

CHAPTER 94

PREDICTION OF WAVE-INDUCED SEAFLOOR MOVEMENTS

by

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ABSTRACT

Foundation design for offshore structures in areas where wave-induced bottom pressures cause submarine mud slides requires a knowledge of the potential depth of slide and of the magnitude and distribution of soil movements below the slide. Several methods have been developed to evaluate the stability of the seafloor due to wave-induced bottom pressures. These methods are reviewed and an improved procedure is presented. This procedure makes use of finite element analysis and combines in a rational manner oceanographic information on wave statistics with stress-strain behavior of soils under cyclic load conditions in order to evaluate the effects of a given storm history on the behavior of submarine sediments.

INTRODUCTION

Storm-generated surface waves can cause cyclic variations of pressure on the seafloor that trigger large scale soil movements manifested by massive soil failures to penetrations as deep as about 100 ft (30 m) below seafloors with slopes of one degree or less. Seafloor sediments most susceptible to these bottom pressure induced movements are the very soft to soft, underconsolidated clays found at the mouths of many major river systems, such as the Mississippi, Amazon, Niger, Ganges-Brahmaputra and Mekong, where active deltaic development results in a rate of sediment accumulation that exceeds the rate of pore pressure dissipation by consolidation. Fig. 1 illustrates that many of these deltaic regions are the site of major activity related to the installation and operation of offshore oil and gas production platforms and pipelines.

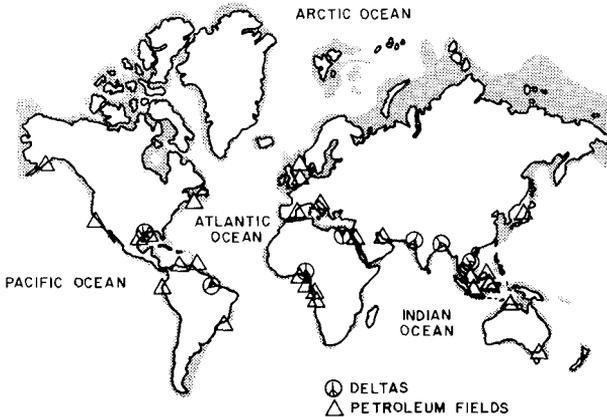


FIG. 1-MAJOR PETROLEUM ACTIVITY ON THE CONTINENTAL SHELVES

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When the seafloor is unstable the moving soil mass exerts lateral forces on offshore structures that can exceed the combined wind, wave and current force. Fig. 2 illustrates the soil loading mechanism. This paper outlines procedures currently used to assess the depth of the failure zone and the magnitude and distribution of accumulated soil movements produced by a given storm history. The application of these results to analyses of soil-structure interaction, which is required for reliable and economic structural design of foundations in these sediments, is beyond the scope of this paper.

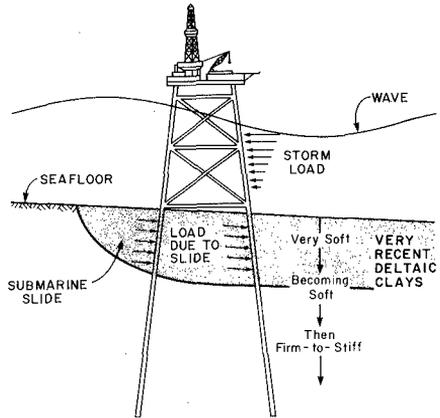


FIG. 2-LOADS DUE TO STORM WAVE AND SUBMARINE SLIDE

HISTORICAL DEVELOPMENT

Fig. 3 shows the failure of a 30-in. (76 cm) diameter flare pile that occurred during Hurricane Carla in 1961. This failure was an early demonstration of the potential depths of soil failure and the magnitude of lateral soil movement that may occur in very soft deltaic sediments. In August 1969, Hurricane Camille demonstrated convincingly the devastating impact of a submarine slide on offshore platforms. In about 300 ft (90 m) of water in South Pass Block 70 one platform was overturned and displaced about 100 ft downslope, and significant lateral soil movements occurred to at least 80 ft (24 m) below the seafloor(20). A second platform was displaced about 3 ft (1 m) downslope without overturning, and a third platform was destroyed in nearby South Pass 61.

These failures stimulated extensive and continuing programs to study submarine slides caused by storm-induced bottom pressure perturbations. The current state-of-the-art has evolved through many studies with contributions from and support by governmental agencies, academia, the petroleum industry and geotechnical consultants. To set the stage for the current state-of-the-art, the significant developments and results of earlier studies are highlighted.

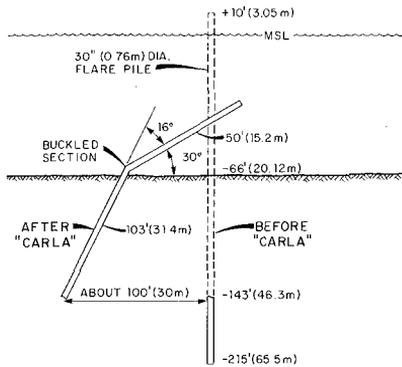


FIG. 3-FLARE PILE FAILURE CAUSED BY SEAFLOOR SLIDE (After McClelland and Cox, 1976)

Limit Equilibrium Analysis

One of the earliest published contributions that demonstrated the significance of storm-induced bottom pressures on submarine slope stability was presented by Henkel(7), who used a limit equilibrium method with an assumed circular failure surface. This concept, which is illustrated in Fig. 4, has been extended in practice to other shapes of potential failure surfaces and is used to estimate the depth of submarine slides, defined by the location of the deepest potential failure surface having a safety factor of one. Fig. 5, which shows the results of a limit equilibrium stability analysis for a location in the Mississippi Delta, illustrates that the potential failure surface with the minimum safety factor may underestimate the depth of the potential slide.

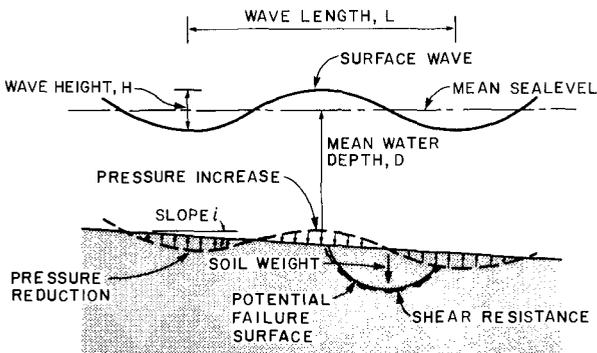


FIG. 4-LIMIT EQUILIBRIUM ANALYSIS FOR WAVE INDUCED SUBMARINE SLIDES

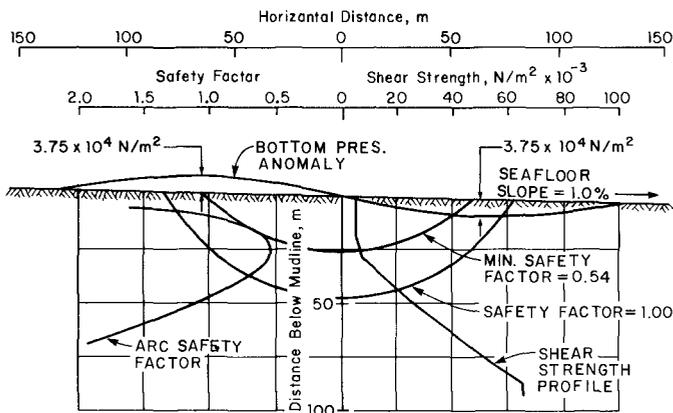


FIG. 5-RESULTS OF TYPICAL LIMIT EQUILIBRIUM ANALYSIS

The depth of slide zone is affected by the shear strength and unit weight profiles of the soil, and the amplitude and wave length of the bottom pressures. The interdependence between the influence of the amplitude and wave length of the bottom pressure anomaly on the stability of seafloor slopes for a very underconsolidated soil mass is illustrated in Fig. 6. Greater bottom pressures associated with longer wave lengths tend to cause deeper slides. If the design wave spectrum is known, analysis using combinations of wave lengths and amplitudes are made to determine the combination of wave data yielding the maximum depth of slide.

Limit equilibrium analyses, however, do impose certain approximations that challenge the accuracy of the depth of slide predicted with this procedure. Limit equilibrium analyses treat the soil as a rigid-plastic material and do not account for either soil softening and strength loss produced by a large number of successive waves or for dynamic effects. Nevertheless, limit equilibrium analyses are useful during preliminary studies to assess the likelihood of submarine slides for a given oceanographic and geotechnical environmental setting. Results of limit equilibrium analyses, however, do not provide estimates of the magnitude and distribution of soil movements.

Shear stresses computed using the theory of elasticity with a sinusoidal loading have also been used to evaluate stability. In this case the computed profile of maximum shear stress is compared to the shear strength profile.

Model Studies

To gain further insight into the stability and deformational patterns of seafloor soils subject to wave-induced bottom pressures and to evaluate the applicability of limit equilibrium approaches, model studies have been performed using wave tanks(4,7,15,19,22). Results of these model tests have demonstrated the validity of the failure mechanism and have shown that the vertical component of the movement decays more rapidly with depth than the horizontal component, but there is a net lateral translation downslope. Below the failure zone the net downslope movement decreases to about zero where the soil experiences nearly elastic behavior.

Results of these model studies demonstrated also that the yielding and movements of the seafloor soils can influence the development of bottom pressure amplitudes, which are usually computed from oceanographic data using linear wave theory that assumes a rigid seafloor. Doyle(4) found the bottom pressures on the soils in the wave tank were about 0.27 of those

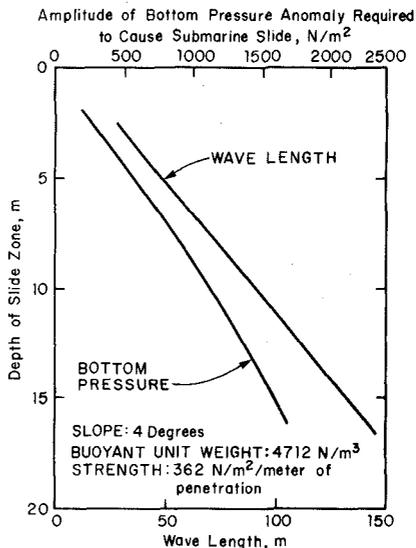


FIG. 6-DEPTH OF FAILURE ZONE FOR ANOMALIES OF VARYING AMPLITUDE AND LENGTH

computed for a rigid bottom. Gade(6) discusses the importance of soil yielding on the interaction between the water and soil. Due to the complex interaction between wave length, water depth and soil properties, the bottom pressure amplitude computed for a rigid bottom may not always exceed the value experienced on a yielding bottom. Since the magnitude and distribution of soil movements are strongly influenced by the amplitude and wave length of the bottom pressure, more attention is required, as will be discussed subsequently, on the interaction between the water and soil.

The qualitative results obtained from the wave-tank studies provided insight into a complex phenomenon and served to confirm general findings from analytic techniques in view of the limited field data available.

Finite Element Analysis: First Generation

At the time some of the wave-tank studies were being made the finite element analysis was being applied to the problem, and laboratory studies were initiated to develop stress-strain data for soft soils subjected to cyclic loads having periods coincident with storm loads. Wright and Dunham (25) first applied a finite element method with a nonlinear stress-strain response for the soils to evaluate seafloor response to wave-induced pressures. Since this model does not include the effects produced by gravity loads, the lateral boundaries are located one-quarter wave length apart for reasons of symmetry and antisymmetry, as shown in Fig. 7. In this case, displacements are allowed parallel to but not normal to the boundary under the crest, and below the null point displacements are allowed normal to but not parallel to the boundary. The lower boundary of the mesh is located at a penetration where soil movements are negligible so that restricting soil movements at this penetration in the model provides a reasonable approximation to in situ conditions. The distribution and size of the elements are selected to be compatible with soil property variations and the wave length and to provide reasonable numerical accuracy.

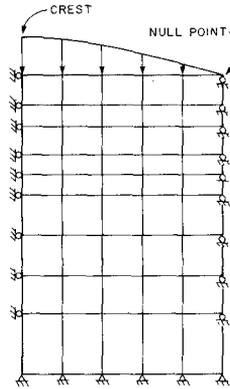


FIG. 7-QUARTER WAVE LENGTH FINITE ELEMENT MESH

The nonlinear and inelastic behavior typified by the undrained behavior of cohesive soils were incorporated into the finite element model using an interactive-incremental procedure(5,25) and a hyperbolic approximation(8) of the soil stress-strain response for loading conditions:

$$\sigma_1 - \sigma_3 = \frac{\epsilon}{\frac{1}{E_i} + \frac{R_f \epsilon}{2 S_u}} \quad , \quad \epsilon \leq \epsilon_f$$

$$\sigma_1 - \sigma_3 = 2 S_u \quad , \quad \epsilon \geq \epsilon_f \quad \dots \dots \dots (1)$$

where $\sigma_1 - \sigma_3$ = deviator stress; ϵ = axial strain; E_i = "elastic" tangent modulus at $\epsilon = 0$; S_u = undrained shear strength; and R_f = failure ratio, influence of the failure strain ϵ_f . The comparison in Fig. 8 between

Eq. 1 and laboratory measured stress-strain response of a Gulf of Mexico clay shows remarkable agreement. For unloading conditions, a decrease in maximum shear stress, the soil modulus value, E_{ur} , is approximately constant with a value that generally equals or exceeds E_i . The parameters associated with Eq. 1 were based on stress-strain data obtained from isotropically consolidated data.

Even though in situ gravity stresses were unaccounted for in these early finite element analyses, it was recognized that with vertical displacement of the seafloor mass under the action of a bottom pressure anomaly the gravity forces would tend to retard the tendency for instability, especially in very soft sediments where larger vertical displacements may occur. The counter pressure due to the buoyant weight of the elevated soil is illustrated in Fig. 9. Wright and Dunham(25) describe an approximate procedure for adjusting displacements to account for the counter buoyant effect. Arnold(1) and Bea and Arnold(2) used the finite element method to evaluate soil movements at a site in South Pass Block 70. Fig. 10 shows some results of their analyses to give perspective to the magnitude and distribution of computed movements due to wave-induced bottom pressures.

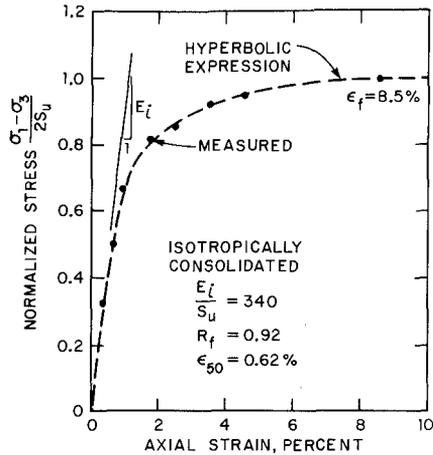


FIG. 8—COMPARISON BETWEEN MEASURED AND THEORETICAL STRESS-STRAIN RESPONSE

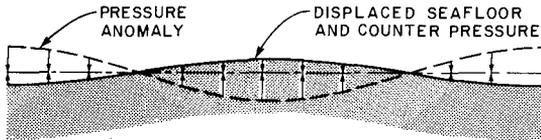


FIG. 9—COUNTER BUOYANT EFFECT DUE TO LARGE SEAFLOOR MOVEMENTS

Finite Element Analysis: Second Generation

Although results from the first generation finite element studies provided reasonable assessments of the distribution of stresses and displacements in a soil mass subjected to bottom pressure anomalies, the previously reported finite element analyses did not accurately account for either alterations in the soil properties due to the storm history or for gravity

stresses on the predicted soil movements. These factors were addressed, but only accounted for in a very approximate way. In addition, the interaction between the soil and water on the development of bottom pressures was neglected. Recently, Wright(24) described a procedure, based on a finite element analysis that includes initial gravity and geologic stresses, for predicting stress distribution and soil movements.

In situ shear stresses due to gravity and geologic stresses influence the seafloor response to wave-induced bottom pressures in three ways. The magnitude of soil stiffness is influenced by the shear stress in the soil due to its non-linear stress-strain behavior, and the increment of additional shear stress that the soil can sustain due to wave-induced loading may be reduced as the initial in situ shear stresses increase. Furthermore, when the gravity and geologic stresses are included, the stiffness moduli in the slope are usually not symmetrical about any parallel lines oblique to the slope. This lack of symmetry exists also when the wave-induced stresses are superposed. For an infinite train of uniform waves and negligible changes in soil properties with each wave cycle, the displacement patterns on lateral boundaries oriented perpendicular to the slope and separated by one wave length should be identical. Therefore, results of finite element analyses are based on equal displacements at corresponding penetrations on the two boundaries. An example finite element mesh for one wave length is shown in Fig. 11 together with the boundary conditions normally used.

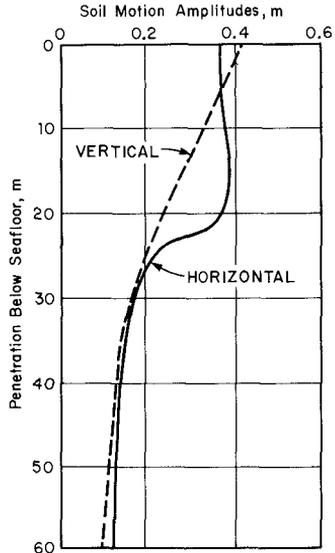
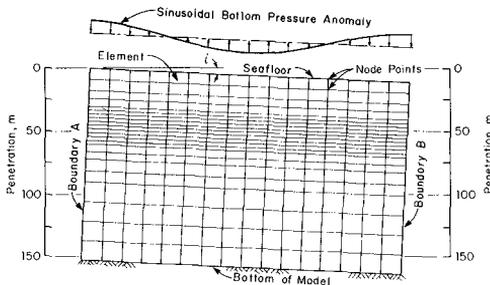


FIG. 10-TYPICAL MAXIMUM SOIL MOVEMENT INDUCED BY A SINGLE WAVE (After Bea and Arnold, 1973)



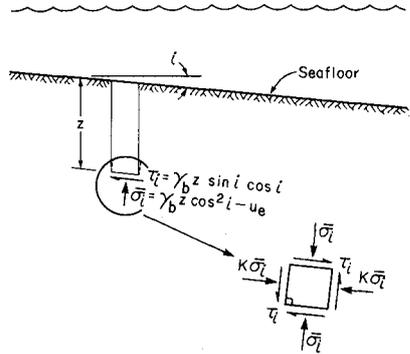
BOUNDARY CONDITIONS
 1. Points on Boundary B move identically to Points on Boundary A
 2. Bottom Fixed
 351 Elements; 391 Nodes

FIG. 11-FINITE ELEMENT MESH - ONE WAVE LENGTH

The seafloor may often be approximated as an infinite slope inclined at an angle i , as illustrated in Fig. 12. In an underconsolidated soil mass, excess hydrostatic pore pressure, u_e , results in a hydraulic force; the resultant hydraulic force being perpendicular to the surface. As shown in Fig. 12, the excess pore pressure, which is required to determine the effective stress state, may be related to the ratio of undrained shear strength of a normally consolidated soil to effective consolidation pressure (referred to as the c/p ratio), the buoyant overburden pressure, and the in situ undrained shear strength of the underconsolidated soil, S_u .

An additional assumption, which is used to compute the complete state of stress in the soil mass, is that the effective normal stresses on some two planes are proportional. For slopes less than a few degrees and for either the principal planes or planes parallel and perpendicular to the slope, the constant of proportionality between normal stresses on these planes may be about equal to the effective coefficient of earth pressure "at rest". These stress conditions are summarized in Fig. 12.

Early finite element results suggested that the magnitude of initial stress can have a significant influence on the computed soil movements. These previous analyses, however, have used stress-strain response measured on isotropically consolidated specimens. The soil in situ experiences anisotropic consolidation, and the stress-strain response of an anisotropically consolidated specimen may differ appreciably from the response of an isotropically consolidated specimen, as illustrated in Fig. 13. The comparison of secant moduli at equal levels of mobilized shear stress, shown in Fig. 13, indicates that larger soil movements may be computed with the smaller secant moduli associated with the isotropically consolidated specimen. Wright(24) attempts to include the effects of both anisotropic consolidation and cyclic loading on the soil



- γ_b : Soil Buoyant Unit Weight
- u_e : Excess Hydrostatic Pore Pressure
 $= \gamma_b z - S_u / (c/p)$
- S_u : In situ Undrained Shear Strength of Underconsolidated Soil
- c/p : Ratio of Undrained Shear Strength of Normally Consolidated Soil to the Effective Consolidation Stress, p

FIG. 12-EFFECTIVE STRESSES IN AN INFINITE SUBMERGED SLOPE

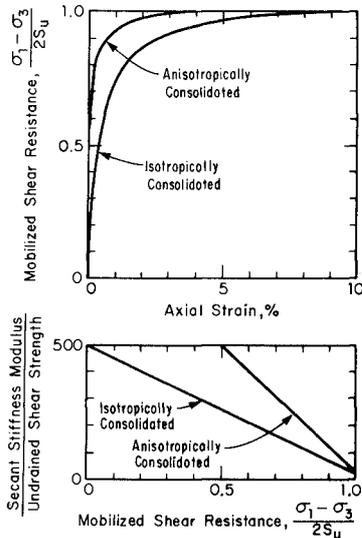


FIG. 13-COMPARISON OF STRESS-STRAIN CURVES

stress-strain response by combining the shear stresses computed from finite element analysis on a plane parallel to the slope with strains obtained from cyclic simple shear tests that simulate the computed stress history.

The second generation finite element procedure for computing wave-induced seafloor movements does not account for:

- (1) the soil stress-strain behavior for an anisotropic-consolidated condition, which is more representative of in situ conditions than the isotropic-consolidated condition;
- (2) the degradation in soil strength and stiffness that occurs due to cyclic stress reversals induced during the passage of a storm; and
- (3) the interaction between the water and soil on the development of the amplitude of wave-induced bottom pressure.

A procedure for overcoming the first two limitations is presented in subsequent paragraphs. A procedure to overcome the third limitation has been developed, but will not be reported here.

CURRENT PROCEDURE

Stress-Strain Model

The finite element model currently used is basically the same as the second generation model with the exception that the stress-strain behavior is described by a hyperbolic expression that accounts for anisotropic consolidation. Results of a recent study by Donaghe and Townsend⁽³⁾ showed that the undrained response of cohesive soils consolidated under a deviatoric stress σ_{dc} could be mathematically described by the modified hyperbolic expression:

$$\sigma_1 - \sigma_3 - \sigma_{dc} = \frac{\epsilon}{\frac{1}{E_i} + \frac{R_f \epsilon}{2S_u - \sigma_{dc}}} \dots \dots \dots (2)$$

Eq. 2 represents soil stress-strain response for a single application of a static load. With the passage of a number of waves the soils may soften and lose strength. This degradation in soil resistance is influenced by the spectrum of bottom pressure anomalies and in general decreases with penetration. The cumulative soil displacements, however, can be estimated using Eq. 2 with appropriately modified parameters, which are determined from the spectrum of bottom pressures, estimates of induced stresses from finite element analysis using unmodified stress-strain parameters, and results from laboratory cyclic testing.

Spectrum of Bottom Pressures

A spectrum of bottom pressure anomalies is developed from deep-water wave statistics and characteristics of the seafloor in the vicinity of the site. A deep-water storm wave record, such as the one in Fig. 14, can be defined by a spectrum, the number of storm waves with various combinations of periods and heights. Depending on the water depth and soils at the site, the shorter period waves

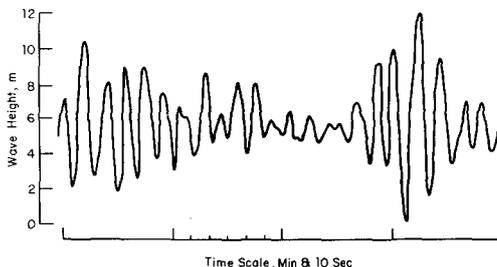


FIG. 14—FOUR MINUTE PORTION OF A TYPICAL STORM WAVE RECORD (After Tricker, 1964)

of smaller heights that do not cause significant bottom pressure amplitudes are not considered in the analysis.

The wave length L at a site with water depth D is computed for each wave period T from linear wave theory(23).

$$\left(\frac{2\pi}{T}\right)^2 = \frac{2\pi g}{L} \tanh\left(\frac{2\pi D}{L}\right) \dots \dots \dots (3)$$

where g is the acceleration of gravity. Analyses, which include the effects of refraction, reflection, shoaling and bottom friction, are then made to determine the changes in heights of storm waves as they prograde from deep water across the continental shelf and over the site. After determining the heights and lengths of waves at the site for each wave period, the bottom pressure amplitude, Δp, produced by this storm-wave spectrum is computed from

$$\Delta p = \gamma_w H/[2 \cosh(2\pi D/L)] \dots \dots \dots (4)$$

As previously mentioned, the yielding of the seafloor with the passage of waves affects the development of wave heights and bottom pressures. Although analyses are not available to fully account for this effect, through a cooperative and interactive effort between the oceanographer and geotechnical consultant, approximate methods are available to evaluate the relative influence of a yielding seafloor on the development of bottom pressure amplitude for a given wave height. These methods are based on a dynamic visco-elastic model(14). The term "yielding" refers to a failure of the soil mass and not simply "elastic" deformations for which analytic methods are available.

A 100-year storm wave spectrum along with its spectrum of wave-induced bottom pressure at a site located in about 200 ft (60 m) of water in the Gulf of Mexico is shown in Table 1. This data demonstrates that the maximum deep-water wave height does not always induce the maximum amplitude of bottom pressure.

Initial Finite Element Analyses

To obtain data for planning a laboratory test program of cyclic loading and to evaluate and interrelate cyclic tests data with finite element analyses for predicting the cumulative soil displacements during a storm, finite element analyses are first performed for a range in amplitudes and wave lengths typical of the expected spectrum on bottom pressure anomalies. Usually results are obtained for 3 to 5 wave length conditions and for 5 to 15 amplitudes. The stress-strain-strength properties of the soils used in these initial analyses are usually based on Remote Vane data(9) and on laboratory data obtained from the SHANSEP concept(10) for a single load cycle preferably applied at a rate representative of the mean period of wave loads. For a given spectrum of bottom pressures, a spectrum of stresses at any location

Wave Period, sec	7-7.9	8-8.9	9-9.9	10-10.9	11-11.9	12-12.9	13-13.9	14+
Mean Wave Length, m	87.8	112.5	139.3	167.7	196.0	224.9	254.2	282.6
Amplitude of Surface Wave in Deep Water, m	Number of Cycles/Pressure Amplitude for Average Wave, kN/m ²							
3.1 - 4.6	112	149	144	125	85	56	24	11
	1.25	2.92	5.22	8.24	10.78	13.22	13.22	16.96
4.6 - 6.1	54	66	65	56	33	28	12	7
	1.77	4.12	7.33	11.60	15.09	18.39	21.27	23.76
6.1 - 7.6	19	24	23	21	15	10	5	3
	2.30	5.27	9.44	14.80	19.40	23.85	27.00	30.56
7.6 - 9.1	4	4	7	7	6	5	2	
	2.78	6.47	11.54	18.11	23.71	29.12	33.48	
9.1 - 10.6	2	2	3	5	5	4	2	
	3.31	7.62	13.65	21.41	28.07	34.44	39.61	
10.6 - 12.1			2	3	2			
			15.76	24.72	32.38			

Note
 (1) Wave Statistics are of waves having an amplitude greater than 3.0m and a period greater than 7.0 seconds
 (2) Mean Period 9.9 seconds
 (3) Mean Amplitude 5.6 m
 (4) Water Depth at Site 57.9 m

**TABLE 1
 100 YEAR HURRICANE WAVE STATISTICS
 AND TYPICAL BOTTOM PRESSURE
 SPECTRUM**

below the seafloor can be determined from the stress distribution computed with the finite element method.

Fig. 15 illustrates a profile of computed lateral movements and the cyclic variation of shear stress on planes parallel to the slope for one wave length, but different amplitudes of bottom pressure. Fig. 15 also includes the soil properties and initial in situ stress conditions. The variation in stresses experienced by an element of soil during one wave cycle is shown in Fig. 16.

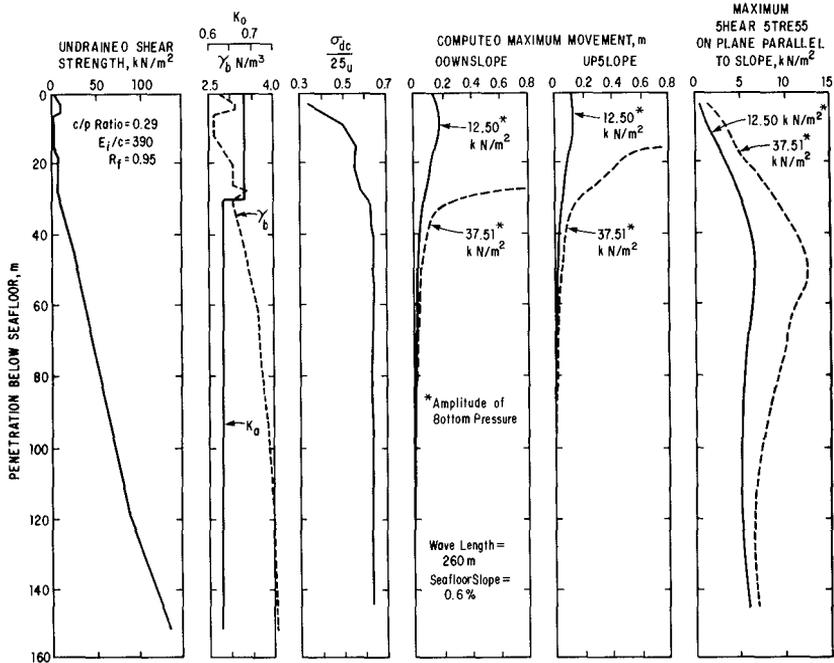


FIG. 15—SOIL PROPERTIES, LATERAL MOVEMENTS AND STRESSES

The depth of soil failure determined from the finite element results for the conditions described in Fig. 15 was approximately 45 ft less than the depth determined from a limit equilibrium analysis. Deviations between the predicted depth of slide from limit equilibrium and finite element results depend on several factors including the strength profile. The predicted depths obtained from the two procedures tend to better agree when the failure occurs in a relatively weaker soil layer at some depth⁽²⁾.

To illustrate the importance of using a stress-strain model representative of anisotropically consolidated conditions, a finite element analysis was made for the conditions shown in Fig. 15, but using stress-strain data representative of isotropically consolidated conditions. The results of this comparison, shown in Fig. 17, demonstrate that stress-strain response in terms of isotropically compared to anisotropically consolidated conditions may result in much larger predicted displacements, but

the differences in stresses computed with both data sets deviated by less than about 30 percent.

Stresses and displacements below the seafloor due to wave-induced bottom pressures can be computed for the other wave lengths in the spectrum, and combining these with the results in Fig. 15, an initial estimate of the soil stress spectrum can be developed. During the progress of a storm the seafloor response to a bottom pressure perturbation depends on the history of cyclic stresses due to previous wave action. The degree of progressive softening and loss of strength affects the stress distribution and the development of soil displacement that may accumulate during the storm. Knowing the spectrum of induced stress, a laboratory testing program is designed to provide modified stress-strain data for incorporation into the finite element analysis to predict cumulative soil displacements with the passage of a storm over a site.

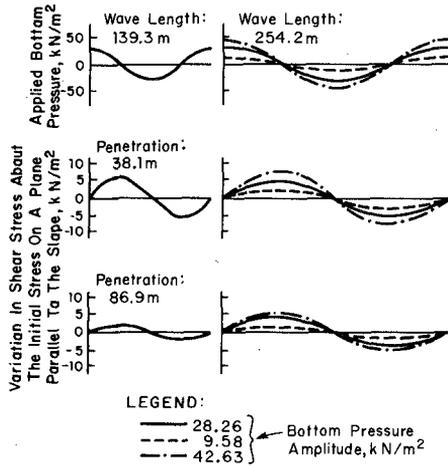


FIG. 16-STRESS VARIATIONS

Modified Stress-Strain Response

A series of cyclic stress pulses can be superposed on the initial consolidation stress in the laboratory to determine the accumulation of strain with each additional stress pulse. Laboratory tests are generally performed by either cycling with a constant stress or by applying a spectrum of stress pulses. Fig. 18 shows data obtained from cyclic tests on a Gulf of Mexico clay.

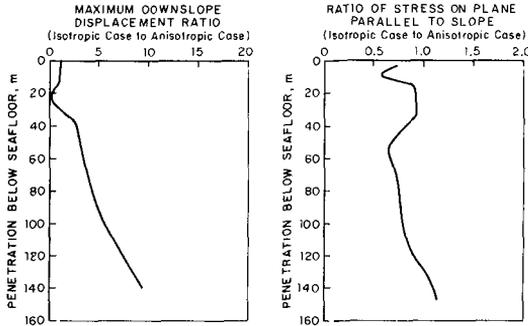


FIG. 17-COMPARISON OF RESULTS FOR ISOTROPICALLY AND ANISOTROPICALLY CONSOLIDATED STRESS-STRAIN DATA

The amount of cumulative strain and reduction in shear strength due to cyclic loading with a constant stress is shown in Fig. 19. If an element of soil experiences an equivalent of 10 stress cycles, the

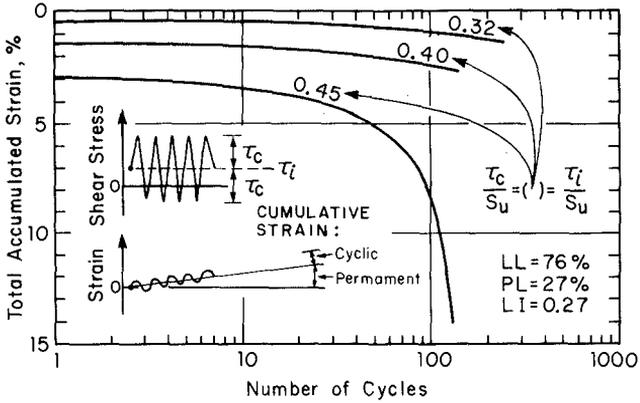


FIG. 18-CYCLIC LOADING TEST RESULTS

stress-strain curve marked $N = 10$ provides a realistic estimate of the expected cumulative displacement. Studies by Lee and Focht⁽¹²⁾ and Norwegian Geotechnical Institute^(17,18) summarize the current experience of cyclic load test data for cohesive soils. The stress-strain response of soil subjected to cyclic loading is a function of the initial stress condition about which the cyclic stress oscillates. Preliminary research results⁽¹⁸⁾ demonstrate that cyclic test data obtained for equal magnitudes of cyclic stress about different initial stresses can be equated if the results are interpreted with a normalized stress R ,

$$R = \frac{\tau_c}{S_u} \left[\frac{1}{1 - (\tau_i/S_u)^4} \right] \dots (5)$$

where τ_c = cyclic maximum shear stress; τ_i = mean maximum shear stress; and S_u = static undrained shear strength.

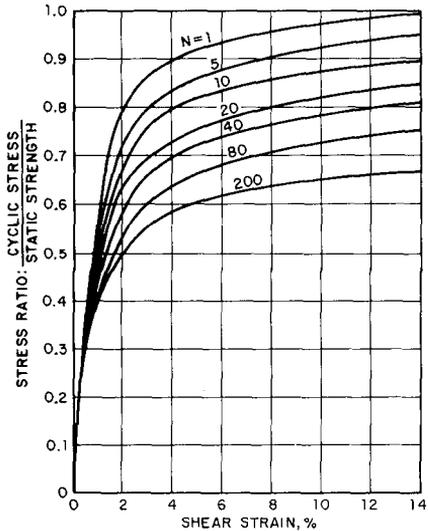


FIG. 19-STRESS-STRAIN RELATIONSHIPS AFTER CYCLIC LOADING

The degradation to soil stress-strain-strength properties induced by a series of N cycles of variable stresses can usually be equated to an equivalent number of cycles, N_{eq} , of some reference stress τ_{ref} . Combining this premise with laboratory cyclic test data and the profile of stress spectra, modified stress-strain-strength data can be generated for use with the finite element analysis to determine the accumulation of soil movements due to a spectrum of bottom pressures. The procedure used

to determine the appropriate modified stress-strain responses to predict these cumulative displacements, representative of the complete storm or some fraction of its history, is explained for a given penetration. The process is repeated to determine the degree of degradation for other penetrations.

For the purpose of analysis, it is necessary to find a cyclic stress of uniform amplitude that has the same effects as the random stress history due to the storm loading. Lee and Focht(11) have presented a method for determining this equivalency, which is based on Miner's damage potential concept(16) developed for the study of metal fatigue. The essential elements of the method are presented in Fig. 20. As shown schematically in Fig. 20(a), the cyclic stress history, which was estimated from initial finite element analysis, contains

N_i cycles of amplitude S_i . The number of cycles of stress of this amplitude, N_{if} , required to produce some predetermined strain, ϵ_p , is determined from the results of laboratory cyclic loading tests as shown in Fig. 20(c). If superposition is assumed then the same strain, ϵ_p , is produced by N_e cycles of stress with amplitude S_e . Thus, N_{if} cycles of stress of amplitude S_i are equivalent to N_e cycles of amplitude S_e . Therefore, N_i cycles of amplitude S_i are equivalent to $(N_i \cdot N_e) / N_{if}$ cycles of amplitude S_e . This procedure is repeated for every cycle in the irregular stress history to yield the number of uniform cycles of stress, N_{eq} , of amplitude S_e that produce the same accumulated strain as the irregular stress history. The equivalency is expressed by:

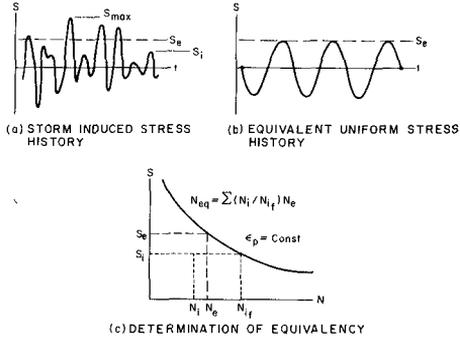


FIG. 20-CONVERSION OF IRREGULAR STRESS HISTORY TO EQUIVALENT UNIFORM STRESS HISTORY

$$N_{eq} = \sum (N_i / N_{if}) N_e \dots \dots \dots (6)$$

Theoretically the amplitude S_e of the uniform stress history equivalent to the random stress history may be selected arbitrarily. Changing S_e leads to a change in N_{eq} in Eq. 6, but any combination of N_{eq} and S_e is equivalent to any other. Each defines the number of uniform cycles required to produce a specified accumulation of strain. However, a program of laboratory tests to confirm the validity of the procedure for a given soil is recommended.

Reference Anomaly

When computing the equivalency, it is convenient to convert the irregular stress history into an equivalent number of cycles of stress corresponding to the stress induced by the "most severe" anomaly present in the spectrum being analyzed because analysis of the effects of this anomaly are used for the basic studies of seafloor response. The term "most severe" refers to the anomaly that induces the largest stresses and displacements in the seafloor. The anomaly of greatest amplitude is not necessarily the "most severe", nor does the anomaly of maximum amplitude

necessarily coincide with the surface wave of maximum height. The "most severe" anomalies are usually those of long wave length and large amplitude. The anomaly used to compute the equivalency is called the "reference" anomaly. Because the anomaly-induced stresses are a function of penetration below the seafloor, the equivalency number, N_{eq} , also varies with penetration.

Fig. 21 shows the profile of N_{eq} values representative of the conditions shown in Fig. 15 for the wave spectrum in Table 1. Fig. 21 also shows a comparison of profiles of downslope movement computed with the appropriate stress-strain curves shown in Fig. 19 for the N_{eq} profile and with the unmodified stress-strain curves. Above approximately 135-ft penetration the downslope movements accumulating with the passage of the storm exceeds the movement due to one application of a larger wave with unmodified soil properties. Below about 135-ft penetration the reverse is true. As the storm develops, the reduction in strength and softening in the upper material prevents transmission of the greater stresses due to larger waves, which generally occur near the middle of the storm.

If sufficient degradation of the soil occurs the material in the failure zone acts essentially as a fluid-like mass. Once this occurs additional accumulation of displacements below the failure zone are very small. When displacement analyses are conducted to assess the forces that must be sustained by structures founded in the seafloor, it is necessary to determine at what stage of the storm the combined effects of progressive softening and changes in the magnitude and distribution of soil movements result in maximum structural loading.

CONCLUDING COMMENT

The depth of submarine slides and profiles of soil movements, such as those illustrated in Fig. 15 and 21, are important input for analyses of soil-structure interaction and for the design of foundations at sites susceptible to submarine slides. Evaluation of the soil movement problem and the effect of soil movements on offshore platform design requires an interaction between the oceanographer, marine geologist, geotechnical engineer, and structural engineer. The scope of this paper has been limited to only oceanography and geotechnical considerations required to estimate soil movements.

The state-of-the-art for prediction of soil movements due to wave-induced bottom pressures has improved significantly during the past eight years. The procedures presented here were based on a static analysis, even though the wave-induced bottom pressures are dynamic loads. Although static analyses are often appropriate for the 10 to 15-second period loads, prediction of soil response in the failure zone and the transfer of stresses into the unfailed soils would be better modelled with a dynamic analysis. More work, however, is required to better define the behavior of soils in post-failure conditions before further improvements in the state-of-the-art can be expected.

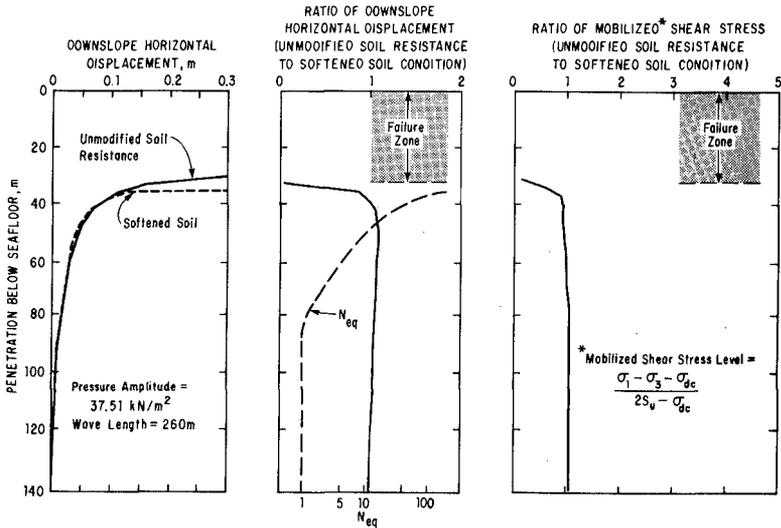


FIG 21-COMPARISON OF LATERAL MOVEMENTS AND STRESSES FOR UNMODIFIED AND SOFTENED SOIL

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APPENDIX II - NOTATION

γ_b	= soil buoyant unit weight
γ_w	= unit weight of water
Δp	= bottom pressure amplitude
ϵ	= axial strain
ϵ_f	= failure strain
ϵ_p	= predetermined amount of strain
ϵ_{50}	= axial strain at a shear stress equal to $1/2 S_u$
σ_{dc}	= deviatoric consolidation pressure
$\bar{\sigma}_i$	= effective normal stress on plane parallel to slope
$\sigma_1 - \sigma_3$	= deviatoric stress
τ_c	= cyclic maximum shear stress
τ_i	= mean maximum shear stress or stress on a plane parallel to infinite slope
τ_{ref}	= reference shear stress
c/p	= strength ratio
D	= water depth
E_i	= "elastic" tangent modulus at $\epsilon = 0$
E_{ur}	= "elastic" unloading modulus
g	= gravitational constant
H	= wave height
i	= slope angle
K	= earth pressure coefficient
L	= wave length
N	= number of cycles
N_e	= number of cycles of amplitude S_e
N_{eq}	= equivalent number of cycles
N_i	= number of cycles of amplitude S_i
N_{if}	= number of cycles of amplitude S_i required to cause a strain of ϵ_p
R	= normalized stress

R_f	=	failure ratio, influence of the failure strain ϵ_f
S_e	=	stress amplitude
S_i	=	stress amplitude
S_{max}	=	maximum stress amplitude
S_u	=	undrained shear strength
t	=	time
T	=	wave period
u_e	=	excess hydrostatic pore pressure
z	=	penetration below seafloor