustrated in Table 4, after the first year all island sizes considered (assuming the base case island design and material property parameters) had a factor of safety greater than 1.5 with the exception of the extreme design load level of 1000 kips/ft. For this extreme design load level only the 400 ft diameter had a factor of safety less than 1.0. After five years all have factor of safety values greater than 1.3. It is important to note that these values are based on the island design and material properties equal to base case values which includes the assumption of no cohesion. If cohesion were present, then the associated island strength would be correspondingly higher than the values reported in Table 4.

- It is important to note that the ability of the gravel structures to resist the ice design depends entirely upon the shearing strength of the gravel fill and the seabed material beneath this fill. As a result to accurately predict the strength a good assessment of the gravel fill and seabed materials and their properties is important. In this regard efforts should be undertaken during the design phase to quantify the materials and their properties in both the unfrozen and frozen state. Special attention should be given to conducting strength tests of the materials using salt water to show how the strength compares with that using fresh water.

- Similarly it also is important after construction to conduct periodic gravel and seabed monitoring tests as a means of ensuring proper island construction and validating the basic design assumptions for use in the design of the next structure.

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TABLE 1

RANGE OF SOIL PROPERTIES INVESTIGATED*

	Gravel Fill Above Water	Gravel Fill Below Water	Silt Layer	Gravel Below Silt Layer
Cohesion* (lbs/ft^2)	0	0	0	0
Internal Angle of Friction* (degrees)	30-40	30-40	30-40	30-40
Dry Density (lbs/ft^3)	120-135	115-130	100	125
Water Content (%)	5-10	8-15	20-35	8-15
Thermal Conductivity (BTU/(hr-ft-°F))				
Frozen	1.7-2.5	2.0-2.5	1.1-1.3	2.7-3.0
Unfrozen	1.3-3.0	1.5-3.0	0.9-1.1	1.4-2.7
Volumetric Heat Capacity ($BTU/(ft^3$-°F))				
Frozen	23.2-24.8	24.2-26.3	27.7-31.2	24.2-26.3
Unfrozen	26.0-30.2	28.6-34.1	37.6-46.6	28.6-34.1
Volumetric Latent Heat (BTU/ft^3)	710-1600	1100-2315	2375-4160	1190-2230

* Cohesion and internal angle of friction on an effective stress basis.

TABLE 2

DAYS TO CONSOLIDATE SILT LAYER

	COEFFICIENT OF CONSOLIDATION (ft^2/day)		
SILT LAYER DEPTH (ft)	1.4	3.5	20.
5	18.	7.	1.
10	71.	29.	5.
15	160.	64.	11.
20	286.	114.	20.

TABLE 3

PARAMETRIC VARIATION OF DESIGN PARAMETERS

	BASE CASE	VARIATION
ISLAND GEOMETRY		
o Top Surface Diameter (ft)	600	400 - 800
o Island Height Above Water Level (ft)	20	10 - 30
o Island Side Slope	1:3	1:2.5 - 1:5
GRAVEL FILL		
o Friction Angle (deg)	35	30 - 40
o Initial Temperature (°F)	29	29 - 40
o Water Content (%)*	7/11	5/8 - 10/15
o Dry Density (lbs/ft^3)*	130/125	120/115 - 135/130
o Thermal Properties - Frozen		
- Conductivity (BTU/(hr-ft-°F))*	2.5/2.5	1.7/2.0 - 2.5/2.5
- Volumetric Heat Capacity ($BTU/(ft^3$-°F))*	23.9/25.9	23.2/24.2 - 24.8/26.3
o Thermal Properties - Unfrozen		
- Conductivity (BTU/(hr-ft-°F))*	2.0/2.0	1.3/1.5 - 3.0/3.0
- Volumetric Heat Capacity ($BTU/(ft^3$-°F))*	27.8/31.0	26.0/28.6 - 30.2/34.1
o Freezing Temperature (°F)	28.8	28.8 - 20
ENVIRONMENTAL		
o Water Depth (ft)	30	20 - 60
o Temperature Factor (n)	1.0 (no snow cover)	1.0 - 0.4

* Above/below water surface.

TABLE 4

FACTOR OF SAFETY AFTER ONE YEAR AND AFTER FIVE YEARS*

ISLAND TOP SURFACE DIAMETER (ft)	ISLAND STRENGTH (kips/ft)		ICE DESIGN LOAD LEVEL					
			300 kips/ft		600 kips/ft		1000 kips/ft	
	1 yr	5 yr	1 yr	5 yr	1 yr	5 yr	1 yr	5 yr
400	880	1250	2.9	4.2	1.5	2.1	0.9	1.3
500	1050	1450	3.5	4.8	1.8	2.4	1.1	1.5
600	1220	1740	4.1	5.8	2.0	2.9	1.2	1.7
800	1600	2230	5.3	7.4	2.7	3.7	1.6	2.2

* All island design and material property parameters are equal to base case values.

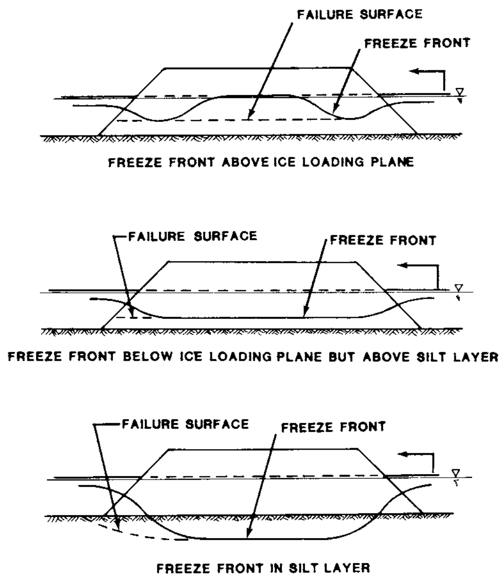
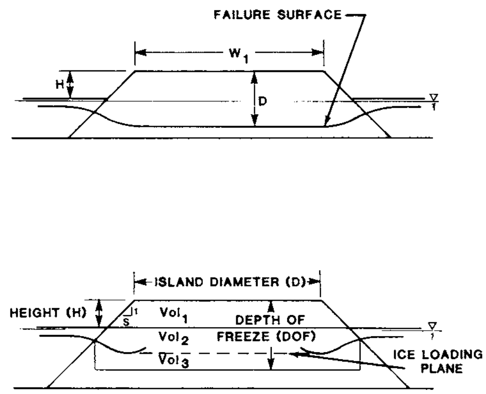


Fig. 1 GRAVEL ISLAND FAILURE MODE



IF DEPTH OF FREEZE IS ABOVE ICE LOADING PLANE:

$$S_{TOTAL} = \left[(1+w_1)\gamma_{d_1}\ Vol_1 + (1+w_2)(\gamma_{d_2} - \gamma_w)\ Vol_2\right]\mathrm{Tan}\,\phi'_2$$

IF DEPTH OF FREEZE IS BELOW ICE LOADING PLANE:

$$S_{TOTAL} = \left[(1+w_1)\gamma_{d_1}\ Vol_1 + (1+w_2)(\gamma_{d_2} - \gamma_w)(Vol_2 + Vol_3)\right]\mathrm{Tan}\,\phi'_3$$

WHERE :

w = WATER CONTENT
γ_d = DRY DENSITY OF SOIL MATERIAL
γ_w = DENSITY OF WATER

Fig. 2

MODEL TO DETERMINE GRAVEL ISLAND STRENGTH

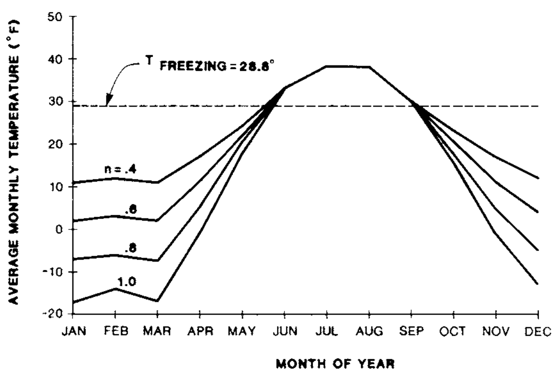


Fig. 3 YEARLY TEMPERATURE DISTRIBUTION

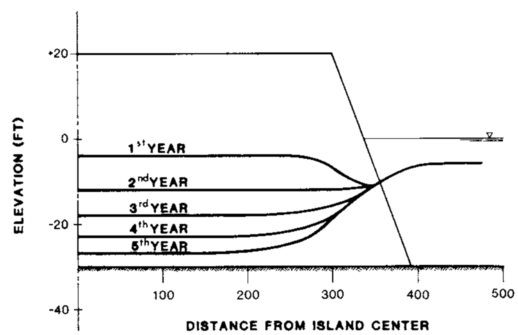


Fig. 4 FREEZE FRONT PROPAGATION WITH TIME

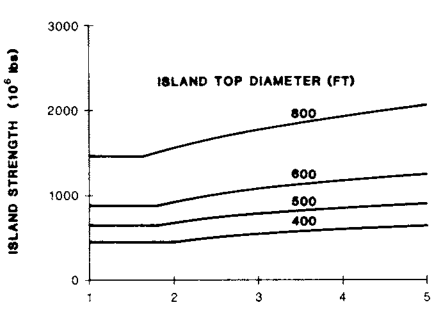


Fig. 5 ISLAND STRENGTH vs DIAMETER

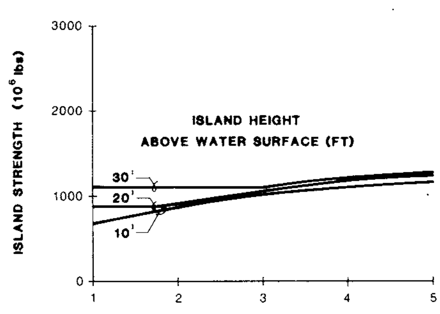


Fig. 6 ISLAND STRENGTH vs HEIGHT

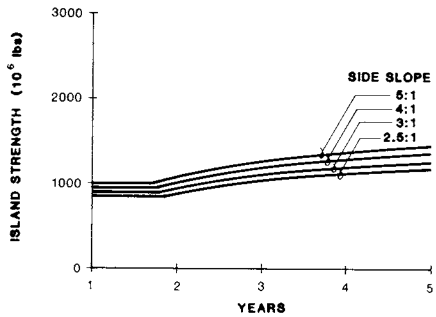


Fig. 7 ISLAND STRENGTH vs SIDE SLOPE

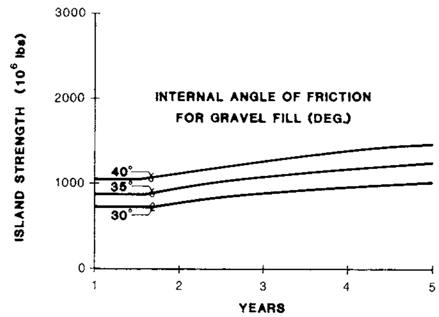


Fig. 8 ISLAND STRENGTH vs FRICTION ANGLE OF GRAVEL FILL

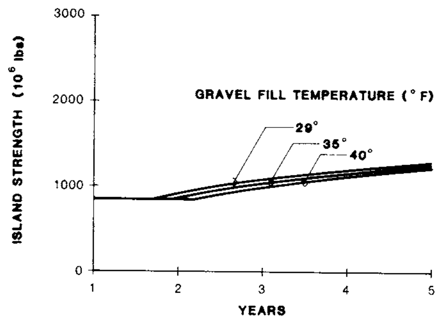


Fig. 9 ISLAND STRENGTH vs GRAVEL FILL TEMPERATURE

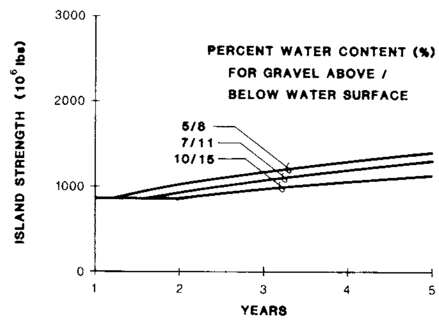


Fig. 10 ISLAND STRENGTH vs GRAVEL FILL WATER CONTENT

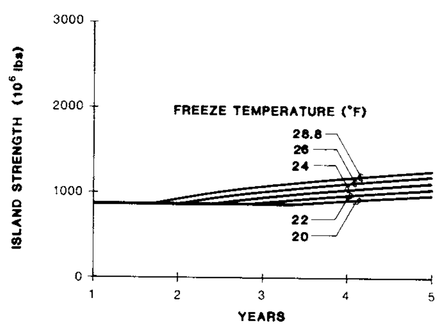


Fig. 11 ISLAND STRENGTH vs FREEZE TEMPERATURE

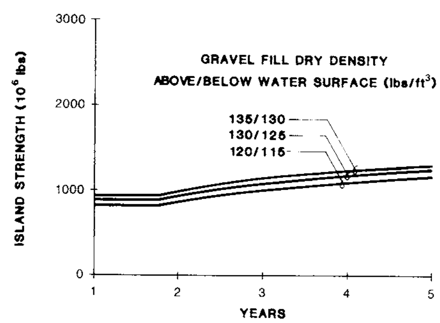


Fig. 12 ISLAND STRENGTH vs GRAVEL FILL DRY DENSITY

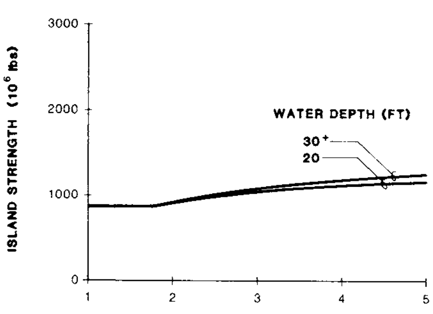


Fig. 13 ISLAND STRENGTH vs WATER DEPTH

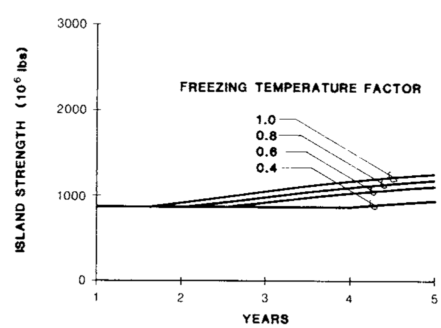


Fig. 14 ISLAND STRENGTH vs INCREASE GROUND SURFACE TEMPERATURE