CHAPTER 84

FAILURE OF SUBMARINE SLOPES UNDER WAVE ACTION

by

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ABSTRACT

The results of model tests, carried out to evaluate the stability of submarine slopes under wave action are presented. A Bentonite clay was sedimented in a glass walled tank 6 feet long by 0.5 feet wide by 2.5 feet deep. The sedimentation and consolidation processes were studied and sediment densities were measured at various depths in the profile. Vane shear strength profiles were also measured at various average degrees of consolidation. Plastic markers were placed in the sediment adjacent to a glass wall so that the soil movements under both gravity and wave induced slides could be documented by photography. Dimensional similitude is discussed and the model test data are presented in a dimensionless form.

All instabilities were observed to be of the infinite slope type. Analysis of the data shows that wave action is instrumental in initiating downslope mass movements in gently to steeply sloping off-shore sediments. General lack of agreement between the model test results and published theoretical analyses was found but there was close similarity in the depths and form of failure under wave action and under gravity stresses alone. The loss of stability under wave action is analyzed on the concept that failure is gravity controlled and the soil strength is reduced to a value commensurate with gravity sliding by the cyclic shearing stresses imposed by progressive waves. A method of evaluating the stability of prototype slopes using a model test correlation and field vane strength measurements is proposed.

INTRODUCTION

Instabilities in submarine slopes have been observed or have been inferred over a wide range of slope angles from less than half a degree up to about 30°. These subaqueous landslides are believed to have caused rupture of submarine cables and to have generated many of the geomorphological features on the ocean bottom. There are numerous records describing these landslides but very few publications discuss the application of the principles of soil mechanics to the analysis of the stability of submarine slopes.

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One of the particular features of many marine sediments is the time lag between the accumulation of sedimenting materials and the dissipation of excess pore water pressures which develop within the sediment. Such sediments are described as being underconsolidated. The excess pore water pressure at any level in the deposit will reduce the effective stresses and the undrained shear strength below values appropriate to the fully consolidated state. Underconsolidated sediments are, therefore, less stable than consolidated sediments.

In areas where oversteepening of slopes develops as a result of local erosion or crustal tilting gravitational forces may be sufficient to cause landslides (Terzaghi, 1956). Earthquakes and other seismic activity are widely accepted as causal agencies in submarine landsliding (Gutenberg 1939; Ambraseys 1960; Morgenstern 1967). Submarine landsliding has occurred in the absence of seismic activity however (Dill 1964; Chamberlain; 1964) and Henkel (1970) shows that the bottom pressure pulses due to ocean waves could be a major factor in initiating movements. This paper is concerned with the role of waves in initiating submarine landslides.

One of the effects of ocean waves is to produce pressure changes within the water below the surface and pressure pulses on the surface of the sediment. As a wave passes a pressure increases δp above the mean hydrostatic bottom pressure is felt below the crest while beneath the trough there is a pressure decrease $-\delta p$. The magnitude of the excess pressure δp, which is in phase with the wave, depends on the wave length L, wave period T, wave height H and the depth of water d, (Wiegel, 1964) and is given by:

$$\delta p = \frac{H}{z} \quad \frac{\cosh k \ (z + d)}{\cosh kd} \quad \cos (kx - \sigma t) \tag{1}$$

where $k = \frac{2\pi}{L}$ and $\sigma = \frac{2\pi}{T}$

At the mudline ($z = -d$) the excess bottom pressure is distributed as shown in Figure 1 and may be calculated as:

$$\delta p = K_p \eta \tag{2}$$

where $K_p = \dfrac{1}{\cosh kd}$ is the pressure response factor on the top of sediment

$\eta = \dfrac{H}{2} \ \cos (kx - \sigma t)$ is the fluctuation of water level due to waves

Values of K_p are tabulated in standard oceanography texts. It may be noted that the excess pressure becomes negligable when the depth of water d is greater than half of the wave length L. Because of the difference in pressure under crest and trough, the passage of a wave induces shear stress in the soil. As the wave passes, soil at a particular point experiences cyclic fluctuation in magnitude and direction of these induced stresses.

(a) ROTATIONAL SLIDING (HENKEL, 1970)
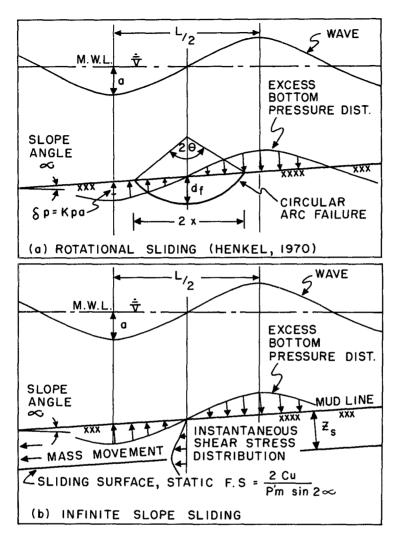
(b) INFINITE SLOPE SLIDING

FIGURE 1 STATIC REPRESENTATION OF POSSIBLE FAILURE MODES

Considering static moment equilibrium on a circular arc sliding surface beneath a standing wave (Figure 1a) Henkel (1970) demonstrated that the excess bottom pressures at a given instant are sufficient to cause shear failure in underconsolidated soft sediments. Henkel substantiated his calculations using strength data from sediments in the Mississippi delta and he suggests that under a natural progressive train of waves, a sequence of circular arc slides would result in progressive down slope movement of the sediment. The critical depth of failure, d_f, in Figure 1a, is shown by Henkel to be a function of the wave length and excess bottom pressure. Henkel also calculates the subsurface pressure required to cause instabilities for a range of wave conditions.

Mass downslope movements of sediments could also develop in the form of infinite slope sliding as shown in Figure 1b. The distribution of shearing stresses in the sediment under a given excess pressure distribution may be estimated from the deformation characteristics of the soil (Wright and Dunham, 1972). At some depth in the sediment the shearing stresses may exceed the shearing strength and sliding would develop on a surface almost parallel to the mudline (for small slope angles). The depth of failure, Zs, in Figure 1b will be a function of the stress-strain strength properties of the sediment and the wave characteristics. Both failure mechanisms shown in Figure 1 are applied to the idealized situation of a uniform soil profile deposited and sedimented in a single layer. Layered materials and time dependent accumulation of sediments are not considered in this paper. Henkel's analysis is an upper bound solution while the analysis due to Wright and Dunham is a lower bound solution with soil movements being predicted using a non-linear stress-strain relationship.
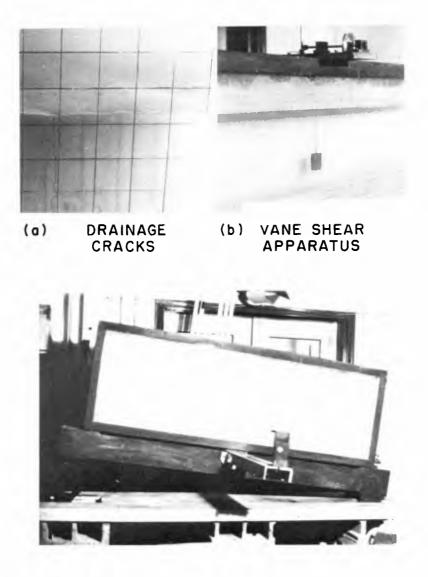
Two dimensional model studies of slope failures initiated by wave action are described in this paper. The wave height (for a given d/L ratio) required to initiate failure, the depth of failure and the form of the soil movements were measured. The data are discussed with reference to the theoretical developments noted above.

MODEL STUDIES

Dimensional considerations

If the gravitational forces acting on an element of soil are to be correctly modelled it is generally necessary that the model and prototype be composed on the same material (Roscoe, 1969). A montmorillonite clay (commercial Bentonite) was chosen as the sediment and was mixed with distilled water which contained sodium chloride at a concentration of 35 p.p. thousand by weight. A 10 percent by weight suspension of clay was prepared for testing. Sea water was represented in the wave flume by a mixture of 35 parts sodium chloride per thousand parts of water. A model tank of dimensions 6 ft. long by 2 ft. deep by 0.5 ft. wide was chosen from the following considerations:

(i) the depth of sediment would be in excess of anticipated depths of failure.

(a) DRAINAGE CRACKS   (b) VANE SHEAR APPARATUS

(c) SLOPE AFTER STATIC FAILURE

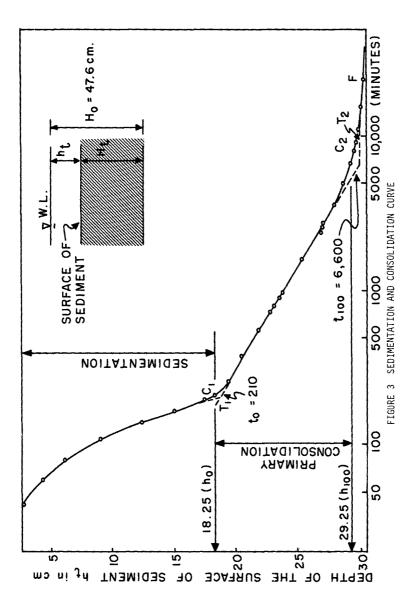FIGURE 2   CONSOLIDATION AND STATIC TILTING EXPERIMENTS

(ii) the depth of water to wave length (d/L) ratio would be within
      the range where excess bottom pressure are considered significant
      in initiating sliding.

(iii) the tank length would be in excess of 1.5 wave lengths so that
      the form of the movements would not be unduly influenced by
      boundaries.

(iv) the width to length ratio of the tank would be representative of
      plane strain conditions.

(v)   economic considerations.
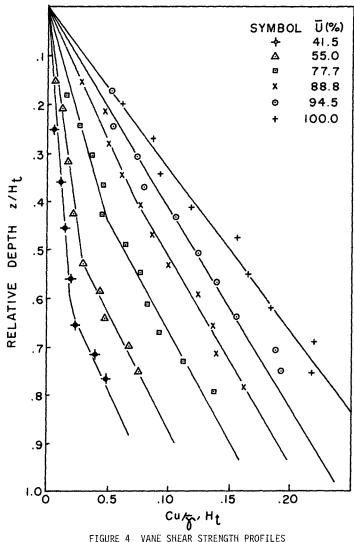
In a sedimented underconsolidated or normally consolidated soil profile
the undrained shear strength Cu of the soil is expected to increase
linearly with depth and is generally expressed as a constant Cu/P_0' ratio
where P_0' is the effective overburden pressure at a given depth ^{0} ($P_0' = Z\bar{\gamma}'$).
This definition of p' does not account for any excess pore water pressure
that will exist in underconsolidated soils. The consolidation process and
average degree of pore water pressure dissipation may be characterized by two
dimensionless parameters; the degree of consolidation $\bar{U}$ (often expressed as a
percentage) and the time factor Tv which may be related to the soil
compressibility, soil coefficient of permeability and length of the drainage
path. For circular arc sliding in a soil exhibiting a linear increase in
strength with depth, Gibson and Morgenstern (1961) have shown that the
factor of Safety (F.S.) is independent of the slope height. The independence
of failure on slope height is apparent for infinite slope sliding. Thus
either of the failure mechanisms shown in Figure 1 could develop in the
model tank. A slope studied in the model tank is considered to be identical
to a prototype slope having the same slope angle α, and the same Cu/P_0' ratio.
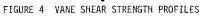
Measured depths of failure and wave heights (or excess bottom pressures)
required to cause failure in the model slopes may be used to evaluate the
theories advanced. The scale factor used for these parameters is in the
order of 100 to 500.

Sediment preparation

The Sedimentation and consolidation processes were studied in the model
tank shown in Figure 2c. Vane shear tests (Figure 2b) and excess pore
water pressure measurements were carried out at various stages of consolidation.
Variations in density through the soil profile were also measured. These
studies were carried out to provide background data for interpretation of the
stability studies and the details of apparatus and procedure are outlined by
Tsui (1972). Only those data directly related to the stability problem are
presented in this paper. A typical sedimentation-consolidation curve for
the test sediment is plotted in Figure 3. During the latter stages of
sedimentation drainage channels with associated surface boils were observed
to form in the sediment (Figure 2a). The beginning of primary consolidation
was marked by an abrupt change in the rate of settling of the mud line,
closing of the drainage cracks and the commencement of excess pore water
pressure dissipation. The average degree of consolidation defined by settlement
of the mudline correlated well with the average degree of consolidation

FIGURE 3 SEDIMENTATION AND CONSOLIDATION CURVE

FIGURE 4   VANE SHEAR STRENGTH PROFILES

calculated from pore water pressure isochrones. After each experiment the soil was thoroughly mixed using an air injection stirring system and sedimented to the desired average degree of consolidation with reference to Figure 3.

Vane shear strengths, measured at various average degrees of consolidation, were found to vary non-linearly with depth as shown by Figure 4. This variation is thought to be related to density variations within the material or to thixotropic effects.

Static tilting tests

In order to arrive at suitable slope angles for the wave flume experiments a series of static tilting tests were carried out. Prior to these tests (at an average degree of consolidation of about 10%) plastic markers were inserted in the sediment adjacent to the glass wall. During latter stages of consolidation and subsequent slope deformations the movements of these markers were recorded by photography. Slope deformation and ultimate failure are produced by jacking one end of the tank such that the tilting rate was about 3 degrees per minute. Sequential photographs of the movements of plastic markers for a typical experiment are reproduced in Figure 5. Discontinuous curved shear surfaces were observed to form and produce a visible rupture zone when the displacements of the markers were quite small. Jacking was discontinued at this stage but the marker movements and rupture zone continued to develop with time. An enlarged view of the ultimate failure zone is shown in Figure 5. The displacements of markers at various times were measured from photographs with reference to a 2 cm. grid marked on the glass wall and are plotted for this typical test in Figure 6. The depth and thickness of the rupture zone and the depth of the ultimate failure plane were measured from visual observation and from the plotted marker displacements. These and other data from the series of static tilting experiments are summarized in Table 1. From the marker measurements the average shearing strain in the rupture zone at the time when this zone was first visible is estimated to be about 6%.

From equilibrium considerations it may be shown that, at failure

$$\frac{Cu}{\overline{\gamma}'Z_s} = \frac{Cu}{P'_m} = \frac{1}{2} \sin 2\alpha f \qquad (3)$$

where Z_s = depth to the sliding surface from the mud line.

$\quad$ $\overline{\gamma}'$ $\quad$ is the submerged density averaged over the depth Z_s

$\quad$ αf $\quad$ is the angle at which rupture was initially observed (failure angle)

$\quad$ Cu $\quad$ is the undrained strength of the sediment at depth Z_s

The values of undrained strength derived from static tilting experiments (equation 3) and those measured by vane tests are shown, on Figure 11, to be in good agreement. Up to about 50% of full primary consolidation the Cu/P'_o ratio

t = 0
EQUILIBRIUM

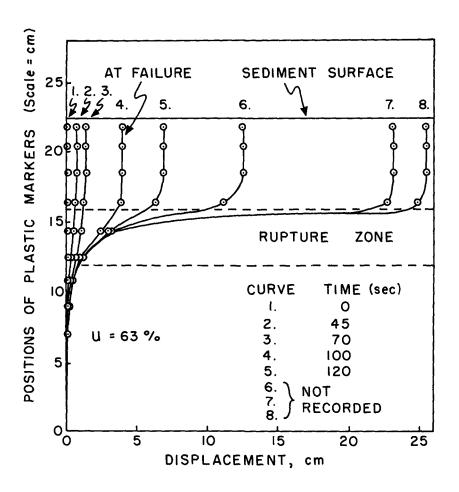t = 45 sec
PRE-FAILURE

t = 100 sec
FAILURE

t = 120 sec
POST FAILURE

t ≈ 150 sec
POST FAILURE

FAILURE ZONE

FIGURE 5   SEQUENTIAL PHOTOGRAPHS OF MARKERS IN STATIC TILTING EXPERIMENT

FIGURE 6   DISPLACEMENT OF MARKERS IN STATIC TILTING EXPERIMENT

TABLE 1   RESULTS OF STATIC TILTING EXPERIMENTS

NO. OF TEST	HEIGHT OF SEDIMENT H_t in cm	AVERAGE DEGREE OF CONSOLI-DATION $\bar{U}$ in %	FAILURE ANGLE OF SLOPE α_f IN DEGREES	c_u / p'_m RATIO	DEPTH OF SLIDING SUR-FACE z_s in cm	DEPTH OF RUPTURE ZONE z_f in cm	z_s / H_t	z_f / H_t
1	24.00	48.5	3.10	.0558	10.00	13.00	.417	.542
2	25.25	37.5	2.50	.0436	13.25	15.25	.525	.603
3	22.40	63.0	4.40	.0765	8.40	10.40	.375	.464
4	27.88	13.0	1.09	.0191	18.88	19.88	.676	.713
5	21.10	75.0	6.70	.1160	6.60	8.10	.313	.384
6	21.85	68.3	5.20	.0903	7.35	8.85	.336	.405
7	23.00	57.7	4.10	.0713	9.50	11.00	.413	.478
8	23.95	49.0	3.00	.0523	10.95	13.95	.457	.583
9	28.65	6.5	0.50	.0088	19.65	21.15	.690	.740
10	26.10	29.5	1.70	.0297	15.60	17.10	.598	.655
11	26.50	25.8	1.70	.0297	15.50	17.50	.585	.660
12	24.80	41.5	2.30	.0401	12.80	14.80	.516	.597
13	20.55	80.0	8.00	.1430	6.55	7.55	.319	.367
14	20.80	77.7	8.20	.1412	6.80	7.80	.325	.375
15	24.00	48.5	3.37	.0587	10.00	13.50	.417	.563
16	19.90	86.0	10.90	.1857	5.90	6.90	.297	.347

increases linearly with degree of consolidation (Figure 11) as predicted
by Morgenstern (1967); for higher degrees of consolidation the rate of
strength increase with consolidation is much higher. This behaviour may
be a result of variations in submerged unit weight with depth, the large
strains involved in consolidation under self weight, and possible thixotropic
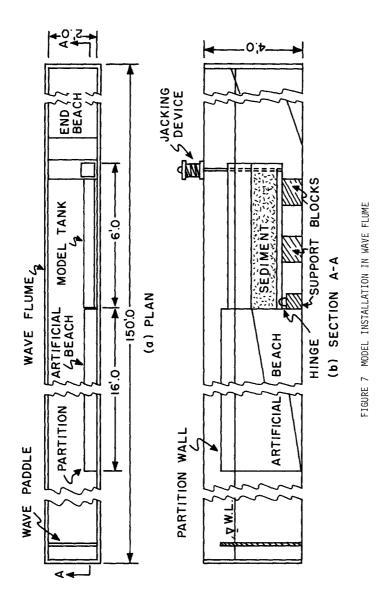effects.

Tests under wave action

    Figure 7 shows the model arrangement in the wave flume. The model
tank used in the wave flume was equipped with sliding end sections that
were lowered to the level of the mud line prior to each test. The artificial
beach was adjusted to the elevation of the mudline to present a smooth
continuous bottom transition from this beach to the mudline. The waves
were generated by an adjustable wave paddle located in a deep section at
the upstream end of the flume and the wave energy was dissipated on a horse
hair beach at the downstream end of the flume. The flume is equipped with
a wire mesh filter and partitioning board to prevent undesirable waves from
reaching the test section. A smooth progressive sinusoidal wave train of
constant characteristics was obtained over the test section.

    Most of the tests were carried out using a wave length of 58 cm and a
d/L ratio of about 0.35. The sediment was prepared in a manner identical
to that used in the static tilting experiments and the tank was tilted to
an angle such that the factor of safety against gravitational failure was
at a known value between 1.25 and 2.80. Displacements of the plastic
markers as a result of static tilting were quite small and an equilibrium
condition was established immediately. The slope was then subjected to a
progressive wave of small amplitude and the height of the wave was increased
incrementally. Each increment of wave height was allowed to act on the
sediments until the progressive downslope displacement of the plastic
markers had terminated. Movements of markers in fixed elliptical orbits
was considered to represent an equilibrium situation. The increment durations
varied from about 2 minutes at the start of a test to about 10 minutes toward
the end of a test. The incremental increases in wave height were chosen
so that a failure condition, defined by continued and often accelerating
downslope movements of the markers, was reached after five or six increments.
Equilibrium positions of markers at the end of certain increments, as
documented by photography, are shown for a typical test in Figure 8. A
rupture zone with shear surfaces similar to these observed in static tilting
and was found to develop with the larger slope movements associated with
failure. The marker displacements measured from the photographs reproduced
in Figure 8 are plotted in Figure 9 and a summary of data from a number of
tests is presented in Table 2.

DISCUSSION OF MODEL TEST DATA

    In all static tilting tests and all tests carried out under wave action
the observed failure mode is best described as infinite slope sliding. The
sketch in Figure 10 shows the approximate form of the rupture zone and the
influence of the rigid ends of the model tank. The depth of this rupture

FIGURE 7 MODEL INSTALLATION IN WAVE FLUME

BEFORE TILTING   AFTER TILTING   EQUILIBRIUM WAVES

AT FAILURE   POST FAILURE   POST FAILURE
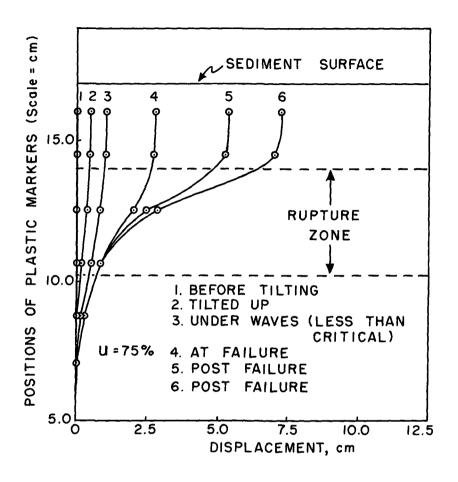
FIGURE 8 SEQUENTIAL PHOTOGRAPHS OF MARKERS UNDER WAVE ACTION

FIGURE 9   DISPLACEMENT OF MARKERS UNDER WAVE ACTION

TABLE 2 RESULTS OF STABILITY TESTS ON SLOPES SUBJECTED TO PROGRESSIVE WAVES

NO. OF TESTS	HEIGHT OF SEDIMENT H_t (in cm.)	AVERAGE DEGREE OF CONSOLIDATION U (in %)	FAILURE ANGLE OF SLOPE, $f\infty$ (in degrees)	ANGLE OF SLOPE ∞ (in degrees)	FACTOR OF SAFETY F	DEPTH OF SLIDING SURFACE, z_s (in cm.)	DEPTH OF RUPTURE ZONE, z_f (in cm.)	H_t/z_s	H_t/z_f	DEEP WATER WAVE LENGTH L_o (in cm.)	WAVE HEIGHT H (in cm.)	DEPTH OF WATER d (in cm.)	d/L_o	PRESSURE RESPONSE FACTOR K_p	CRITICAL SUBSURFACE PRESSURE, δp_c (in gm/cm²)	RESERVED STRENGTH, S_r (in gm/cm²)
1	21.9	53.8	3.55	1.60	1.970	8.9	10.9	.406	.497	58	.75	22.4	.386	.1695	.0652	.0282
2	21.2	62.4	4.50	2.32	1.800	7.2	9.2	.340	.433	58	.73	21.7	.374	.1818	.0680	.0253
3	18.6	83.0	9.50	4.70	2.010	5.1	5.5	.270	.296	58	.75	17.3	.298	.2790	.1070	.0482
4	21.1	62.5	4.50	3.46	1.300	7.6	10.0	.360	.475	58	.45	18.6	.320	.2472	.0568	.0145
5	19.5	75.0	6.20	3.60	2.000	6.3	7.3	.325	.373	58	.80	19.7	.340	.2210	.0906	.0376
6	23.1	43.5	2.70	1.65	1.635	10.6	13.1	.458	.567	58	.50	21.2	.366	.1904	.0487	.0200
7	18.9	76.5	7.20	2.87	2.510	5.4	6.9	.286	.365	58	1.25	22.2	.382	.1735	.1110	.0407
8	20.6	62.9	4.55	1.60	2.840	6.6	8.6	.320	.417	58	1.20	24.2	.417	.1411	.0866	.0338
9	22.8	47.2	3.55	2.02	1.475	10.4	12.5	.457	.548	58	.53	21.4	.369	.1872	.0508	.0180
10	22.5	40.2	2.55	2.07	1.265	11.6	13.6	.517	.605	58	.30	20.7	.357	.2005	.0309	.0114
11	19.8	39.9	2.50	1.70	1.470	10.1	12.8	.511	.644	58	.45	22.1	.382	.1735	.0402	.0170
12	22.0	62.3	4.50	2.17	2.081	9.2	10.9	.419	.495	58	1.00	23.7	.409	.1480	.0758	.0353
13	17.0	52.4	3.30	1.50	2.200	7.8	9.5	.460	.560	58	1.05	25.6	.442	.1214	.0655	.0316
14	18.0	75.4	6.90	2.70	2.560	5.5	7.2	.306	.400	65	1.20	24.8	.381	.1745	.1075	.0388
15	20.3	60.0	4.33	2.15	2.020	8.3	10.5	.408	.516	58	.95	24.4	.421	.1378	.0671	.0311
16	21.7	46.0	2.90	1.50	1.930	11.0	13.1	.507	.606	65	.65	24.6	.378	.1776	.0593	.0271
17	21.7	33.9	2.15	1.10	1.940	11.7	13.7	.540	.631	74	.35	24.3	.328	.2506	.0450	.0225

FIGURE 10   MODE OF FAILURE OF A SUBMERGED SLOPE IN STATIC AND WAVE EXPERIMENTS

FIGURE 11 UNDRAINED STRENGTH VERSUS CONSOLIDATION

zone and the depth of the sliding surface was found to vary with the average degree of consolidation as shown on Figure 12. When the sediment was fully consolidated the downslope movement of sediment was confined to a thin surface layer. The few tests carried out using wave lengths different from 58 cm (hence a different d/L ratio for a given degree of consolidation) showed no influence of wave characteristics on the depth of failure. Since there is considerable variation in the strength-depth relationships in prototype situations (Bea, 1971; Sangrey, 1972) the relative influences of soil characteristics and wave characteristics on the depth of sliding appears to warrant further study. In an attempt to produce a circular arc slip a slope of about 10 cm in height was excavated in a fully consolidated sediment at an angle of approximately 25 degrees. When subjected to wave action this slope gradually flattered out by downslope transport of thin layers (2 cm thick) of sediment and no rupture or slip circle was observed.

The excess bottom pressure required to initiate mass movement of the sediment (critical bottom pressure, δp_c) increased with increased factor of safety (F.S.). In order to compare tests carried out at different F.S. a reserve strength (Sr) was defined as

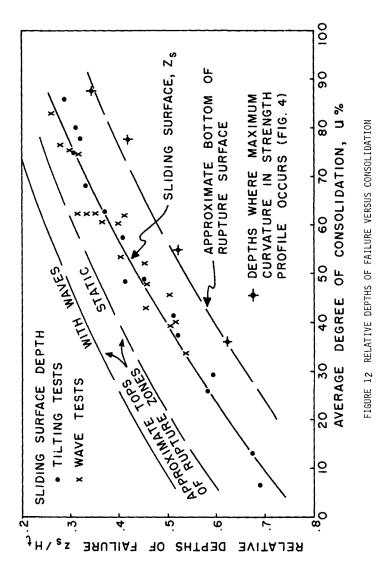$$Sr = \frac{\bar{\gamma}' z_s}{2} \ (\sin 2\, \alpha_f - 2\, \alpha) \qquad (4)$$

where α_f is the failure angle in static tests and α is the angle of the slope subjected to wave action.

From the plotted data in Figure 13 a linearly relation can be proposed between the critical bottom pressure and the reserve strength. Then the dimensionless parameter indicative of a failure situation is $\delta p_c/Sr$ and failure was observed to occur when

$$\delta p_c/Sr = 2.40 \qquad (5)$$

While it would be possible to use the model data presented in Figures 12 and 13 directly to analyze the stability of a prototype sediment having similar strength vs depth characteristics it is preferable to compare the model test observations to theoretical analysis in an attempt to evaluate a more general and rational approach to prototype stability calculations.

The fact that circular arc sliding was not observed should not be regarded as sufficient evidence to discard the upper bound stability analysis sketched in Figure 1a. The dynamic nature of the problem was recognized by Henkel (1970) and it is not difficult to picture that the cumulative effect of rotational movements under the action of progressive waves would be mass downslope movement. The circular arc is merely a convenient mechanism to obtain a numerical solution. The lower bound solution published recently by Wright and Dunham (1972), however, offers the advantage of making predictions of soil stresses and displacements prior to failure (or for bottom pressures less than the critical pressure) so that the effects of these movements on various types of foundations could be evaluated. The predictions of most general significance, however, are still the critical bottom pressures (or wave characteristics that will induce failure) and the maximum depths of the failures.

FIGURE 12 RELATIVE DEPTHS OF FAILURE VERSUS CONSOLIDATION

$$Sr = \left(1 - \frac{1}{F.S.}\right) \overline{\gamma}' \, \overline{z}_s \, \frac{\sin 2\alpha}{2}$$
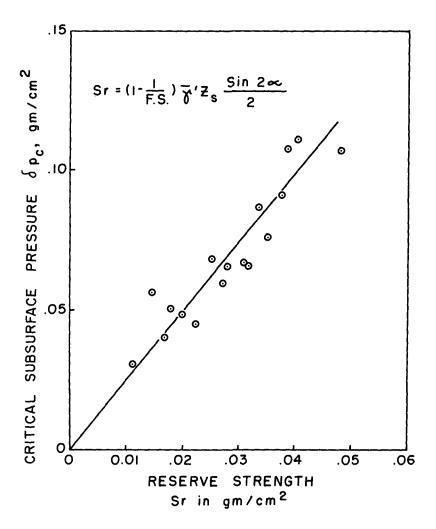
FIGURE 13   CRITICAL BOTTOM PRESSURES FOR SLOPES UNDER WAVE ACTION

For a simple soil profile having an undrained strength increasing linearly with depth both analyses indicate that failure begins at the mudline. Henkel's (1970) solution predicts a threshold value of excess bottom pressure, δp , when surficial soil failure begins. As δp increases this technique predicts an increasing depth below mudline for the failure. Model test failures did not begin as surficial or shallow displacements under small waves, instead the system was stable until an initial failure at significant depth. This is obviously inconsistent with Henkel's predictions; however, there was a reasonable relationship between the observed depth of failure and the prediction based on the waves acting at that time. Agreement between observed depth of failure and prediction was generally within 40%.

Wright and Dunham (1972) also predict that the maximum horizontal displacements and maximum ratio of applied stress to available strength will occur at the surface for a simple soil profile. Deeper stressed zones are associated with higher excess pressures. In general, this lower bound technique predicted maximum stress above the zone that actually failed in weaker soils and below the actual failure for stronger soils.

The discrepancies between the model test observations and the analytical predictions regarding the progress of failure from the mudline down may be due to the assumption that the soil has zero strength at the surface. It is more likely that the soil has a small finite strength at the surface derived from interpartical electro-chemical forces.

STRENGTH REDUCTION ANALYSIS

Previous analytical approaches are based on the concept that some additional downslope force must be provided to overcome a known shearing resistance. The discussion that follows is based on the concept that gravitational forces alone cause down-slope mass movement (failure) and the wave serves only to cause strength reduction in the soil. The undrained shearing strength at any depth in the soil profile may be effectively reduced due to excess pore water pressure build up or disturbances generated due to the cyclic shearing stresses (or strains) induced by a passing wave. To bring about failure the necessary percentage reduction in soil shearing resistance (or measured undrained strength) may be calculated as

$$R = \frac{S_r}{Cu} = \left(1 - \frac{Sin\ 2\alpha}{2Cu/_{p'm}} \right) \times 100\% \tag{6}$$

For a normally consolidated clay with a sensitivity of about 2, Sangrey, Henkel, and Esrig (1969) show that the strength can be reduced to the fully remoulded (not slickensided) critical state strength under cyclic loading. Following this general approach, and assuming that the minimum strength that can occur is the fully remoulded critical state strength, wave induced failure will occur only if the sensitivity of

the soil be greater than or equal to the factor of safety against gravitational failure. Thus, for waves to initiate failures

$$St = \frac{Cu\ undisturbed}{Cu\ remoulded} \geq F.S. \qquad (7)$$

In the model tests carried out in this study, values of factor of safety between 1.3 and 1.8 corresponding to strength reductions of 22% to 64% were used. It is expected that sedimented marine clays will have sensitivity ranging from about 0 to 10 (Sangrey, 1972). It may also be expected that the sensitivity will increase as the average degree of consolidation increases (see Figure 12).

For this series of model tests, equation 5 may be used directly with equation 6 to express the bottom pressure required to initiate failure (δp_c) in terms of the soil strength and the slope angle (or F.S. against gravitational failure) as:

$$\delta p_c = 2.4\ Cu\ (1 - \frac{Sin\ 2\alpha}{2^{Cu}/P'_m})$$

$$\qquad (8)$$

$$\delta p_c = 2.4\ Cu\ (1 - ^1/F.S.)$$

From these simple expressions the critical excess bottom pressure may be calculated at any depth for a given slope. As with former analytic approaches these expressions predict that surficial failure would develop initially for a simple soil profile (Cu/P'_0 constant). Since the failure is envisaged to be induced by wave action but gravity controlled, however, any increases in density with depth or any finite surface strength may lead to a more critical sliding surface at some depth in the profile. Using the measured vane shear strengths at the observed depths of failure, equation 8 was found to predict the value of δp_c to within ± 15% for the model tests.

Predicting the depth of failure appears to be a most formidable problem and should be done, in all cases, with respect to the strength profile. Weak layers or non-linear strength profiles may be considered directly using equation 8 although it is possible that extremely soft zones in some profiles (Bea, 1971) are wave induced (i.e. the soil has been softened or remoulded to some degree due to wave action). Equation 8 indicates that for a flat lying sediment (F.S. → α) an excess bottom pressure of 2.4 Cu would completely remould the soil to some depth. The depth of major remoulding may be expected to correspond with the depth of maximum cyclic shearing stress (or shearing strain). From the elastic predictions of Wright and Dunham (1972) the maximum shearing stress occurs at a depth of 0.19 Lo independent of variations in soil deformation modulus and is fairly uniform across the model. For the model value of Lo = 58 cm the depth of maximum cyclic shearing stress would then be 11 cm.

In discussing the general problems of submarine slope stability it

should be noted that the published theoretical solutions are essentially
two dimensional. Henkel (1970) incorporates gravitational forces in
his analysis but Wright and Dunham (1972) do not. Questions arise
regarding the effect of a wave train progressing at some angle to the
dip of the slope. The concept of wave action producing strength
reduction and gravity producing slope instability is essentially
independent of wave direction and mass movement would always occur down-
dip. Model studies appear to be the most appropriate method of
investigating this more general condition.

CONCLUSIONS

1. The usefulness of model studies in investigating submarine
    landsliding has been demonstrated. Model studies show that
    wave action can initiate an infinite slope type of failure
    in bottom sediments.

2. Contrary to analytical predictions the initial failure for
    a simple sedimented soil profile occurred at some finite
    depth in the profile. The depth of failure was observed to
    decrease almost linearly with increased average degree of
    consolidation.

3. The critical bottom pressure (calculated from the wave
    characteristics) required to initiate failure was found to
    increase linearly with the reserve strength (defined as
    the strength in excess of that required for stability of the
    slope in the absence of wave action).

4. For the observed depths of failure in sedimented soil
    profiles the upper bound solution due to Henkel (1970)
    appears to over-estimate the excess bottom pressure required
    to cause failure by a factor of 1.5 to 2.0.

It is suggested that the mechanism of failure may involve strength
reduction due to cyclic shearing stresses (or strains) induced by wave
action. An alternative method of calculating the critical excess
bottom pressure is proposed. This method uses an empirical relationship
derived from the model tests together with insitu vane strength data.
The proposed failure mechanism is independent of wave train orientation
and indicates that wave action cannot induce mass movement unless the
soil has a sensitivity greater than or equal to the gravitational
factor of safety.

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LIST OF SYMBOLS

α Half of the wave height, H .

C_u Undrained strength

d Mean depth of waver above mud line

d_f Depth of circular arc failure

F.S.   Factor of safety of a slope against gravitational sliding

H Wave Height

H_t Height of sediment at time t

k $2\pi/L$

L Wave length

L_o Deep water wave length

p_o' Vertical effective stress calculated from $Z\overline{\gamma}'$

p_m' Vertical effective stress at depth of failure

R Strength reduction factor

S_r Reserve strength

T Wave period

U Average degree of consolidation

z_f Depth of rupture zone

z_s Depth of sliding surface

α Angle of slope

α_f Failure angle of slope

$\overline{\gamma}'$ Average submerged density

η Fluctuation of water level due to wave

σ $2\pi/T$

δp_c Critical maximum wave pressure at slope failure

δp Maximum wave pressure under crest of wave