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BREAKOUT RESISTANCE OF OBJECTS EMBEDDED IN OCEAN BOTTOM

by

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ABSTRACT

This study is devoted to factors affecting the magnitude of force needed to cause complete withdrawal of objects embedded in sediments of the ocean bottom. Following a literature review, most discussions are centered around the basic problem of a centrally loaded object pulled by a vertical force from a sediment with level surface. Considerations of effects of eccentric and inclined loading, as well as of slope of the ocean bottom are added toward the end of the report.

It is found that failure patterns in the overburden soil, which greatly affect the magnitude of contribution of that soil to the breakout force, depend on the relative depth of the object, as well as on the type of soil and extent of its possible remolding by the operation of placing the object. In undisturbed dense and stiff soils at relatively shallow depths failure occurs in general shear, with a convex, torical slip surface. In the case of compressible, semi-liquid soils, as well as in the case of significant remolding around the object during placing, this convex surface degenerates into a vertical cylinder. At greater depths only punching failure, similar to that occurring under deep foundations, is observed, regardless of soil type.

A theoretical analysis, based on the assumption that the soil behaves at shallow depth as a rigid-plastic solid, shows better agreement in the case of soft and loose soils than in the case of stiff and dense soils. This is, however, only apparently a paradox, as analogous comparisons made with vertically loaded plates on soil surface show an intriguingly similar trend.

Perhaps the least understood components of the breakout force are those attributed to soil suction and to adhesion between the object and surrounding soil. It appears that the problem of soil suction could be treated as a problem of pore-pressure difference on the two sides of the object by pull-out loading.

It is suggested that the problem of breakout force may have a relatively simple solution for soils having water contents above the liquid limit, thus behaving as viscous liquids. There are reasons to believe that a similar approach, using a more complex rheological model, (such as Bingham's), would be worth trying for at least semi-liquid soils within the plastic range.

It is, finally, recommended that the future research on breakout forces be centered around the following four problem areas:

1) The effect of soil liquidity and/or compressibility on failure pattern in the overburden soil and magnitude of breakout factors.

2) The effect of time on development of adhesion between objects and the surrounding soil; the relationship between adhesion in tension and adhesion in shear.

3) The nature and magnitude of force of soil suction on objects; the effects of load inclination and eccentricity on magnitude of this force.

4) The nature and magnitude of breakout force in liquid soils; connection between composition and structure of such soils and their rheological constants of liquid and semi-liquid soils.

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Introduction

Increased interest in exploration and utilization of ocean resources over the past decade has resulted in special attention for the problem of breakout forces of objects embedded in sediments of the ocean bottom. This geotechnical problem is encountered in many marine operations in both shallow and deep waters, which require the use of anchors to transmit upward-directed forces to the ocean bottom. Typically, any mooring system for ocean surface or submerged platforms owes its stability to the ocean-bottom anchors (1). The problem appears equally often in design and construction of deep-sea habitats (2), as in design of salvage operations of sunken ships (3) or in repositioning of deep-sea platforms (4). In all these situations the problem of breakout forces can be defined as follows (Fig. 1):

Given an object of known shape, dimensions and weight, brought by some operation to rest at a depth D below the ocean bottom. The object is, thus, partially or fully embedded in the bottom sediments of known physical characteristics. Find the magnitude of force F needed to cause complete withdrawal of the object from the ocean bottom sediments.

The magnitude of force F depends, among other factors, upon its direction, its position with respect to the centroid of the object as well as upon the nature of connection between the force-transmitting element and the object. The force can be generally applied in a vertical direction (Fig. 2a, b) or at an angle α to the vertical (Fig. 2c,d); in both cases it can act centrally as shown in Fig. 2a,c or eccentrically, as shown in Fig. 2b, d. The connection with the force-transmitting element (usually a cable) is normally moment-free, as shown in Figs. 2a-d; if it transmits a moment as well (Fig. 2e) the problem is essentially the same as that of an eccentrically loaded object (Fig. 2b,d). The magnitude of force F may also be significantly affected by the slope of the ocean bottom, which will generally be denoted by β (positive as shown in Fig. 2f).

In this report we shall consider first the basic problem of a centrally loaded object pulled by a vertical force ($\alpha = 0$) against a horizontal bottom ($\beta = 0$), see Fig. 2a. Other cases will be considered, as appropriate, in subsequent sections. A brief literature review will precede the discussions.

Literature Review

Published literature on the subject of breakout forces from ocean bottom is relatively scarce. It consists principally of three reports describing the investigations performed during the last four years at the U.S. Naval Civil Engineering Laboratory in Port Hueneme, California (3) (4) (5), see also (6). These investigations involved breakout tests with objects weighing up to 40,000 lb., which were forced into soils and recovered at three different field locations, as well as in a large-scale laboratory facility. From results of tests in San Francisco Bay Muga (4) (6) proposed an empirical formula for evaluation of breakout force. He also presented a numerical analysis of the plane-strain problem of breakout of an object from an elastic-plastic solid. Assembling all results available from NCEL investigations, Liu (5) presented another empirical correlation in dimensionless terms, which took into account the time since embedment of the object, as well as the breakout

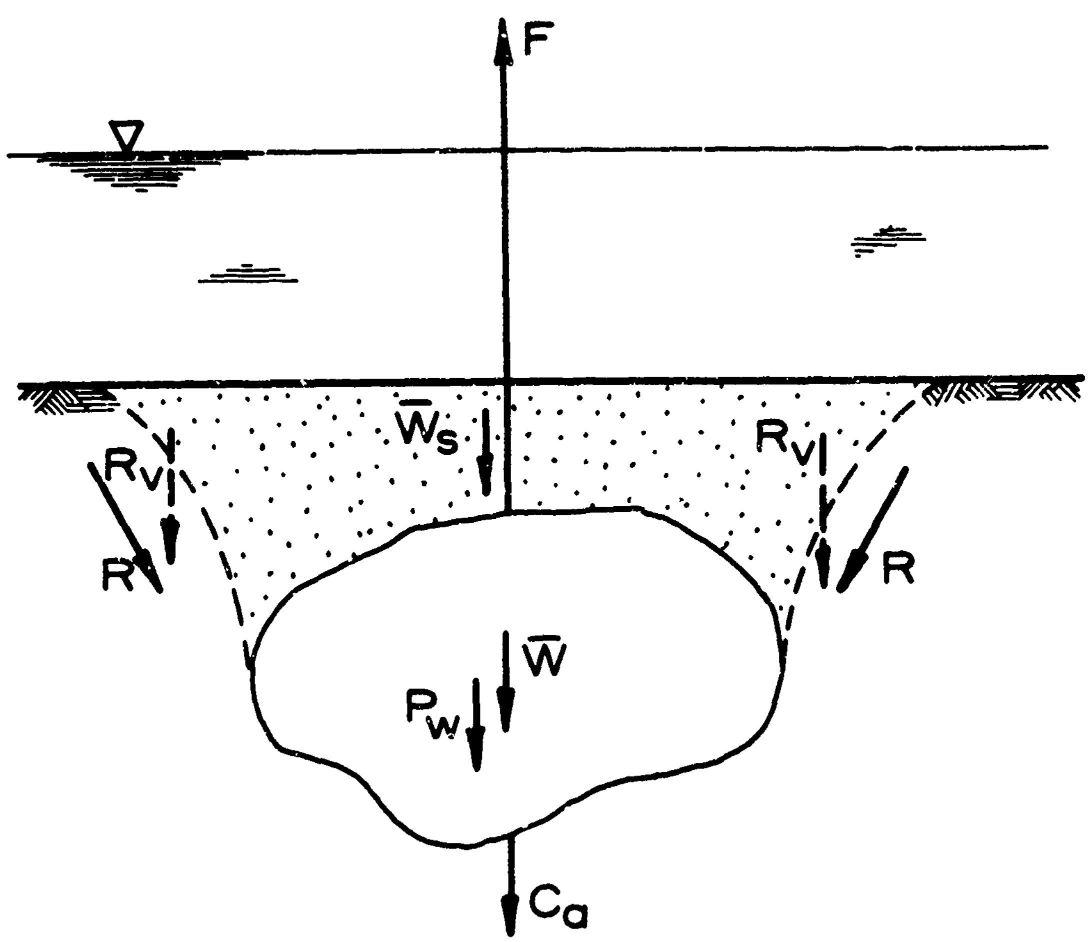

Fig. 1. Basic problem of breakout force

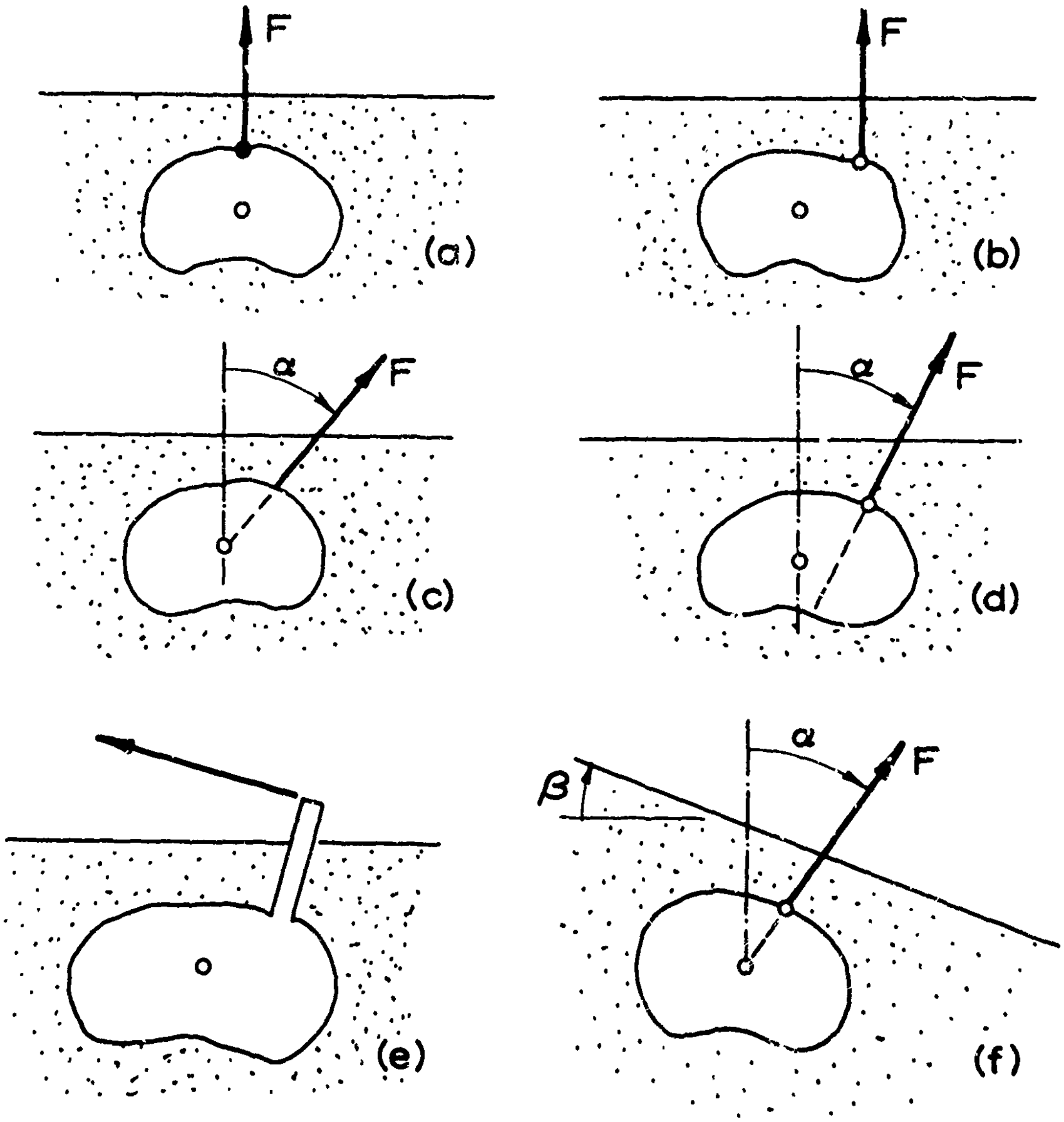

Fig. 2. Variants of the problem of breakout force

time, already present in Muga's formula. He also showed that it was very difficult to predict the breakout time to any reasonable accuracy.

Another known investigation was conducted at the Southwest Research Institute in San Antonio, Texas (7), where plates weighing up to 200 lb. were lifted from the surface of a soil model (sand or clay) inside a tank with simulated water pressures up to 3,000 psi. The principal finding of this preliminary investigation was that the breakout force depended upon the size of the object, soil type as well as embedment time. It was also confirmed experimentally, that hydrostatic pressures up to 3,000 psi did not have any effect on breakout force. The effects of breakout time or depth of embedment were not investigated.

In contrast to limited information available on breakout behavior of objects in ocean bottom environment, quite extensive investigations of pull-out capacity of anchors in ordinary terrestrial soils have been performed in the past. The work prior to 1960 consisted largely of testing of foundations for transmission towers and theorizing about the magnitude of soil resistance to pull-out (8, 9). Modern research on this subject started with a paper by Balla (10), who determined the shape of slip surfaces for shallow anchor plates in dense sand and proposed a rational method for analysis of pull-out forces based on observed shapes of these slip surfaces. Baker and Kondner (11) confirmed Balla's major findings regarding anchor plates in dense sand; however they showed that deep anchors behaved differently from shallow anchors. Mariupol'skii (12), who also noted the difference in behavior of anchors at greater depth, proposed separate analytical procedures for analysis of shallow and deep anchors. Sutherland (13) presented well documented results of pull-out tests with model plates up to 6-in. in diameter in loose and dense sand, as well as with 94-in.-diameter shafts in medium dense to dense sands. He found that the mode of failure varied also with sand density and showed that Balla's analytical approach may give reasonable results only in sands of some intermediate density. Kananyan (34) also presented well-documented results of pull-out tests with model plates up to 48 in. in diameter buried in a deposit of loose to medium dense fine sand. His experiments included one series of tests with inclined plates pulled out by central, inclined loads in the direction of the plate axis. He observed similar failure patterns as the preceding investigators, with well-defined slip surfaces. In the case of inclined plates, the pattern was unsymmetrical and the movement of the soil particles above the plate appeared to be predominantly vertical. The ultimate breakout forces generally increased with the inclination of the plates.

More recent research at Duke University involved testing of model plates and piles in loose and dense sand (14), as well as in a very soft clay (undrained strength of about 100 lb/ft^2), (15), and a stiff silty clay (undrained strength about 1,000 lb/ft^2), (16). Modes of failure were investigated in greater detail: it was found thattransition to deep anchor behavior occurred in very soft clay and loose sand at depths of only two to three plate diameters, as compared to five plate diameters in stiff clay and ten plate diameters in dense sand. These tests also revealed the relative importance of soil suction force in very soft clay, which proved to offer the predominant resistance to pull-out for all shallow anchors in soft clays. Some of these conclusions can be confirmed by analyzing the results of parallel investigations made at Hydro-Electric Power Commission on Ontario and Nova Scotia Technical College (17). The latter investigations contain also some data on pull-out capacity of model groups of piles, as well as on the effect of sustained loading on ultimate pull-out force in cohesive soils.

The Basic Problem

Returning to the basic problem of a centrally loaded object pulled by a vertical force against a horizontal bottom (Fig. 1), it is not difficult to find that the breakout force F consists of the following components:

(a) the effective weight of the object, $\bar{W}$, including the weight of the connecting cable;

(b) the effective weight of the mass of soil, W_s, involved in breakout together with the object;

(c) the vertical component R_v of the forces of shearing resistance R of the overburden soil along the slip surfaces separating that part of the soil involved in breakout from the rest of the soil mass;

(d) the vertical component C_A of forces of adhesion c_a between the skin of the object and adjacent soil;

(e) the soil suction force P_w, resulting from differences in pore-water stresses above and below the object, caused by attempted certical upward movement of the system.

The factors affecting the magnitude of individual components of the breakout force, listed above, are discussed and analyzed in the following paragraphs.

The Effective Weight of the Object and the Soil Mass

The determination of the effective weight $\bar{W}$ of the object poses no problems: it is always equal to the difference between the object's total weight W ("weight in air") and the buoyancy in water, U

$$\bar{W} = W - U \tag{1}$$

The effective weight of the involved soil mass can easily be determined if the effective unit weight γ' and the volume of that mass are known. For the effective unit weight γ' we can use the formula:

$$\gamma' = \frac{(G_s - 1)\gamma_w}{1 + e} \tag{2}$$

where G_s is the specific gravity of solids, γ_w unit weight of water and e the void ratio of the soil. For saturated soil the latter quantity is equal to

$$e = wG_s \tag{3}$$

where w is the water content of the soil.

Should there be steady vertical seepage of gradient i in the soil mass in question the apparent soil weight will be changed to

$$\gamma'' = \gamma' \pm \gamma_w i \tag{4}$$

where the plus sign applies to downward and the minus sign to upward flow of water.

To determine the volume of soil involved in breakout, it is essential to know the exact shape of the slip surface in the soil.

The shape of this surface has been the object of extensive speculation in the past, mostly in connection with analysis of footings of transmission towers subjected to vertical pull-out forces. On the basis of experiments on model anchor plates in dense sand, as well as from some theoretical considerations, Balla (10) suggested that the slip surface for circular buried objects should be part of a torus, with a generatrix consisting of a circle, such as that shown in Fig. 3. The circle should meet the soil surface at statically correct angle $\theta_o = 45 - \phi/2$ and the plate edge at kinematically correct angle $\theta_1 = 90^{\circ}$.

Observations in small-scale model tests with anchor plates and anchor piles at Duke University (14) (15) (16) proved that the shape shown in Fig. 3 occurs only in the case of relatively shallow anchors in dense sand or stiff silty clay. For shallow anchors in loose sand or soft clay, the slip surface, though not clearly established, is closer to being a vertical cylinder around the perimeter of the anchor.

Thus, for objects embedded in loose and compressible sediments, it is more reasonable to assume that the soil involved in breakout is essentially only soil immediately above the object. This may also prove to be a reasonable assumption in any case where the soil immediately surrounding the object is weakened by remolding. At the same time, the assumption of a torical slip surface (such as 01 in Fig. 3) will yield the maximum possible effective weight of the involved soil mass.

It may be added that the difference between the soil weight for an assumed torical slip surface, as compared with an assumed cylindrical slip surface, is small for relatively shallow and long objects (small D/B and large L/B). However the difference can be very significant for circular objects at greater depth.

It should be noted that very deep anchors do not fail in general shear failure such as that shown in Fig. 3, regardless of the relative density of the soil. Experiments indicate that they can be moved vertically for considerable distances by producing a failure pattern similar to punching shear failure in deep foundations (18). Only after being pulled up to relatively shallow depths may they eventually produce general shear failures such as that shown in Fig. 3. This is illustrated in Fig. 4, taken from Ref. 16.

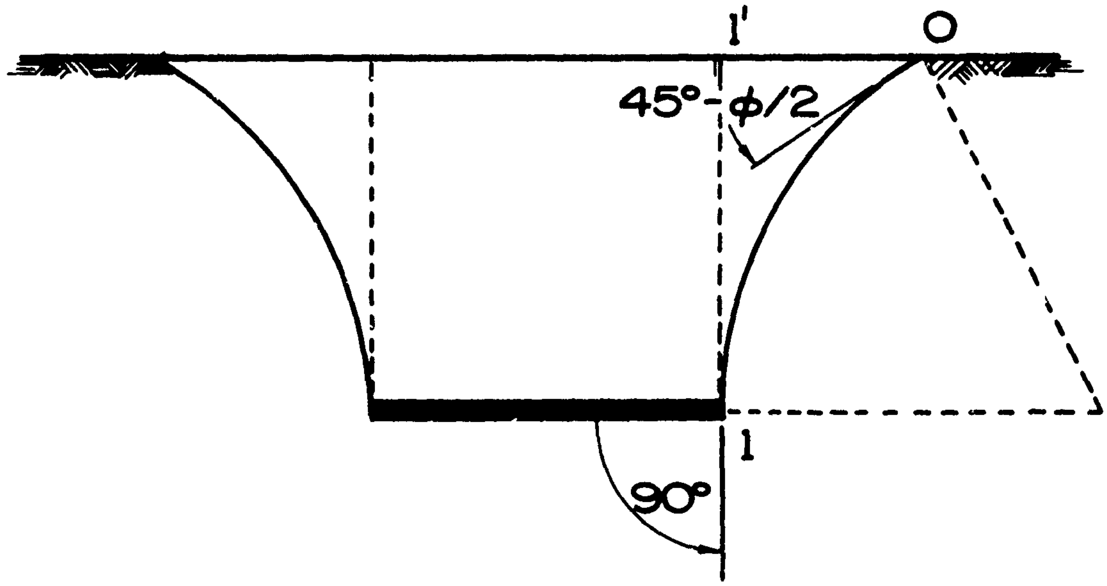

Fig. 3. Shape of slip surface for circular buried objects

The critical relative depth D/B above which embedded objects should behave as shallow anchors depends on the relative density of the soil and possibly some other yet unclarified factors. Available experimental evidence from experiments with 3-in.-diameter plates suggests that this limiting depth D/B in sand may increase from perhaps 2 for a very loose deposit to over 10 in a very dense deposit (14). In very soft bentonite clay the limit is at about D/B = 2, (15) while in a stiff silty clay it appears to be around D/B = 5 (16).

The Shearing Resistance of the Overburden Soil

In all cases where a definite slip surface such as that shown in Fig. 3 appears, the vertical component of the shearing resistance of the overburden soil, R_v, can be determined by an appropriate analysis. A rigorous computation by the methods of theory of plasticity is very difficult, unless some assumptions are made regarding the shape of the slip surface and the distribution of stresses along that surface.

Balla (10) has proposed a simplified analysis of this problem for a circular anchor plate, under the assumptions about the shape of the slip surface mentioned in the preceding paragraph and shown in Fig. 3. To find the distribution of stresses in the slip surface he applied Kötter's equation and assumed that the distribution in the axially symmetrical case is the same as in the plane strain case. The result of his computations is presented by an expression of the form:

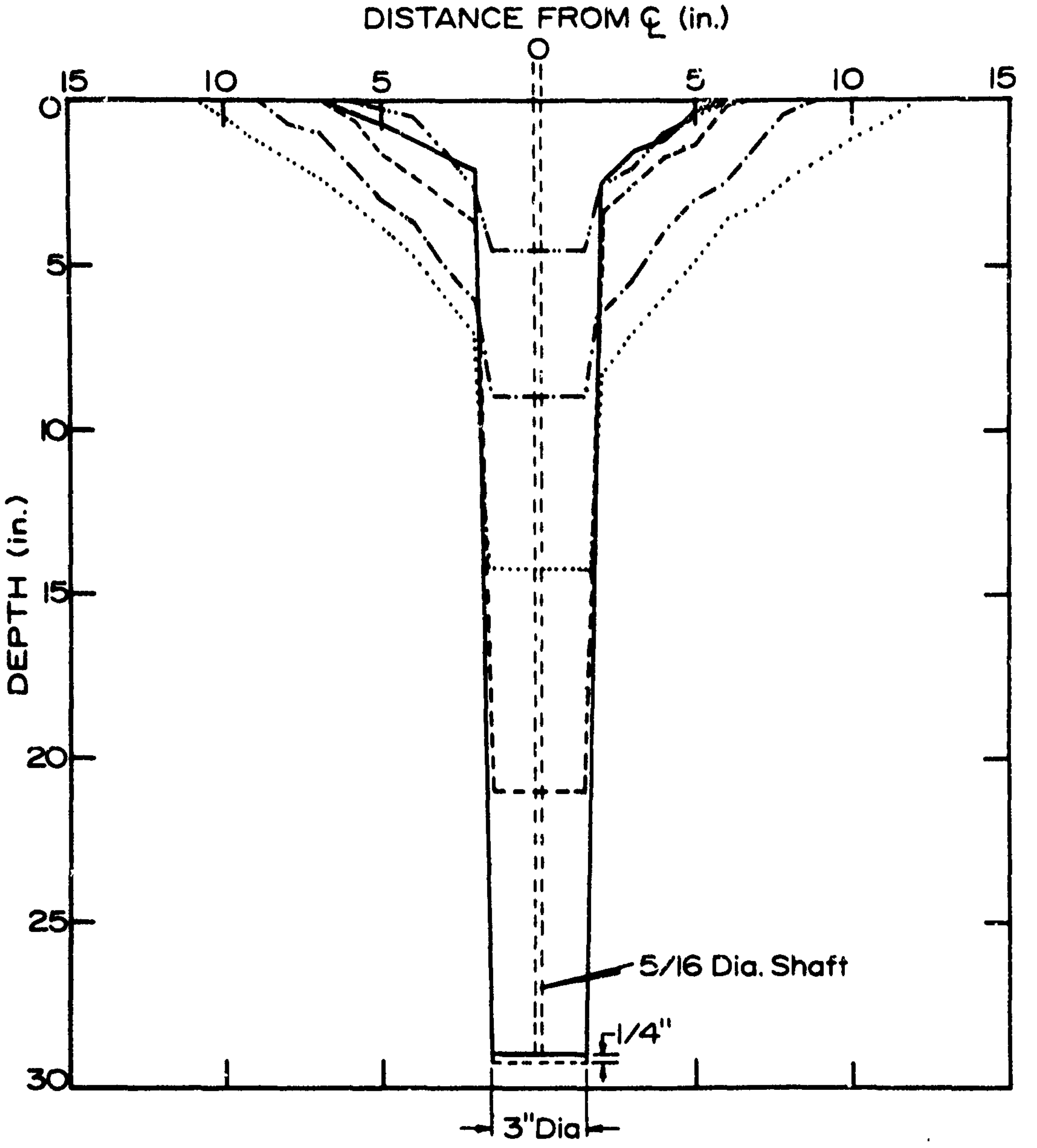

Fig. 4. - Observed shapes of slip surfaces caused by withdrawal of a circular plate from stiff silty clay (from Bhatnagar, 16)

$$R_v = F_2(\phi, \lambda)cD^2 + F_3(\phi, \lambda)\gamma D^3 \qquad (5)$$

In this expression c and ϕ are the strength characteristics of the soil (defining a Coulomb-Mohr failure criterion in a linear form), γ is the effective weight of the soil, and F_2 and F_3 are two complex functions of the angle of shearing resistance ϕ of the soil and of the relative depth $\lambda = D/B$ of the embedded circular anchor. Balla's paper (10) contains numerical values of factors F_2 and F_3, which, unfortunately, appear to be incorrect.

A different analytical approach to this problem is available from the solutions proposed by Vesić et al for the problem of expansion of cavities close to the surface of a semi-infinite rigid-plastic solid (19). These solutions give the ultimate radial pressure q_o needed to break out a cylindrical or a spherical cavity of radius R placed at a depth D below the surface of the solid. They are presented in the form

$$q_o = cF_c + \gamma DF_q \qquad (6)$$

where F_c and F_q are the cavity breakthrough factors, which depend on the shape and relative depth of the cavity, as well as on the angle of shearing resistance of the soil. As shown in Ref. 14, these solutions can be applied to the problem of anchor plates. They contain essentially the vertical component R_v of the soil resistance, plus the weight of the soil above the cavity, both reduced to the area of the plate. As such they could be used directly for embedded spheres or embedded horizontal cylinders. For embedded plates the equation (6) is corrected to

$$q_o = \frac{R_v + W_s}{\frac{\pi}{4}B^2} = c\bar{F}_c + \gamma D\bar{F}_q \qquad (7)$$

where $\bar{F}_c$ and $\bar{F}_q$ are plate breakout factors. It can be shown that, for any plate $\bar{F}_c = F_c$. However, for a circular plate:

$$\bar{F}_q = F_q + \frac{1}{3}\frac{B}{D} \qquad (8a)$$

while for a long rectangular plate:

$$\bar{F}'_q = F'_q + \frac{\pi}{8}\frac{B}{D} \qquad (8b)$$

For other shapes the difference in weight between the volume of the object protruding above its maximum width and the corresponding volume of overburden soil may be included, if significant. It should be emphasized that the expressions (6) or (7) include both R_v and W_s reduced to the maximum area of the embedded object, measured perpendicularly to tge applied breakout force F. The magnitude of factors $F_c = \bar{F}_c$, F_q and $\bar{F}_q$ for circular anchors and factors $F'_c = \bar{F}'_c$, F'_q and $\bar{F}'_q$ for long-rectangular anchors are given in the following Tables 1 and 2.

Table 1

HORIZONTAL CYLINDER OR

LONG RECTANGULAR PLATE

BREAKOUT FACTORS

(after Vesić & al, 1965)

φ \ D/B	0.5	1.0	1.5	2.5	5.0
0°	0.81 0.21 1.00	1.61 0.61 1.00	2.42 0.74 1.00	4.04 0.84 1.00	8.07 0.92 1.00
10°	0.84 0.30 1.09	1.68 0.77 1.16	2.52 0.99 1.25	4.22 1.26 1.42	8.43 1.75 1.83
20°	0.84 0.38 1.17	1.67 0.94 1.33	2.52 1.23 1.49	4.19 1.67 1.83	8.37 2.57 2.65
30°	0.79 0.45 1.24	1.58 1.03 1.47	2.37 1.45 1.71	3.99 2.03 2.19	7.89 3.30 3.38
40°	0.70 0.51 1.30	1.40 1.19 1.58	2.11 1.61 1.87	3.51 2.30 2.46	7.02 3.83 3.91
50°	0.58 0.53 1.32	1.17 1.25 2.04	1.75 1.70 1.96	2.92 2.44 2.60	5.84 4.12 4.20

First number $F'_c = \bar{F}'_c$ Second number F'_q (cylinder)

Third number $\bar{F}'_q$ (long rectangular plate)

Table 2

SPHERE OR CIRCULAR PLATE BREAKOUT FACTORS

(after Vesić & al, 1965)

ϕ \ D/B	0.5	1.0	1.5	2.5	5.0
0°	1.76 0.33 1.00	3.80 0.67 1.00	6.12 0.78 1.00	11.6 0.87 1.00	30.3 0.93 1.00
10°	1.87 0.51 1.18	5.10 1.04 1.37	6.69 1.37 1.59	13.0 1.95 2.08	36.0 3.60 3.67
20°	1.90 0.69 1.36	4.23 1.42 1.75	7.01 1.98 2.20	13.9 3.12 3.25	38.9 6.64 6.71
30°	1.84 0.85 1.52	4.19 1.78 2.11	7.06 2.57 2.79	14.3 4.28 4.41	41.6 9.82 9.89
40°	1.69 0.98 1.65	3.95 2.08 2.41	6.79 3.08 3.30	14.2 5.32 5.45	42.7 12.9 13.0
50°	1.47 1.06 1.73	3.53 2.28 2.61	6.19 3.34 3.56	13.3 6.14 6.27	41.6 15.6 15.7

First number F_c (sphere or circular plate)

Second number F_q (sphere)

Third number $\overline{F}_q$ (circular plate)

A somewhat different approach to the problem of soil resistance in breakout has been attempted by Muga (4, 6). He has developed a numerical procedurre, based on the discrete-element model introduced by Harper and Ang (31), for analytical determination of the breakout force. The soil in this analysis is assumed to behave as a homogeneous elastic-perfectly plastic solid, following the Huber-Mises yield criterion in the plastic state. A good agreement between the results of this analysis and the experimental data from the San Francisco Bay was reported, at least for the soil in question - a highly plastic clay. In view of the yield criterion used it should not be expected that the analytical method used could be applied to other types of soil. An adaptation of the same procedure to other yield criteria, in particular to Coulomb-Mohr's is, in principle, possible, and should be attempted.

Another analytical approach to the same problem can be found in the mentioned paper by Mariupol'skii (12). He determined the shape of the slip surface and the state of stress in the wedge of soil above the shallow anchor by using the following two assumptions: (1) that the maximum shear stress is mobilized in every vertical cylindrical surface (such as 11' in Fig. 3) around the anchor axis; (2) that failure occurs in tension at different points along a line such as 01 in Fig. 3 whenever the vertical shear force exceeds the shearing strength along the vertical cylindrical surface over which it is to be transmitted.

Mariupol'skii's solution can be written in the following form:

$$F = \overline{W} + \frac{\pi}{4}(B^2 - B_o^2)\ \frac{1 - (B_o/B)^2 + 2\,K \tan\phi\ D/B + 4c\ D/B}{1 - (B_o/B)^2 - 2n\ D/B}\ \gamma D \qquad (9)$$

where F is, as before, the breakout force, $\overline{W}$ effective weight of the anchor, D the depth and B the diameter of the circular anchor plate, B_o the diameter of the anchor shaft, γ the effective weight of the soil above the anchor, c and strength characteristics of the soil, K the coefficient of lateral earth pressure in the soil wedge above the anchor and $n = 0.025\,\phi^o$ an empirical function of the angle of shearing resistance ϕ of the soil ($\phi^o = \phi$ in degrees).

Since the values of the parameter n were determined from author's experiments, the reported agreement of theory and experiments is of very limited meaning. It should be noted that the assumptions made in analyzing the state of stress in the soil wedge above the anchor are entirely arbitrary and in contradiction with the elementary theory of earth pressure.

Mariupol'skii has also presented a solution for the soil resistance to pullout of a deep anchor. This solution is based on the assumption that the work done by the anchor during vertical displacement should be equal to the work needed to expand a vertical cylindrical cavity of radius $R_o = B_o/2$ to the radius $R = B/2$. Using the same notations as above, the ultimate breakout force F is given by:

$$F = \overline{W} + \pi B_o \left[D - B + B_o\right] f_o + \frac{\pi/4(B^2 - B_o^2)}{1 - 0.5 \tan\phi}\, p_u \tag{10}$$

where f_o is the unit skin resistance along the stem of the anchor (of radius B_o) and p_u is the ultimate pressure for expansion of a deep cylindrical cavity. Mariupol'skii determines thie ultimate pressure by trial and error from a lengthy equation. However, this could be done more conveniently by using a rigorous solution of this problem given by Vesić (32, 33) in the following form:

$$p_u = cF_c' + \gamma DF_q' \tag{11}$$

Here F_c' and F_q' represent cylindrical cavity expansion factors:

$$F_q' = (1 + \sin\phi)\left(\frac{I_r}{2\cos\phi}\right)^{\frac{\sin\phi}{1+\sin\phi}} \tag{12}$$

$$F_c' = (F_q' - 1)\cot\phi \tag{13}$$

The quantity I_r represents the rigidity index of the soil, defined in terms of strength characteristics c, ϕ and deformation characteristics E, ν of the soil as:

$$I_r = \frac{E}{(1+\nu)(c + \gamma D \tan\phi)} \tag{14}$$

It should be noted that for $\phi = 0$

$$F_q' = 1 \tag{15}$$

$$F_c' = \ln\left(\frac{I_r}{2}\right) + 1 \tag{16}$$

Numerical values of factors F_c' and F_q' for different values of ϕ and I_r are given in Table 3.

The derived solution is based on assumption of no volume change in the plastic zone surrounding the cavity. To introduce the effect of volume change occurring in that zone as well, it is necessary to evaluate its average volumetric strain, Δ. A relatively simple procedure of doing this is outlined in Ref. 19. It is shown in the same reference that, once Δ is known, the same equations (12) and (13) resp. (15) and (16), can be used however with a reduced rigidity index I'_{rr} defined as:

$$I'_{rr} = \frac{I_r}{1 + \frac{1}{2}\frac{I_r \Delta}{\cos\phi}} = \zeta_v' I_r \qquad (17)$$

where ζ_v' is the volume change factor for a cylindrical cavity. Numerical values of ζ'_v have been computed and assumbled in Table 4.

Table 3
CYLINDRICAL CAVITY EXPANSION FACTORS
(after Vesić, 1963)

ϕ \ I_r	10	20	50	100	200	500	1,000
0°	2.6 1.0	3.3 1.0	4.2 1.0	4.9 1.0	5.6 1.0	6.5 1.0	7.2 1.0
5°	2.7 1.2	3.5 1.3	4.7 1.4	5.6 1.5	6.5 1.6	7.9 1.7	9.0 1.8
10°	2.8 1.5	3.7 1.7	5.1 1.9	6.2 2.1	7.6 2.3	9.5 2.7	11.1 2.9
15°	2.9 1.8	3.8 2.0	5.4 2.5	6.8 2.8	8.5 3.3	11.0 4.0	13.3 4.6
20°	2.9 2.1	4.0 2.4	5.8 3.1	7.4 3.7	9.4 4.4	12.6 5.6	15.5 6.6
25°	2.9 2.4	4.1 2.9	6.0 3.8	7.9 4.7	10.2 5.7	14.0 7.5	17.7 9.3
30°	2.9 2.7	4.1 3.4	6.2 4.6	8.3 5.8	10.9 7.3	15.4 9.9	19.9 12.5
35°	2.9 3.0	4.2 3.9	6.4 5.5	8.6 7.1	11.5 9.1	16.8 12.7	22.0 16.5
40°	2.9 3.4	4.2 4.5	6.5 6.4	8.8 8.4	12.0 11.1	17.7 15.8	23.7 20.9
45°	2.9 3.8	4.1 5.1	6.5 7.5	9.0 10.0	12.3 13.3	18.3 19.3	24.9 25.9
50°	2.8 4.3	4.0 5.8	6.4 8.6	9.0 11.7	12.4 15.8	18.9 23.5	25.8 31.8

Upper Number F'_c Lower Number F'_q

Table 4
VOLUME CHANGE FACTORS ζ_v' FOR A CYLINDRICAL CAVITY
(after Vesić et al 1965)

I_r	ϕ \ Δ	0.01	0.02	0.03	0.04	0.05	0.10	0.15
5	0	0.975	0.953	0.930	0.909	0.889	0.800	0.728
	15	0.975	0.951	0.928	0.906	0.885	0.795	0.717
	30	0.973	0.945	0.920	0.896	0.874	0.776	0.698
	45	0.965	0.935	0.905	0.875	0.850	0.738	0.653
10	0	0.953	0.909	0.870	0.833	0.800	0.667	0.572
	15	0.951	0.906	0.866	0.829	0.795	0.659	0.563
	30	0.945	0.896	0.852	0.812	0.776	0.635	0.536
	45	0.928	0.875	0.825	0.780	0.739	0.586	0.485
20	0	0.909	0.833	0.769	0.714	0.667	0.500	0.400
	15	0.906	0.829	0.762	0.706	0.659	0.491	0.391
	30	0.896	0.812	0.743	0.683	0.634	0.464	0.366
	45	0.875	0.779	0.702	0.639	0.586	0.414	0.320
50	0	0.800	0.667	0.572	0.500	0.445	0.286	0.211
	15	0.795	0.659	0.563	0.492	0.431	0.279	0.205
	30	0.776	0.635	0.536	0.465	0.409	0.257	0.187
	45	0.739	0.586	0.484	0.414	0.361	0.221	0.158
100	0	0.667	0.500	0.400	0.333	0.286	0.167	0.118
	15	0.659	0.491	0.392	0.326	0.278	0.162	0.114
	30	0.634	0.465	0.366	0.302	0.258	0.148	0.104
	45	0.585	0.414	0.320	0.261	0.211	0.124	0.086
200	0	0.500	0.333	0.250	0.200	0.167	0.091	0.063
	15	0.493	0.326	0.242	0.194	0.162	0.088	0.060
	30	0.465	0.302	0.224	0.178	0.148	0.080	0.055
	45	0.414	0.262	0.191	0.150	0.124	0.066	0.045
500	0	0.286	0.167	0.118	0.091	0.074	0.038	0.026
	15	0.278	0.162	0.114	0.088	0.072	0.036	0.025
	30	0.257	0.147	0.103	0.080	0.065	0.033	0.023
	45	0.221	0.124	0.086	0.066	0.054	0.027	0.019
1000	0	0.167	0.091	0.063	0.048	0.038	0.020	0.013
	15	0.162	0.088	0.061	0.046	0.037	0.019	0.012
	30	0.148	0.080	0.055	0.042	0.034	0.017	0.011
	45	0.124	0.066	0.045	0.034	0.028	0.014	0.009

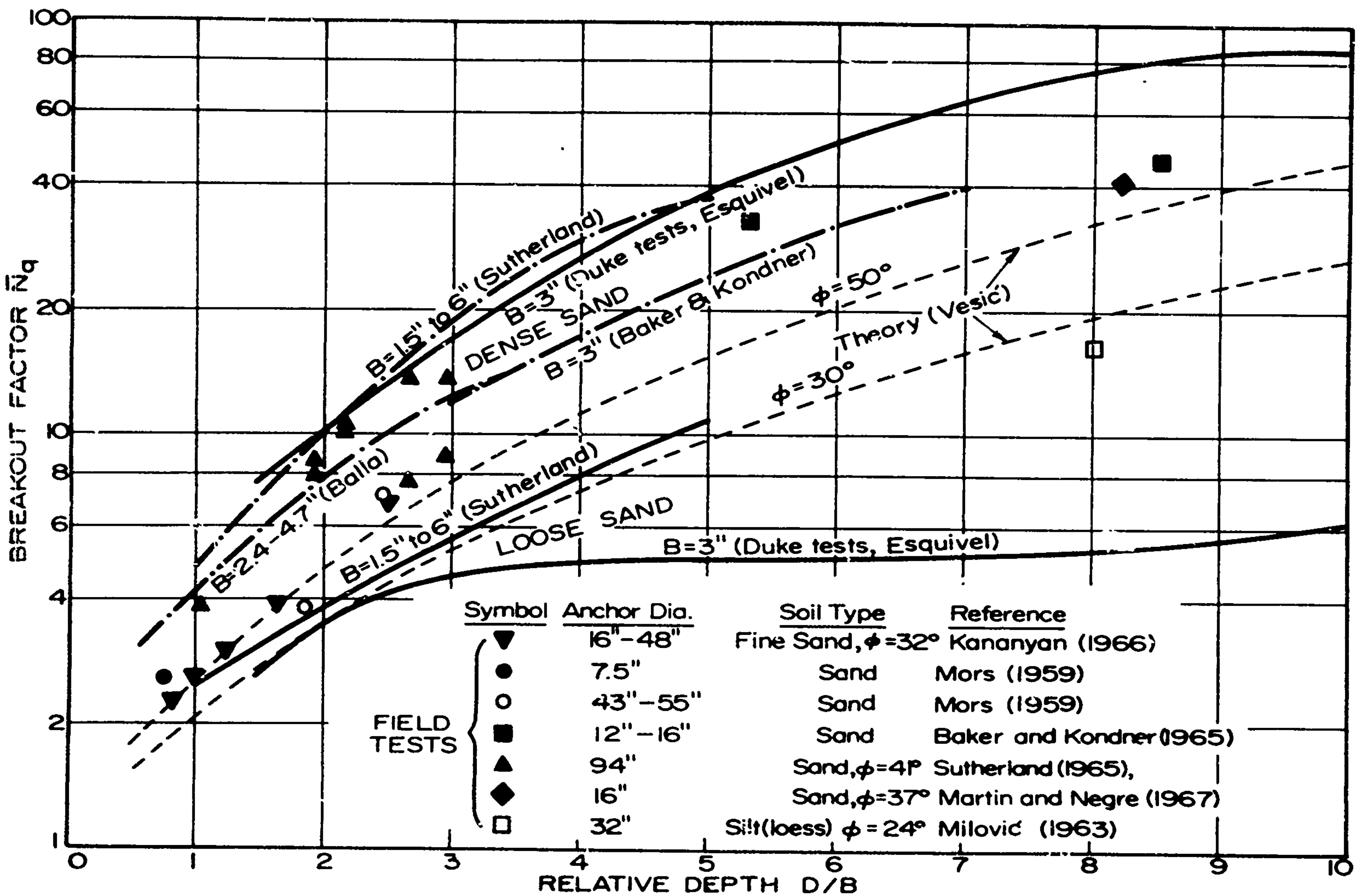

Fig. 5. - Breakout factor N_q in sands and silt

Comparison of Theory and Experimental Data

Comparisons of observed shearing resistances of the overburden soil with the magnitudes computed by the Vesić solution, as given in the preceding paragraph, are presented in Figs. 5 and 6. Both figures show the expected trend of increase of observed breakout factors with depth only at shallow depths. For each soil type there is a characteristic relative depth D/B beyond which the anchor plate starts behaving as a deep anchor and beyond which breakout factors reach constant, final values. As mentioned earlier this characteristic relative depth for sands increases with relative density from about 3 for loose sands to over 10 for dense sands (Fig. 5). For clays it also appears to increase from about 2 for very soft clays to about 5 for stiff clays (Fig. 6).

Though the observations confirm the expected trend of increase of breakout factors with depth, they also show that the absolute magnitude of observed factors does not generally agree with theory. The difference

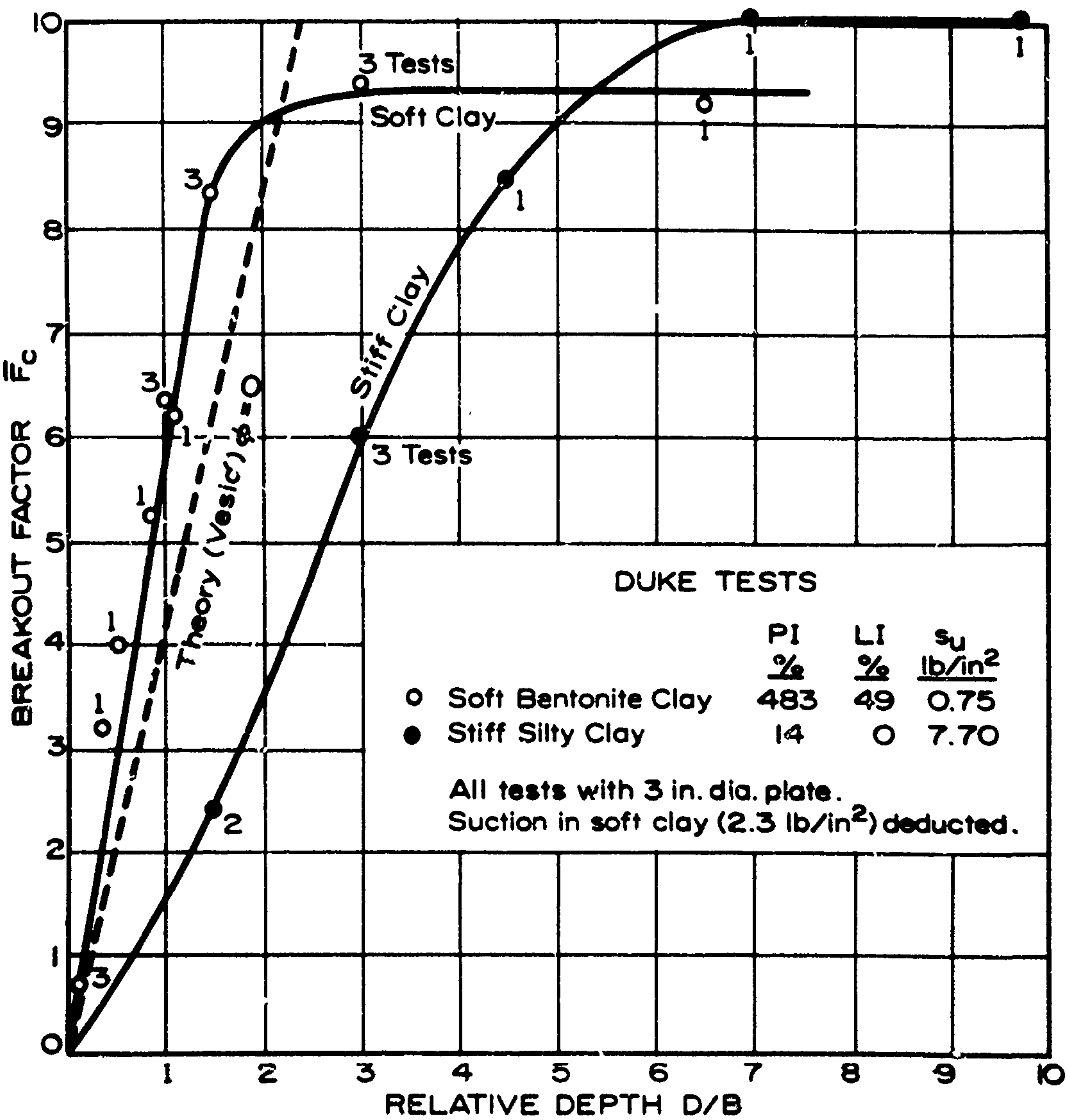

Fig. 6. Breakout factor F_c in clays

is the most pronounced in dense sands, where the observed factors are over 100% higher than the theoretical ones and in stiff clays where the observed factors are as low as 40% of those predicted by the theory.

While the causes of this disagreement are yet to be sought, it is interesting to note that the differences in sand are of the same nature and magnitude as those observed in loading tests with plates on sand surface (18).

It is also interesting to note that the breakout factors for deep anchors are practically equal to corresponding point bearing capacity factors of deep foundations. In particular, it is found that for deep anchors in clay $\bar{F}_c \approx 9$ to 10 while $\bar{F}_q$ increases from about 6 in loose sand to about 90 in dense sand. These values are comparable to those reported for deep foundations in clay (20) and sand (18).

The Effect of Soil Remolding

In applying the major findings of investigations described in this study to the problem of breakout of objects from the ocean bottom one must keep in mind the possible effects of soil remolding. It is well known that most cohesive soils lose a portion of their strength on remolding. Left to rest after that, they regain part or all of that strength by a regeneration process known as thixotropic regain. Computations of breakout forces should be based on estimated soil strength at the time of attempted breakout. This strength will generally be different from undisturbed strength determined by appropriate in-situ or laboratory tests. As mentioned earlier, the soil zones remolded by the operation of placing the object inside the mass of the ocean bottom soil are zones of weakness, which may considerably alter the shape of slip surfaces in the soil during breakout. This has often been noticed in the case of anchor footings for transmission lines, which are usually constructed in bored or excavated holes and backfilled. The soil immediately surrounding these footings is almost always weaker, causing the slip surfaces to develop as vertical cylinders around the perimeter of the footing (21).

Effects of Rate and Character of Loading

Analyses of breakout forces, such as those presented in preceding paragraphs, are based on presumed knowledge of shear strength of ocean bottom sediments, which is normally to a certain degree strain-rate dependent and which also varies with the character of loading (single, repeated or pulsating). The effects of rate and character of loading on strength of soils have been studied rather extensively over the past twenty years and are, at least qualitatively, well understood. A very good review of the current knowledge on this subject is summarized as follows (22):

1) For a single load application there is little variation in effective strength of saturated sands as the rate of loading varies from very fast (say 10 milliseconds to failure) to very slow (say 10 hours to failure). However under very fast loads there may be apparent strength increase due to negative pore-water stresses, if the sand density is above critical as well as apparent strength decrease due to positive pore-water stresses, if the sand density is below critical, (23).

2) Under very fast, repeated loading, almost total loss of apparent shear strength may occur in sands, even at densities far above critical (24). This phenomenon, attributed to build-up of pore-water pressures, had caused a number of catastrophic failures during earthquakes (25) and is, in all likelihood, responsible for flow slides in submarine canyons.

3) For a single load application there is in cohesive soils a strength increase under very fast loads and a strength decrease under very slowly applied loads. This phenomenon has been attributed to viscous effects in adsorbed water surrounding active soil particles. The strength typically increases twofold if the time to failure is reduced from one hour to 5 milliseconds, and may be reduced to as little as 50% of the one-hour value, if the load is sustained over several months. For obvious reasons, this effect should increase with increased clay content and increased activity of soil minerals and must become more and more pronounced as the liquidity index of soil increases toward 1 (the latter effects will still be discussed in subsequent paragraphs).

4) Repeated loading causes some loss of strength of cohesive soils as well, though the loss is rarely as spectacular as that in cohesionless soils (26). This effect increases considerably with the amount of stress increments applied, as well as with the soil sensitivity to remolding.

5) Vibratory loads cuase generally loss of strength of both cohesionless and cohesive soils, though the loss is much more pronounced in the former. The amplitude and frequency of vibrations as well as characteristics of the footing in contact with the soil affect strongly this phenomenon (27). Very little is known, however, in quantitative sense about the response of footings to vibratory loads in pullout.

Contribution of Soil Adhesion

Cohesive soils, containing active minerals, will develop adhesion in contact with almost any material. The process is of physico-chemical nature and requires some time.

Experiences with steel, concrete and wood piles seem to indicate that, at least in soft soils, the adhesion equals or exceeds the undrained shear strength after a period of a few days to, perhaps, six months. Little is known about the development of adhesion within the first hours or days after the objects have been brought into contact with soil.

It should be noted that development of adhesion is parallel to the process of regeneration of shear strength of soils. Comparative studies of development of both with time for at least some soil types and object materials would be highly desirable.

Most of known adhesion studies were concerned with measurements of resistance to shear. However in the breakout problem we have to deal also with resistance to tension between the buried object and the underlying soil. Very little, if anything, is known about such a force, except that is exists.

Contribution of Soil Suction Force

Penetration of an object through ocean bottom soil before coming to rest, causes some excess total stresses underneath, which may be taken mostly by excess pore-water stresses. If the object has been resting at the bottom for a sufficiently long time, at least a portion of pore-water stresses may have been dissipated.

On application of breakout force the overburden soil immediately above the object is heavily compressed, while the underlying soil is relieved from stress. Unless the soil is so highly pervious as to respond immediately to stress changes, there will be increase of pore-water stresses above the object and decrease of pore-water stresses below the object. The difference results in a suction force.

Very little is known about this force in any general sense. The measurements in Duke tests (15) with 3-in.-ciameter plate anchors indicated an average suction pressure of 2.8 psi. This pressure is significantly higher than the measured adhesion of 0.5 psi between this soil and the anchor plate. (It should be noted that the indicated value was measured with plate on soil surface and that the suction pressure at some depth might be still higher.)

A possible way of analyzing the suction pressure is suggested in Fig. 7. If the initial stress conditions σ_i, u_i above and below the object, as well as total stress increments $\Delta\sigma$ imposed by object withdrawal are known, the pore pressure increments Δu can be determined by appropriate tests on undisturbed soil samples. Recent research on yield behavior of soils at Duke (28) offers also the possibility of pre-determining these pore pressure increments analytically if the basic strength characteristics c', ϕ' of the soil are known. In either case the difference in pore-pressure increments on the two sides of the objects represents the maximum possible suction pressure, which would occur whenever the rate of load application is must faster than the rate of dissipation of pore-water stresses. To find the breakout time in the situation where a known sustained load is applied, one could develop, in principle, the needed solutions by using the three-dimensional theory of consolidation.

It should be observed that the solutions of this kind could be used only as long as the liquidity index of the soil is low enough that no significant flow of soil itself occurs toward the potential cavity formed under the object. Possible approaches for liquid soils, which, for obvious reasons, must be basically different, will be discussed a little later in the report.

Effect of Ocean Bottom Slope

The preceding considerations were limited to the basic case of breakout from horizontal ocean bottom. Should the ocean bottom be sloped at an angle β to the horizontal, the weight and resistance of the soil mass opposing breakout will be different. As in the preceding case, an analogy can be found with the problem of expansion of a cavity close to the surface of a sloped terrain (Fig. 8). This problem has been solved recently at Duke, in connection with studies of the cratering problem (29). The solution indicates that the ultimate breakout pressure of shallow objects can be found by using the same equations and factors as in the case of an horizontal bottom. It is necessary, however, to replace the depth D in Eqs. 6 or 7 by the distance of least resistance D_o, and the effective weight γ of the soil by its component $\gamma \cos \beta$ in the direction perpendicular to the slope. As the shearing resistance of the soil vanishes when the slope angle β becomes equal to ϕ, a correction factor such as $\sin (\phi - \beta)/\sin \phi$ should be used for that resistance. Regarding

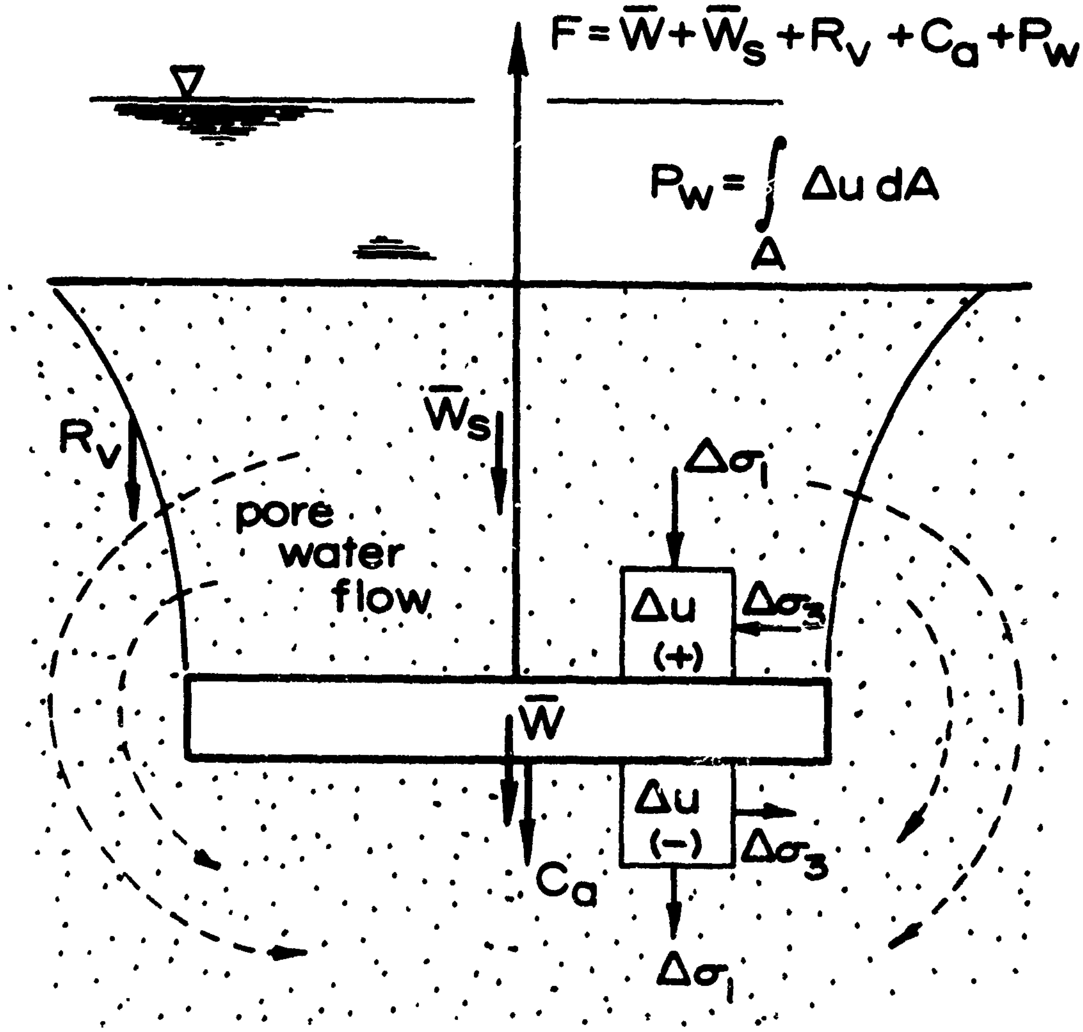

Fig. 7. Analysis of suction force

the breakout pressures of deep objects under a slope the available information suggests that they should be no different from those under a horizontal ocean bottom.

Effect of Load Inclination

In the case of central loads acting at an angle different from 90° to the ocean bottom, the failure pattern in the soil mass is changed, becoming unsymmetrical with respect to the line of least resistance (Fig. 9). For shallow objects the analysis can be performed by assuming a realistic shape of slip surfaces, such as that shown in Fig. 9, and using the conventional earth pressure theory, see Ref. 30. It is significant to note that the only known experimental investigation of the effect of load inclination, made with plates in fine sand (34), showed an increase of breakout force with inclination of the plate. As the incident angle

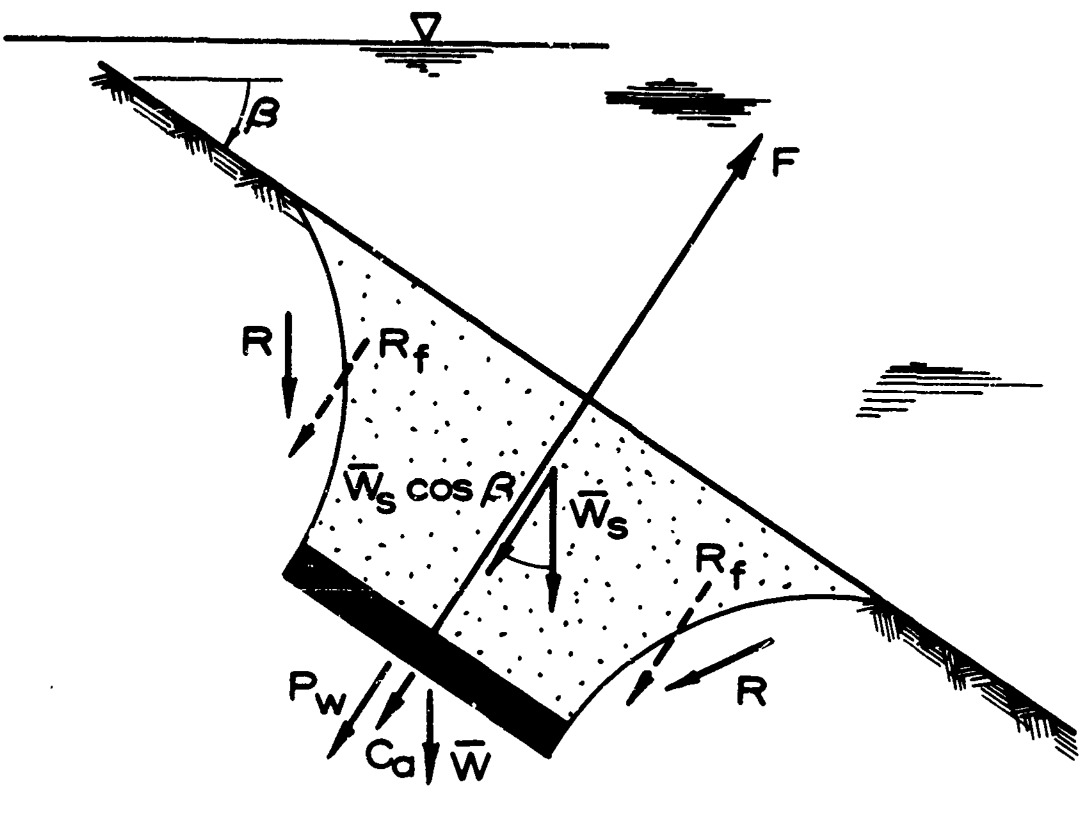

Fig. 8. Analysis of breakout of an object on sloped ocean bottom

α (Fig. 2c) increased from 0 to 45° with all other variables remaining the same, the breakout force was practically doubled. The object in question - a circular steel plate - was placed perpendicularly to the applied load.

It should be of great interest to investigate also the breakout phenomena with inclined loads acting at angles different from 90° to the major plane of the object. In such cases one should expect the object to rotate prior to pullout - as long as the load connection allows such rotation. This may change the failure pattern in the soil and also cause significant difference in the soil suction force. Analyses of this kind, nonexistent at present, should be of great practical interest to the ocean technologist.

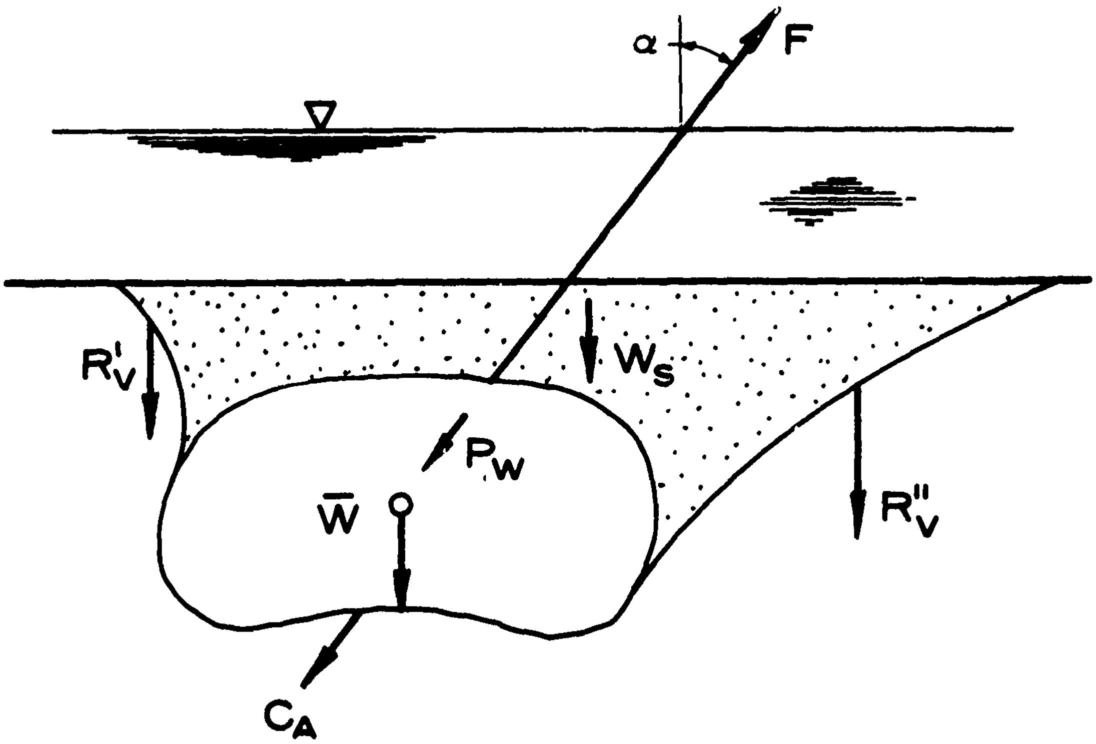

Fig. 9. Analysis of breakout by an inclined load

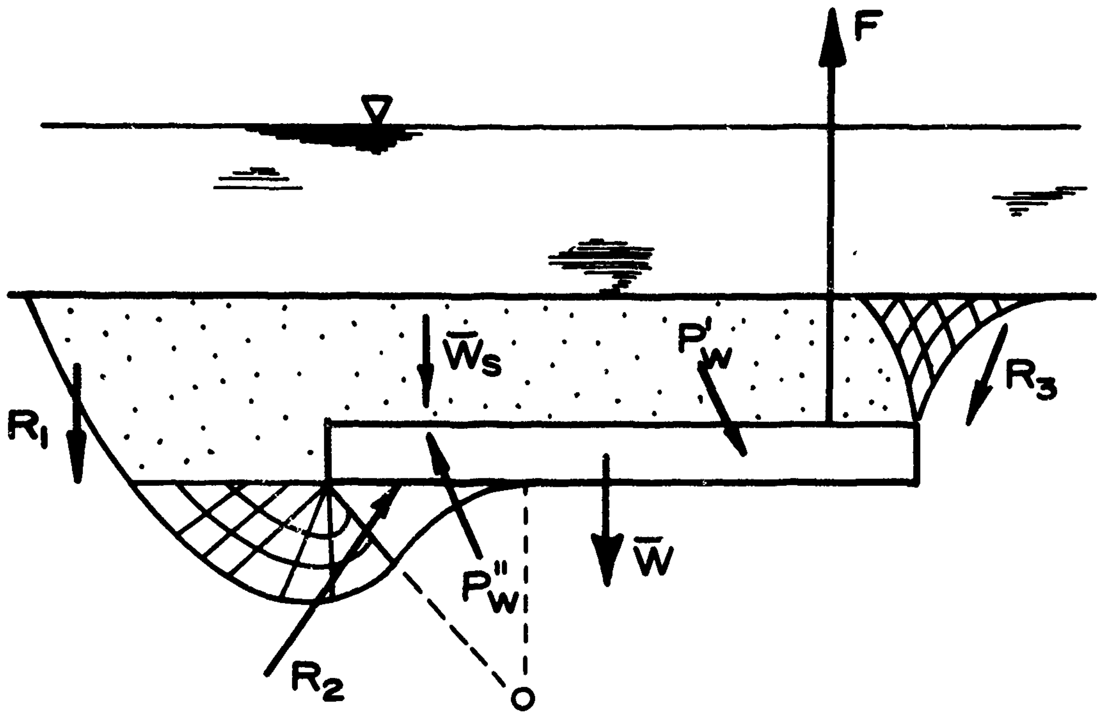

Fig. 10. Analysis of breakout by an eccentric load

Effect of Load Eccentricity

In the case of eccentrically applied loads, the rotation of the object prior to pullout must be very significant, resulting in a failure pattern quite different from those occurring under central loading. A possible failure pattern is shown in Fig. 10. The center of rotation is located below, causing plastic failure on both sides of the object. Suction forces P_w acting are forming a couple, thus balancing each other, at least in part. This may explain why it is often so much easier to pull-out an object from ocean bottom by pivoting. Analyses such as that shown in Fig. 10 can be very helpful in determining the most favorable position for application of the breakout force in a salvage operation. In view of complete absence of experimental information on this subject, the analyses should be accompanied by small-scale tests.

Effect of Soil Liquidity

As mentioned earlier, all the preceding analyses are, in principle, applicable to the computation of breakout force in media which possess some finite shearing strength. These include, theoretically, all cohesionless soils, such as sand, as well as cohesive soils, such as clay, at water contents below the liquid limit. However, soils at water contents above the liquid limit have practically no residual shearing strength, at least when remolded or sheared at large strains. For such soils a fundamentally different approach to analysis of breakout force should be attempted: they should be treated as viscous fluids of appropriate rheological characteristics.

The implementation of this approach to solution of the breakout force problem would require extensive basic studies of rheological behavior of soil pastes. Such an approach should in all probability be simpler than the approach for plastic soils outlined in the preceding paragraphs. Its main advantage may lie in the direct way in which the effects of time on breakout force can be introduced.

Should this rheological approach indeed prove as promising as it appears, an attempt should be made to investigate also the possibility of extending the range of its application to plastic soils of sufficiently high liquidity index. The difficulties in using a more complex rheological model with this purpose in mind may well be compensated by advantages of a unified approach for all cohesive sediments.

Summary and Appraisal

The discussions presented in this study reveal a very complex nature of phenomena involved in breakout of objects embedded in ocean bottom. It should be obvious that no formula, no matter how elaborate, could be fully satisfactory for all variety of soil conditions as well as of methods of placement and types of objects to be pulled-out.

The empirical formulae, such as those proposed by Muga (4, 6) and Liu (5) have the advantage of simplicity, associated with inclusion of only a few selected parameters that affect the magnitude of the breakout force. It is significant that they include explicitly the breakout time or the time since the object has been placed as factors affecting the breakout force. However, the realm of their application remains limited to a particular soil type and a particular set of placement and pull-out conditions.

The numerical approach used by Muga (4), applying discrete-element techniques and high speed computer calculation could have a potentially broader application if it were designed with more realistic stress-strain-time characteristics of surrounding soil, and better defined boundary conditions at the soil-object interface. An extension to the axially-symmetrical case would also greatly improve the range of its possible application. In its present form the use of this numerical approach remains restricted to pull-out of shallow long objects from deposits of soft clay.

The earth-pressure theory or soil mechanics approach, which uses the solutions for expansion of spherical and cylindrical cavities, has, as outlined in this report, a potentially broad realm of application. This approach is, in principle, recommended as the best available at present for prediction of breakout forces. Examples of use of this approach are enclosed in the Appendix. It should be emphasized that the selection and determination of soil strength and deformation parameters for analysis must be associated with high-level engineering judgment. A reasonable degree of expertise in soil mechanics and geotechnical engineering should be a prerequisite for successful application of this approach. The chief disadvantage of this kind of analysis remains in the fact that the time effects on breakout are introduced only indirectly through strength and deformation parameters of the soil.

In view of the semi-liquid and liquid consistency of many ocean bottom sediments, a rheological approach to the breakout force problem appears to offer a very promising way for direct introduction of time effects into analysis. Considering the complex soil and boundary conditions of the problem, this approach may be implemented with use of numerical, finite element analysis and a visco-plastic idealization of soil behavior.

Recommendations

The analysis of individual components of the breakout force reveals a number of problem areas that should be given detailed attention in future research. Foremost among these are probably the following four problem areas:

1) The effect of soil liquidity and/or compressibility on failure pattern in the overburden soil and magnitude of breakout factors.

2) The effect of time on development of adhesion between objects and the surrounding soil; the relationship between adhesion in tension and adhesion in shear.

3) The nature and magnitude of force of "soil suction" on objects; the effects of load inclination and eccentricity on magnitude of this force.

4) The nature and magnitude of breakout force in liquid soils; connection between composition and structure of such soils and their rheological properties; development of in-situ methods for measurement of rheological constants of liquid and semi-liquid soils.

To develop a general approach to analysis of breakout forces which would be equally valuable for a variety of anchor problems as well as for ship-salvage problems, it would be desirable to attack simultaneously all four problem areas listed above. Priorities could be established by limiting oneself to one of the two major classes of breakout problems, (anchoring, ship salvage) or to one of the two major classes of soils (cohesionless, cohesive). However, considering the present status of development of ocean engineering, it might be wise to invest, under any circumstances, some funds into basic studies such as those listed under 4) and 1).

The following research studies, listed in order of priority, are particularly recommended at this time:

(a) A study of suction under objects subjected to pullout. This study should be both theoretical and experimental. It should explore the possibilities of the "pore-pressure approach" as well as "viscous flow approach" for prediction of suction forces. The experiments should be made primarily on laboratory models in strictly controlled conditions. A final phase of the project would include a field verification of the developed theories.

(b) A study of the nature and magnitude of adhesion between ocean sediments and embedded objects. This study should be conducted experimentally, first in the laboratory and later in the actual ocean environment. It should center around effects on adhesion of variables such as time, pressure, mineral composition of the sediment, material of the object and physico-chemical characteristics of the environment. It should also shed some light on relative magnitude of adhesion in tension to adhesion in shear.

(c) A study of rheological properties of seafloor sediments. This study should include laboratory investigations based on vane, cone and viscosimeter tests. Development of suitable equipment for field measurement of rheological parameters should follow in a second phase of the project. A further development would be to work out solutions of the breakout problem for one or two rheological models that can best simulate the actual response of liquid and semi-liquid soils.

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APPENDIX - SAMPLE PROBLEMS

Problem 1

Find the breakout force for a steel sphere 2 ft. in diameter, embedded at a depth of 5 ft. in a loose to medium dense sand deposit (γ_{dry} = 90 lb/cu.ft, G_s = 2.65, $\phi = 30^\circ$, c = 0)

Submerged unit weight of sand (2.65 - 1) 90/2.65 = 56 lb/cu.ft.
From Table 2: (D/B = 2.5, ϕ = 30°) F_q = 4.28
q_o = (4.28)(5)(56) = 1,200 lb/sq.ft.

Soil resistance:	(1,200)(0.785)(2^2)	3,770 lb.
Effective weight of the sphere:	(4/3)(1^3)(3.14)(500 - 62.4)	1,830 lb.
	Total breakout force	5,600 lb.

Problem 2

A cylindrical object, having a diameter of 12 ft, and a length of 60 ft. is embedded in a horizontal position in the ocean bottom so that ir protrudes 3 ft. above the bottom. The surrounding soil is a soft organic clay, with an undrained shearing strength varying with time to failure t as

$$s_u = s_\infty + (s_o - s_\infty) \exp\left(1 - \sqrt{t/t_o}\right) \qquad (18)$$

Here s_o represents the undrained shearing strength for time to failure t_o and s_∞ long term undrained shearing strength of the soil. In the considered case s_o = 180 lb/sq.ft at t_o = 10 min and s_∞ = 100 lb/sq.ft. The submerged weight of the object is 17,200 lb. and the submerged unit weight of the soil is 40 lb/cu.ft. The adhesion c between the object and the surrounding soil is assumed to be equal to 20% of the undrained shear strength. It is further assumed that the suction varies as

$$u = u_o \exp(-\sqrt{T/t}) \qquad (19)$$

where u_o = 2,100 lb/sq.ft = suction for zero pullout time t, and T = 1 hr.

Find the breakout forces corresponding to pullout time of 1 hr., 24 hr.

(a) Breakout force for 1 hr. pull-out
Undrained strength
$s_1 = 100 + (180 - 100) \exp(1 - \sqrt{1/0.16}) = 119$ lb/sq.ft
Adhesion
$c_a = 0.20\, s_1 = 24$ lb/sq.ft
Suction
$u_1 = 2{,}100 \exp(-\sqrt{1/1}) = 775$ lb/sq.ft

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From Table 1 ($D/B = 3/12 = 0.25, \phi = 0$): $F'_c = 0.41$, $F'_q = 0.10$
Ultimate soil resistance:
$q_o = (119)(0.41) + (3)(40)(9.10) = 60.8$ lb.sq.ft

Computation of breakout force:

Effective weight of the object:		17,200 lb.
Resistance of overburden soil:	(12)(60)(60.8)	43,800 lb.
Adhesion:	(12)(60)(24)	17,300 lb.
Suction:	(12)(60)(775)	557,500 lb.
	Total breakout force:	635,800 lb.

(b) Breakout force for 24 hr. pull-out

Undrained strength
$s_{24} = (100) + (180 - 100) \exp (1 - \sqrt{24/0.16}) \approx 100$ lb/sq.ft
Adhesion $c_a = (0.20)(100) = 20$ lb/sq.ft
Suction $u_{24} = 2{,}100 \exp (-\sqrt{1/24}) = 15.9$ lb./sq.ft

Ultimate soil resistance:
$q_o = (100)(0.41) + 3(40)(0.10) = 53$ lb/sq.ft

Effective weight of the object		17,200 lb.
Resistance of the overburden soil:	(12)(60)(53)	38,200 lb.
Adhesion	(12)(60)(20)	14,400 lb.
Suction	(12)(60)(15.9)	11,400 lb.
	Total breakout force:	81,200 lb.

Note: The relationship (18) between the time to failure and undrained shearing strength can be determined experimentally. However, the present state of our knowledge does not provide a rational method for determination of relationship (19) between suction and breakout time. The selected example points to the significance of suction for the case of partially embedded objects in cohesive sediments.

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13. ABSTRACT

A study of factors ~~affecting~~ the magnitude of breakout resistance of objects embedded in ocean bottom. Following a literature review most discussions deal with the basic problem of a centrally loaded object pulled by vertical force from a sediment with level surface. Considerations of effects of eccentric and inclined loading and slope of the ocean bottom are added. It is found that failure patterns depend on relative depth of the object, as well as on soil type. Theoretical analyses show better agreement in soft and loose soils than in stiff and dense soils. Suggestions for analysis of suction force and effects of soil liquidity are given. Recommendations for future research include four problem areas: (1) Effects of soil liquidity and compressibility on failure patterns; (2) adhesion between soil and buried objects; (3) nature and magnitude of suction force; (4) rheological properties of soils in breakout.

14. Key Words

Breakout forces

Anchors

Ocean bottom

Pullout capacity

Soil behavior

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