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STUDY OF SLOPE INSTABILITY IN THE OCEAN FLOOR

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ABSTRACT

There are three sections to the report: (a) ocean floor soil types and characteristics; (b) stability analysis of submarine slopes; and (c) discussion of problems, with conclusions and recommendations. In the first section the physical and mechanical properties of the soils encountered in various types of ocean-floor terrain are described and summarized in tables and diagrams. The application of conventional subaerial slope stability analyses to ocean-floor soils and their environment is examined in the second section. Some aspects of the stability of underconsolidated soils forming in areas of rapid deposition are given particular attention. Following a summary of the conclusions reached in the study, some detailed recommendations regarding future studies and experimental work are given in the final section of the report.

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STUDY OF SLOPE INSTABILITY IN THE OCEAN FLOOR

Introduction

The objective of this study is to summarize and appraise all available information within the present state of the art for predicting the occurrence of slope instability on the ocean bottom. Consideration shall be given to all types of potential slope instability from deep-seated landslides to mud flows and creeps. All ocean bottom environments will be considered.

This paper is divided into three major categories. First, properties of the ocean floor itself and those characteristics of the ocean floor which are important to slope stability considerations; secondly, slope stability analysis of submarine slopes; and thirdly, an analysis of the major problems encountered and recommendations for further study.

I. CHARACTERISTICS OF THE OCEAN FLOOR

(a) Ocean Floor Sediments; Distribution, Properties, and Slopes.

Before embarking on the study of submarine slides, it will be of some use to consider properties of ocean floor sediments themselves. To the engineer concerned with slope stability analysis, the physical properties of the soils, particularly the strength parameters and the existing natural slopes, are necessary inputs to his analysis. Since reliable strength measurements for most of the ocean floor are unavailable at present, the engineer is often forced to seek correlations between sediment-type and physical properties.

Keller (31) has compiled a useful summary of shear strength and other physical properties of ocean sediments, based on 300 sediment cores in the Atlantic and 200 sediment cores in the Pacific. The results of Keller's study are shown in Figures 1 through 8. In all cases the information shown represents only an average value for the upper few feet of the ocean floor.

It is important to realize this limitation on the data presented by Keller. Extrapolating these values to greater depths requires that some information be gathered on the variation of the strength parameters with depth. It is likely, however, that the properties of the underlying material are related to those of the surficial deposits in each area. If this is so, the surface information will provide some feeling for the variability of ocean floor soils throughout the expanse of the ocean floor. Sediment types shown in Figures 1 and 2 are divided into six classes:

1. Fluvial-marine (sand-silt grains larger than 0.016 millimeters): representing the coarser fraction;

2. Fluvial-marine (silt-clay grains smaller than 0.016 millimeters): the finer fraction of material derived from terrestrial drainage;

3. "Red-clay": a term applied to inorganic pelagic clays which vary considerably in color, but are usually chocolate brown;

4. Calcareous ooze: sediment composed of at least 30% calcium carbonate in the form of skeletal material from various planktonic animals and plants;

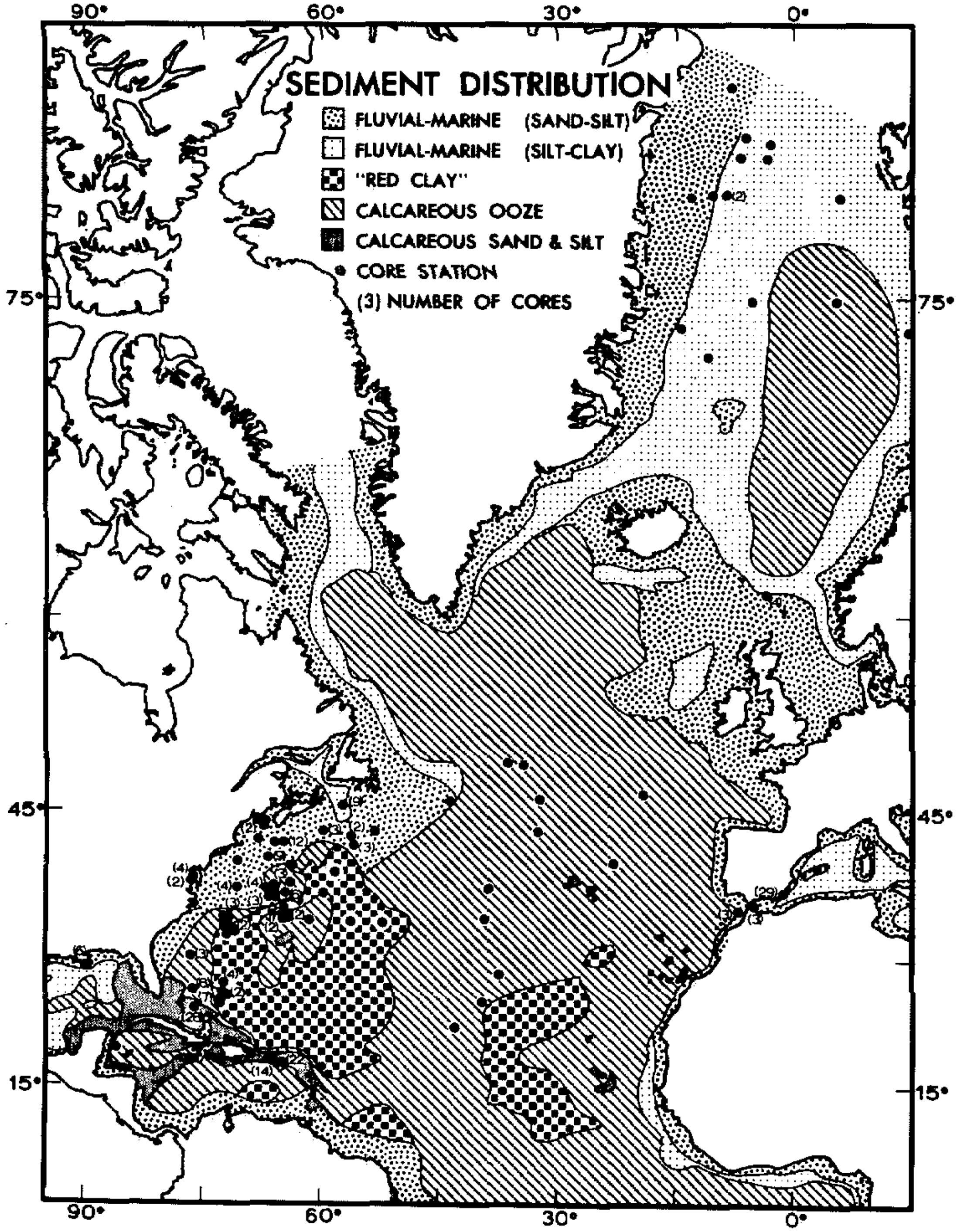
5. Calcareous sand and silt: shell fragments and coralline debris of sand and silt-sized particles; and

6. Siliceous oozes: deposits containing 30% or more of siliceous, skeletal material derived from either diatoms or radiolarians.

A conspicuous difference between the North Pacific and the North Atlantic basins is the percentage of the sea floor covered by "red clay" deposits. The area of sea floor covered by "red clay" in the Pacific is at least four times that found in the Atlantic. The differences between the North Pacific and North Atlantic basins are also evident from the study of engineering properties of the sediments. The shear strengths of ocean-floor sediments shown in Figures 3 and 4 were obtained in soils composed essentially of fine-grained cohesive material with a few seams of fine sand occurring in a small number of the core samples. Shear strength measurement was made by either a laboratory vane shear test or the unconfined compression test. The vane shear test is performed by inserting a small four-bladed vane into the retrieved sample and applying an increasing torque. The shearing stress corresponding to the maximum torque is the shearing strength of the specimen. This was the only test that could be used when very low shear strengths were encountered. Unconfined compression tests were utilized for many of the stronger samples. It should be borne in mind that deep-sea sampling and sample return techniques subject the soils to considerable amounts of disturbance, and the in-situ strengths of the soils are likely to be greater than those reported by Keller.

In his study, Keller considered sediments to be clays or materials behaving like clays, and the shear strength was taken as one-half of the unconfined compressive strength. Average shear strength values in the ocean basins range from less than 0.5 to 2.5 psi for the upper few feet of sea floor sediments. Sediments with a shear strength of 0.5 to 1.0 psi appear to predominate in the North Atlantic basin. Shear strengths of 1.0 to 1.5 psi are the highest observed in the North Atlantic and are associated with calcareous deposits. Values of less than 0.5 psi are often found in coastal areas where local drainage or current conditions strongly influence the depositional environment. In these immediately offshore areas minor changes in the environment can result in significant variations in the mass properties of the sediments. Other areas of low shear strength are found in association with "red clay" deposits and in pockets of sediment along the mid-Atlantic ridge. The most prominent portion of the Atlantic basin displaying shear strengths of less than 0.5 psi is that east of Greenland. As shown by these values, this section of the ocean floor differs considerably from other portions of the North Atlantic.

Some question exists as to the accuracy of the measurements in this area and further information should be gathered to confirm or revise this low value of shear strength. In contrast to the North Atlantic, large portions of the North Pacific sea floor are covered with sediments with an average strength of less



After Keller (31)

Fig. 1 Sediment Distribution

Atlantic Ocean

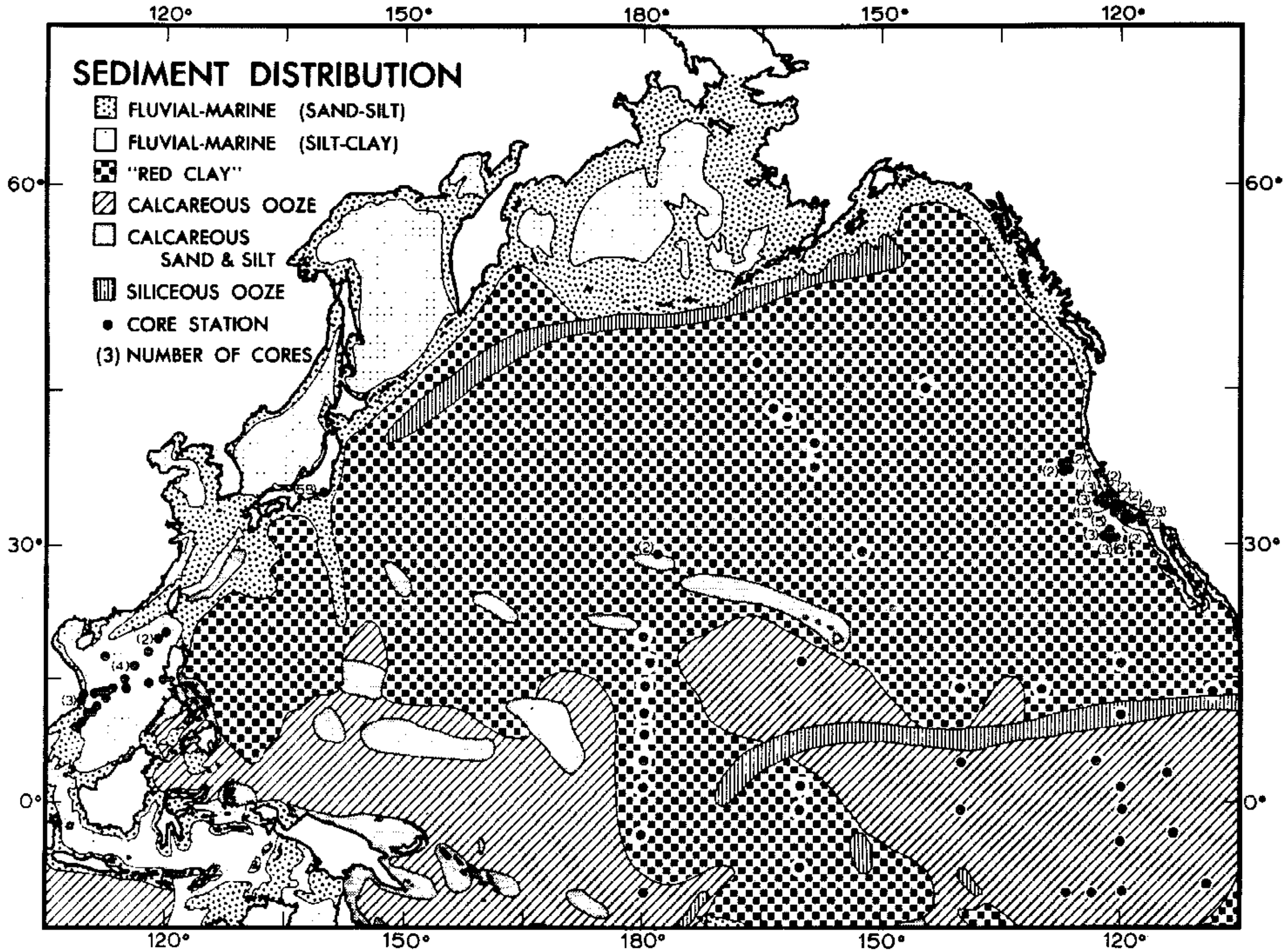
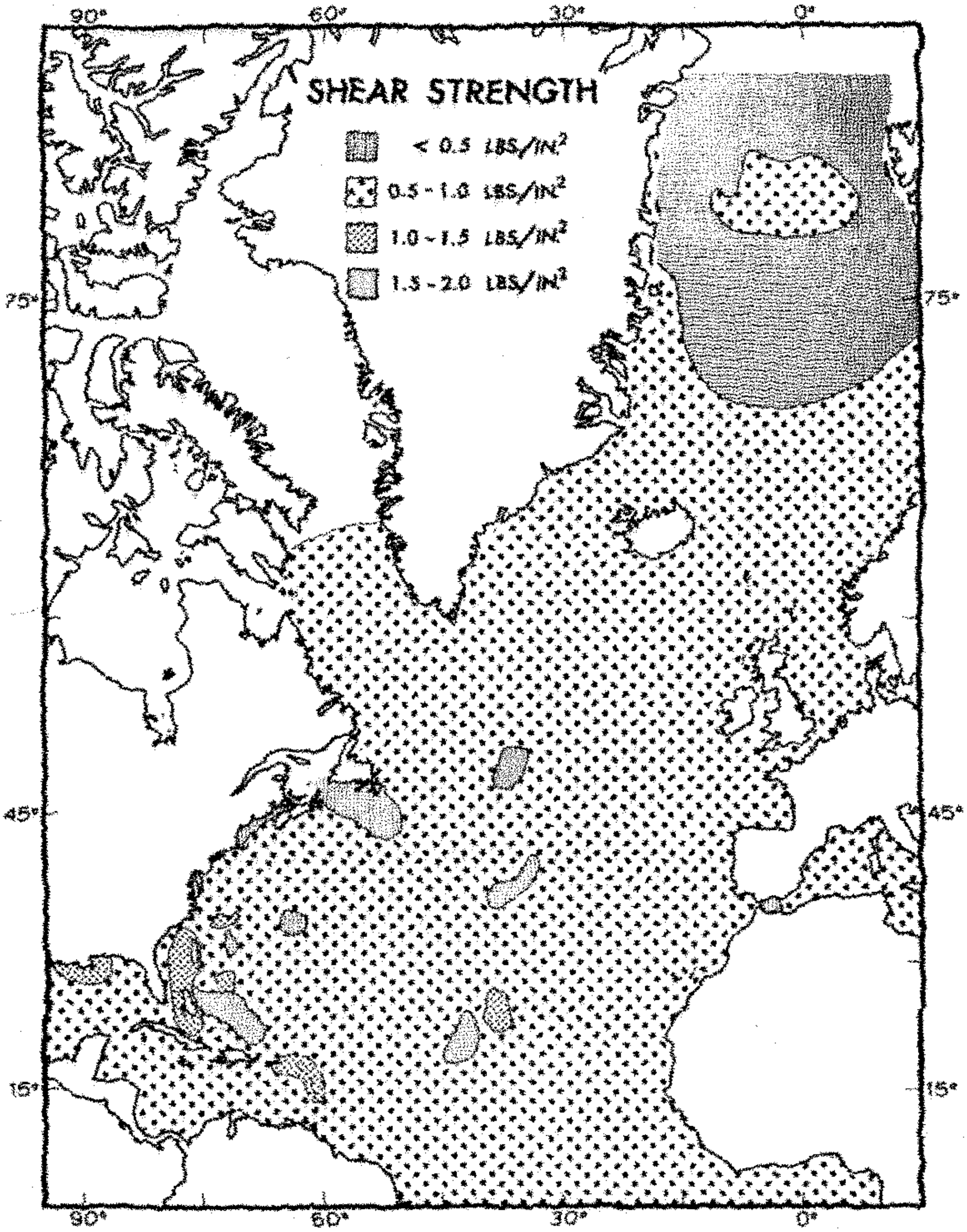


Fig. 2 Sediment Distribution

Pacific Ocean

After Keller (31)



After Keller (31)

Fig. 3 Shear Strength

Atlantic Ocean

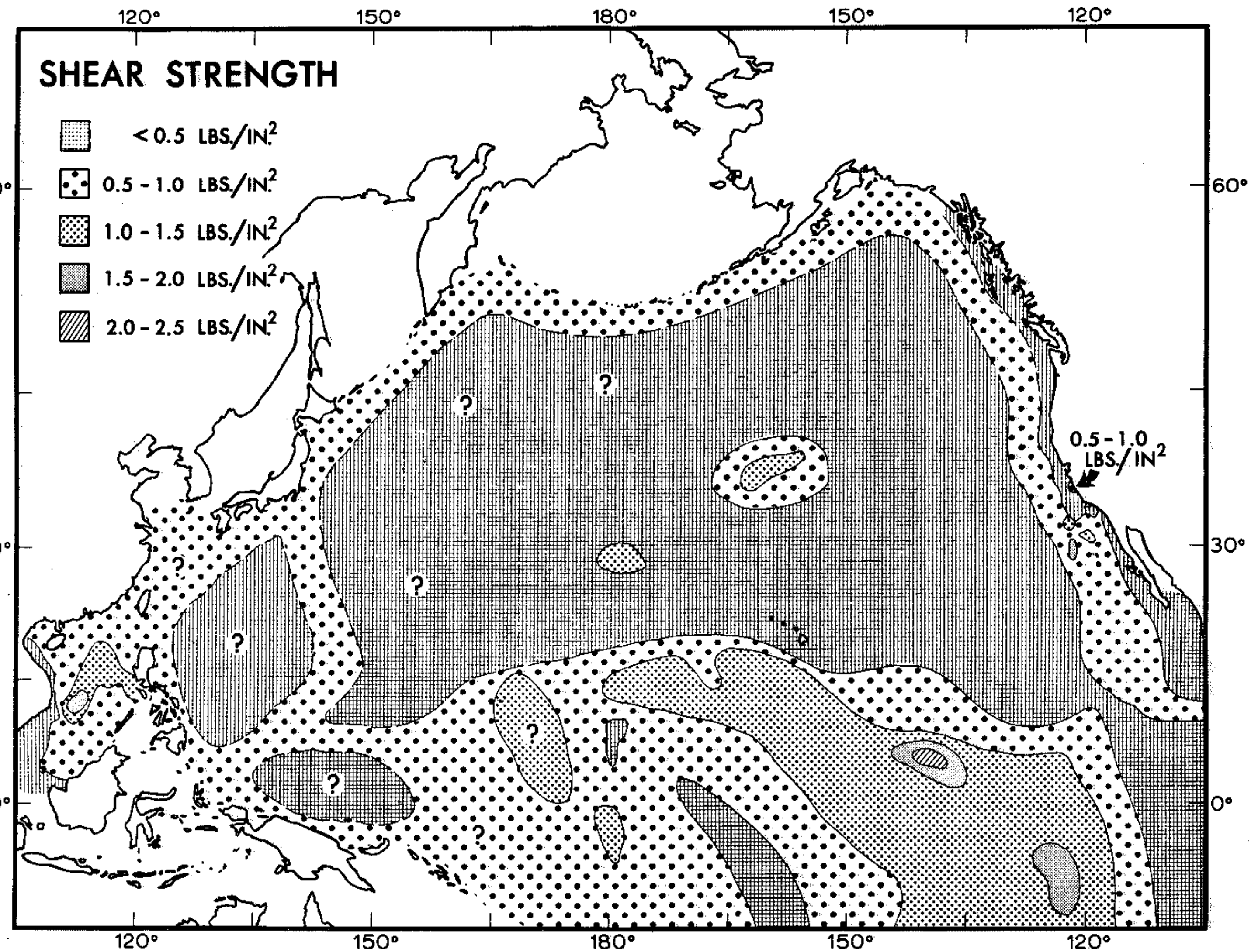


Fig. 4 Shear Strength

Pacific Ocean

After Keller (31)

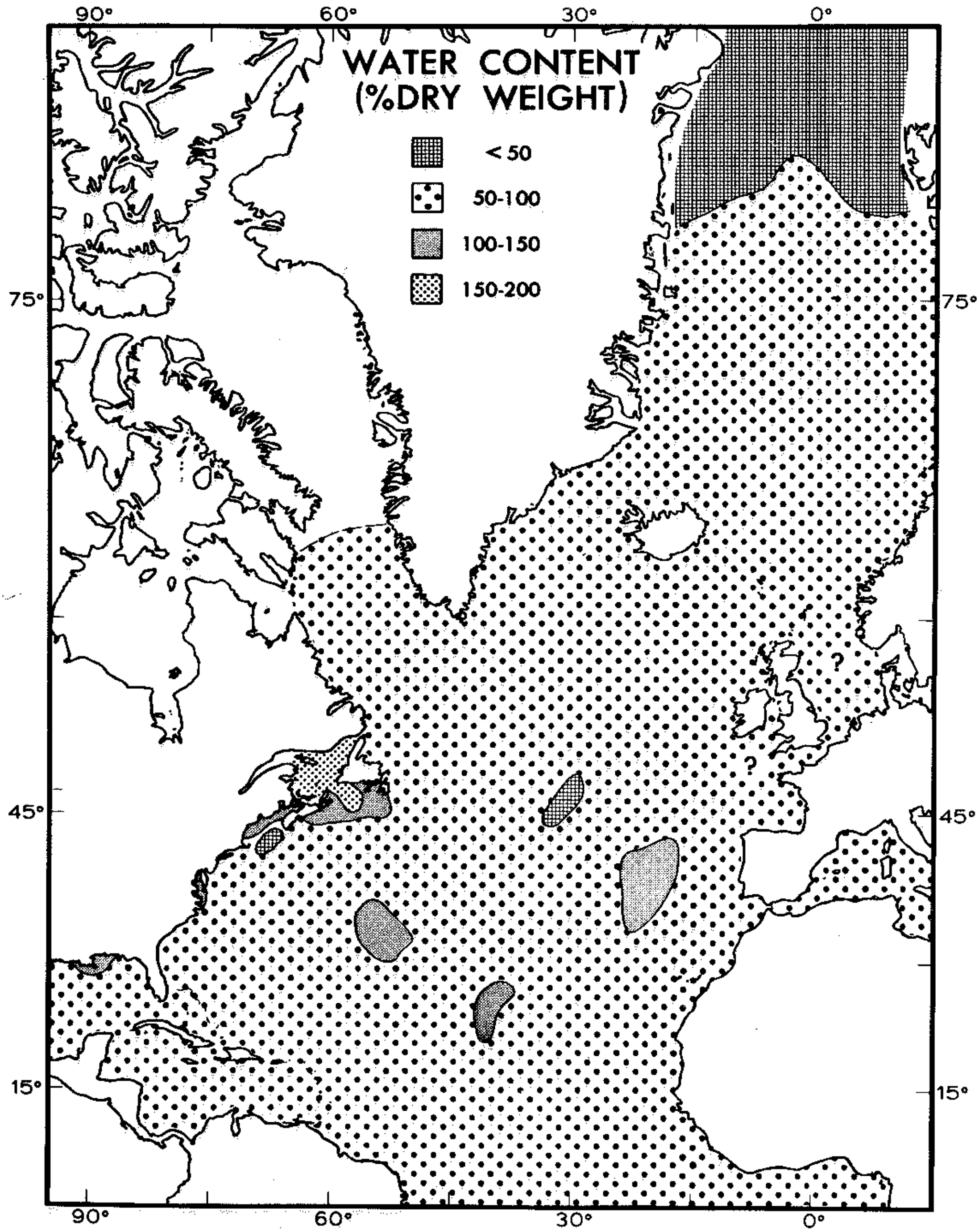


Fig. 5 Water Content After Keller (31)

Atlantic Ocean

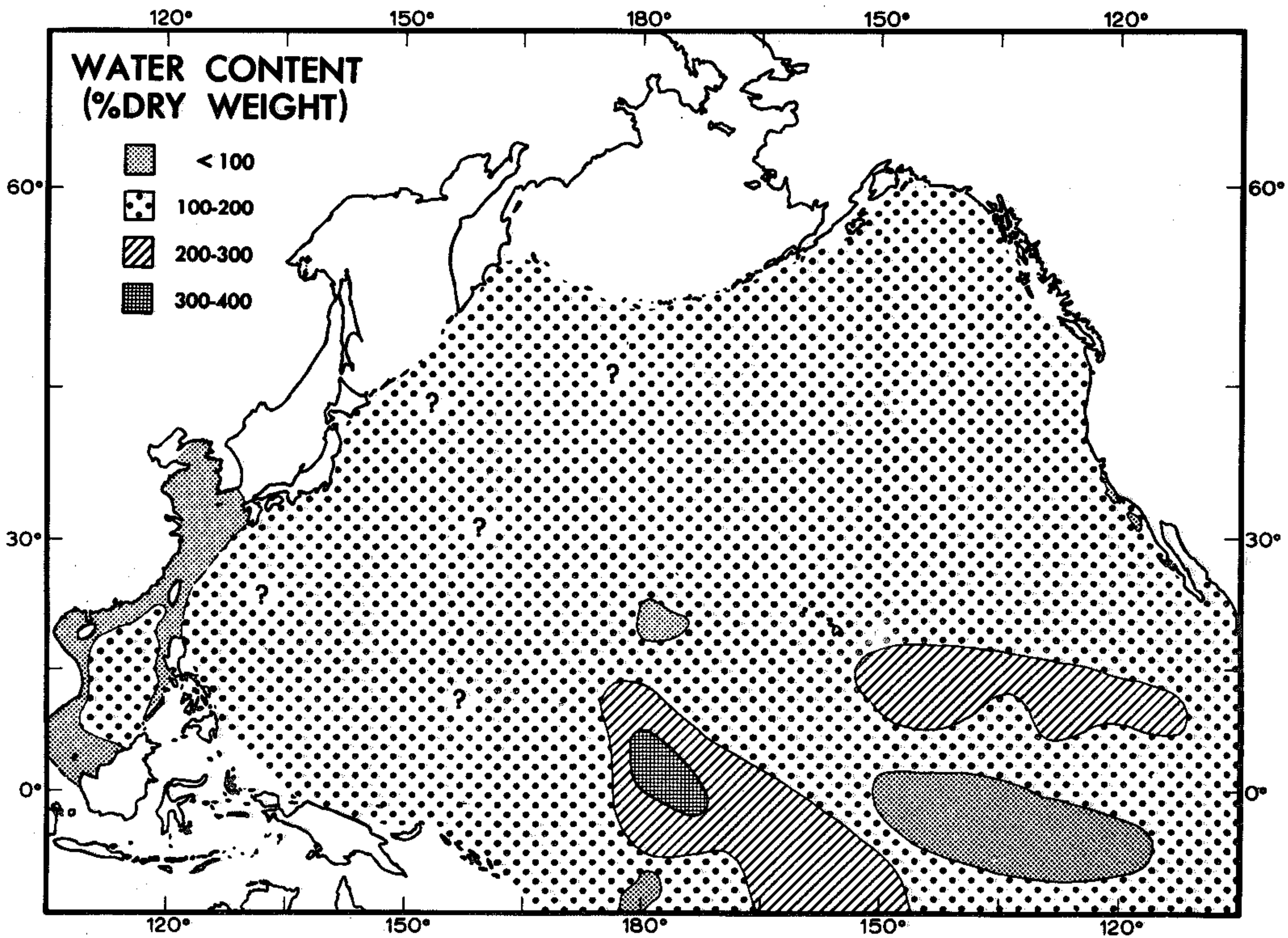


Fig. 6 Water Content

Pacific Ocean

After Keller (31)

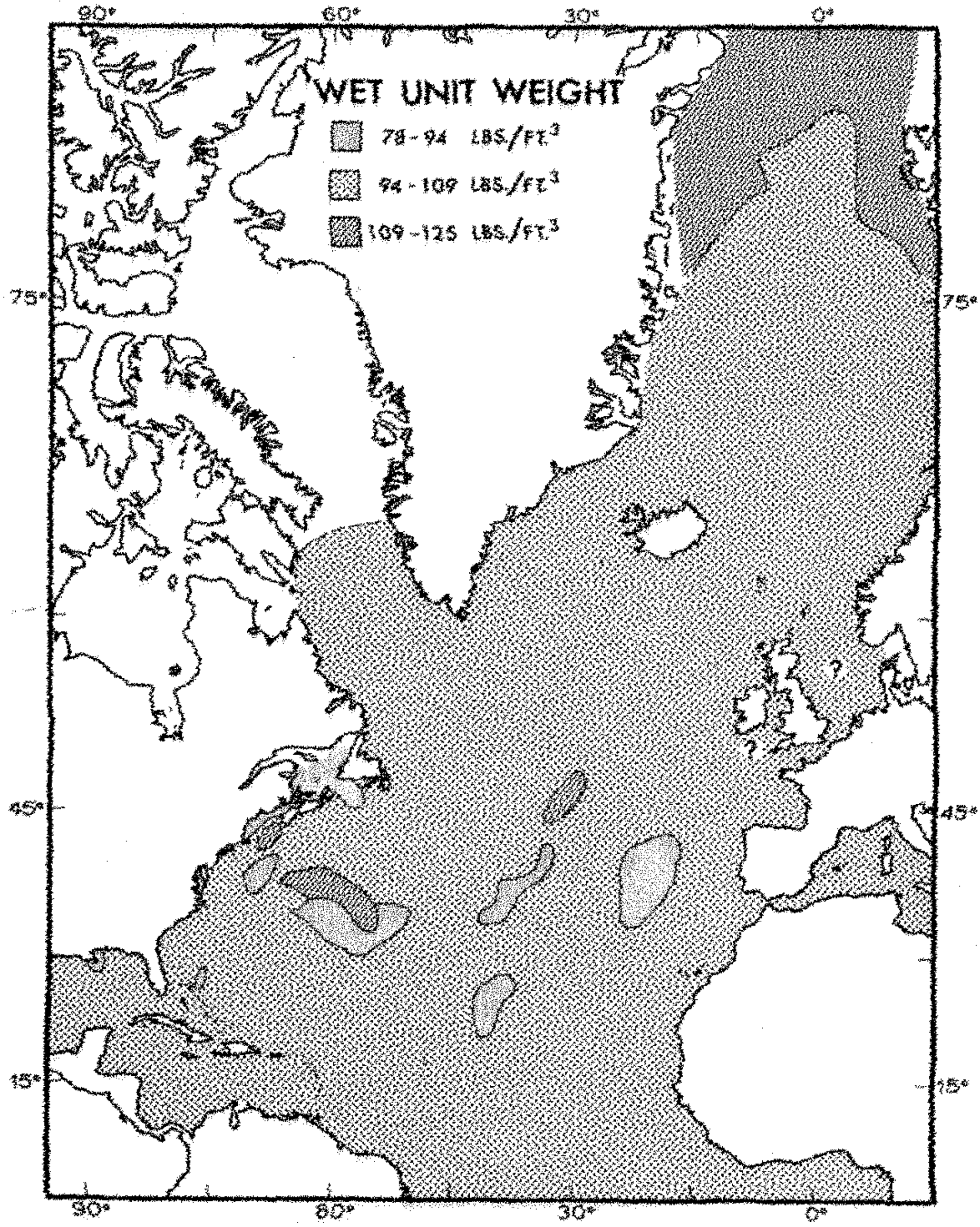


Fig. 7 Wet Unit Weight After Keller (31)

Atlantic Ocean

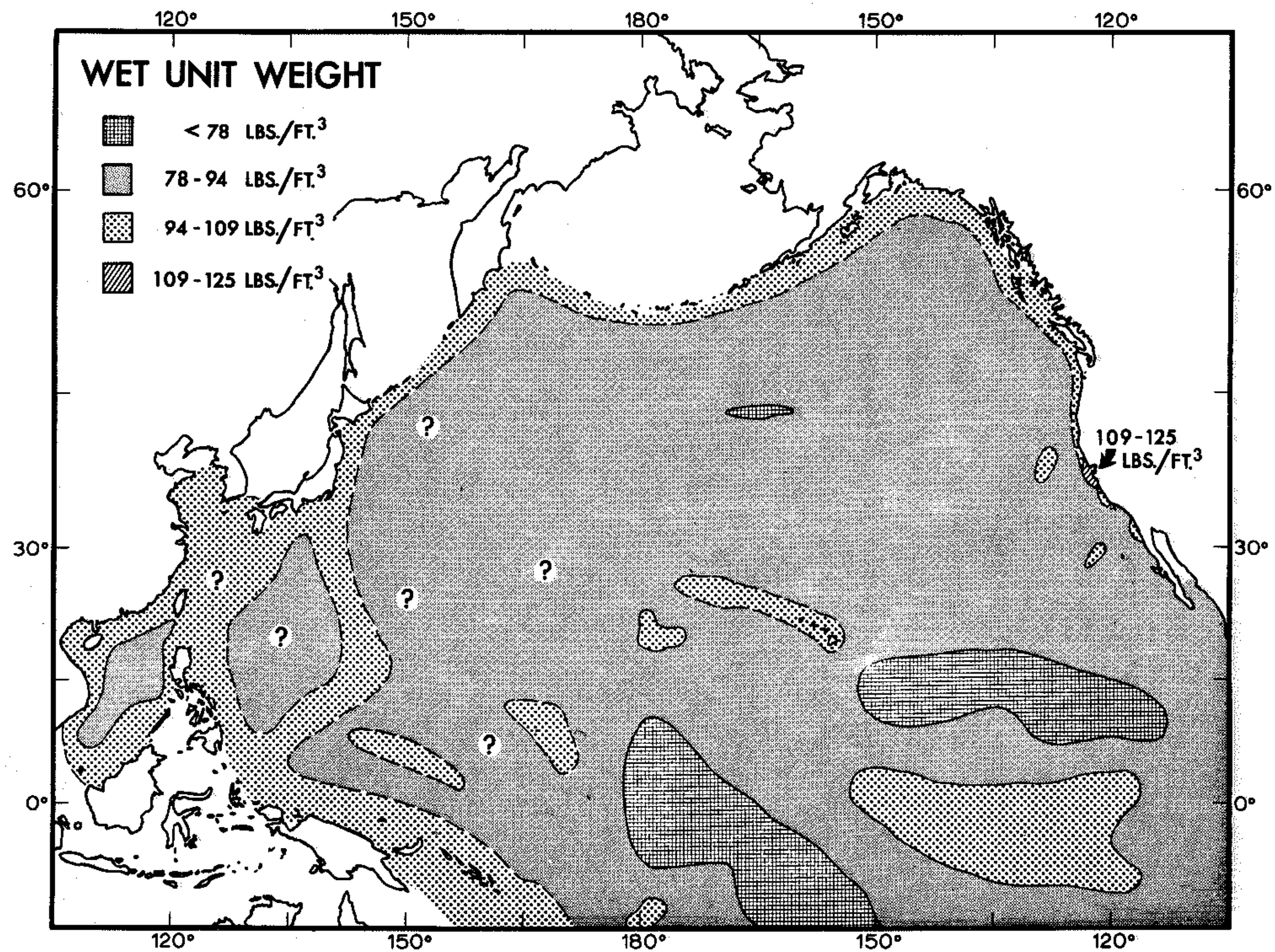


Fig. 8 Wet Unit Weight After Keller (31)

Pacific Ocean

than 0.5 psi. These areas coincide closely with the distribution of "red clay" which comprises a major portion of the sea floor. Localized areas of higher shear strength can occur within an area of low shear strength. This is attributed by Keller to changes in bottom topography which favorably influence the depositional environment. Local currents in and around topographic features can account for changes in the distribution of certain sediment properties. Shear strength values are generally higher along the margins of the basin and in the lower latitudes. In areas with abundant calcium carbonate, such as the low latitudes, shear strength has a tendency to increase. The highest shear strength values observed thus far occurred in the calcareous oozes of the Pacific basin, and were in the range of 2.0 to 2.5 psi.

In summary, therefore, Figures 3 and 4 indicate that North Atlantic soils possess relatively higher shear strength than those of the North Pacific. The North Pacific basin can be divided into two sedimentary provinces, each distinctly different. The northern portion consists of sediments possessing strengths ranging from 0.25 to 0.5 psi. In contrast, the southern portion consists of sediments whose shear strength varies from 0.1 to 1.5 psi. The North Atlantic does not display these sedimentary provinces, except for the questionable area east of Greenland discussed above.

Figures 5 and 6 show the water content of sediments in the Atlantic and Pacific, respectively. The measurements shown are expressed as the percent of the weight of pore water to the weight of oven dried solids in a given soil. Water content ranges between 50 and 100 percent in most areas of the North Atlantic and 100 and 200 percent in most areas of the North Pacific. The finer Pacific sediments possess a considerably higher water content than the relatively coarser sediments of the North Atlantic. The higher values of water content are often found in association with "red clay" deposits, and, therefore, areas of lower shear strength; whereas, calcareous sediments in local coastal areas appear to possess relatively low water content and can be associated with higher shear strengths.

Information on wet unit weight is shown in Figures 7 and 8. Wet unit weight is the weight per unit of total volume of a sediment mass. Samples taken from the sea floor are sufficiently close to 100% saturation to allow use of the term "saturated unit weight", which is the in-place total unit weight shown in these figures.

Total unit weights are slightly higher in the North Atlantic than in the North Pacific. Soil unit weights of 94 to 109 pcf predominate on the Atlantic sea floor; in contrast, extensive regions of the Pacific are covered with sediments in the range from 79 to 94 pcf. Regions of low density material, less than 78 pcf, are shown to exist in the North Pacific, but as yet such areas have not been discovered in the North Atlantic. These low densities are commonly associated with areas of "red clay." The low values occurring in the vicinity of 3° south and 170° west may possibly be attributed to the particular depositional environment caused by the numerous islands found in the area.

Other areas of low density material are associated with the relatively flat abyssal floor which rules out the possibility of bottom topography influencing the depositional environment in these particular areas. The highest densities observed are in the soils east of Greenland where values as high as 125 pcf are reported.

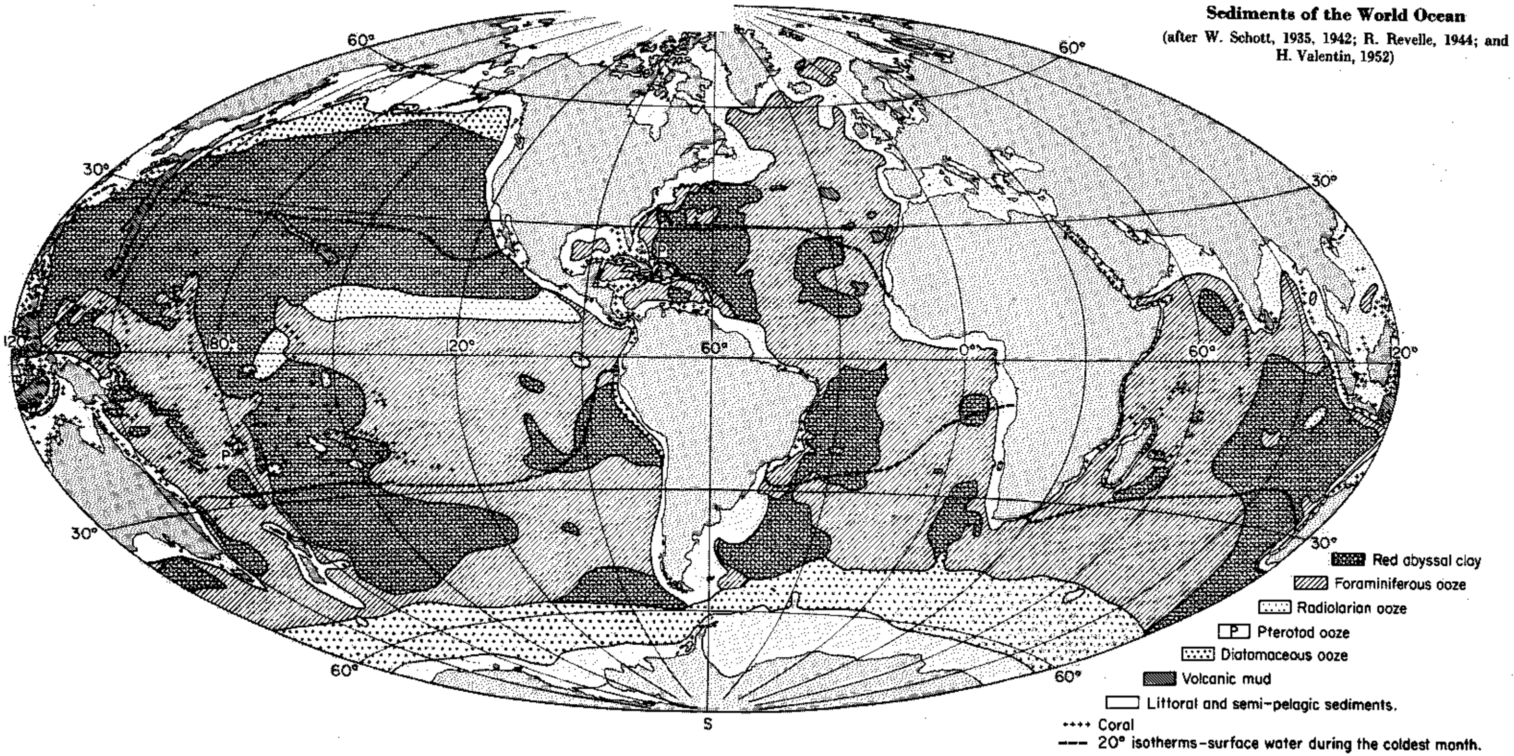
Keller's observations show clearly a direct relationship between sediment type and the mass physical properties observed. A direct relationship between mass properties and such factors as water depth and ocean currents is also observed in some areas.

Keller concludes his study with the following generalizations: The extensive "red clay" deposits commonly possess high water content, but low shear strength and bulk density. Exceptions to this are usually in areas affected by variations in the sea floor topography. Calcareous deposits are more or less the opposite with relatively low water content, but with high shear strengths and bulk density. The presence of various types of skeletal debris along with cementation characteristics for certain calcareous deposits account strongly for a wide variation of mass properties from one deposit to another. Mass properties in coastal areas vary considerably from area to area because of the influence that local drainage and currents have on the depositional environment. In the vicinity of major rivers, shear strength and bulk density are relatively low; water content values are higher than those found on the deep-sea floor.

The area east of Greenland is given special attention by Keller because of its noticeable difference from the other areas studied. This area can be described as a small basin in itself which is strongly influenced by both currents and inflow of sediment from the surrounding area. The low water content and high bulk density values have been attributed to the generally coarse, heavy mineral-rich deposits found in this basin. The presence of relatively low shear strength values in this same area cannot readily be explained. It is pointed out that low strength values are generally found in conjunction with clayey sediments which possess high water content and low densities. The possibility that these samples were overly disturbed is not ruled out by Keller, but until more substantial data is obtained, this area should be considered one of uncertainty.

Keller's studies of sediment distribution are limited solely to the northern portions of the Pacific and Atlantic ocean floors.

A world-wide picture of ocean floor sediment distribution has been compiled by Dietrich (7). His chart of sediments of the world ocean, based on seven classes of sediments, is shown in Figure 9. Unfortunately, Dietrich did not concern himself with the correlation between sediment type and mass physical properties. Terry (64) also divided ocean sediments into seven classifications, but did not present the geographic distribution of ocean



After Dietrich (7)

FIG. 9

TABLE I

DISTRIBUTION OF VARIOUS DEPOSITS

Type of Deposit	Percentage of Sea Floor	Average Depth (Meters)
Shelf Sediments	8	100
Muds (blue, green, volcanic, coral)	18	2300±
Globigerina ooze (calcareous)	35	3600
Pteropod ooze (calcareous)	1	2000
Diatom ooze (siliceous)	9	3900
Radiolarian ooze (siliceous)	2	5300
Red Clay	28	5400

After Terry (64)

sediments; however, he did compile figures on the percentage of the sea floor occupied by each of the seven sediment types. The results of his studies are shown in Table 1. In addition, Terry observed that bottom sediments are generally extremely weak. In many cases, bottom sediments exhibit great lateral variation. Shear strength in one location cannot be assumed to continue uniformly in lateral or vertical directions for any significant distance without verification. Accumulated sediment on slopes may be unstable and weak in bearing strength.

Good correlation often exists between topography and type of sea-floor material. Geographic locations which deserve particular attention are those areas immediately off shore from large river deltas. Morgenstern (43) points out after Terzaghi (66) that in these regions of high rates of sedimentation, there exists a lag between the accumulation of the material and the consolidation associated with it. This gives rise to an excess pore pressure, and the sediment is accordingly weaker. This underconsolidated material is thought to be prone to slumping.

The information discussed above deals in terms of general geographic areas. Considerable work has been done to investigate soil properties with depth in some locations, particularly those which were part of the Mohole experiment. Principal investigators in this field have been Hamilton (19, 20); Moore (38, 40, 41); Richards (46, 47, 48); Arrhenius (2) and Fisk and McClelland (12). Some typical strength versus depth results are shown in Figures 10a through 10d. Additional work in this field was conducted by Emery (10). Although information on some ocean floor sediments to considerable depths has been obtained in a few areas, reliable data on the upper few meters of soils which most likely determine slope stability are scarce. Some of the difficulties encountered when collecting core samples of ocean floor soils were pointed out by Ross and Riedel (49) and Inderbitzen (30).

Perhaps one of the most significant questions which an engineer might ask, if he were required to make an estimate of the stable slope angle in a given area, concerns the distribution of existing natural slopes. A natural slope presumably indicates the angle at which the material is marginally stable over a relatively long time period. Shepard (56) has compiled some statistics, shown in Table 2, for the average slope from the edge of the continental shelf to a water depth of 6000 feet in four major regions of the ocean. Shepard observed that most soundings indicate an uneven continental slope with depressions that, judging from surveys off the United States, may be either valleys or basins. Slopes are more gentle off large rivers, especially those off deltas. The average slope near large river deltas is only $1^\circ 20'$ while the average off the general coast is $5^\circ 40'$. In these off-coast areas, slopes appear to be affected by geological faulting. Some of the steepest slopes in the world, however, are found off lands where no indication of recent faulting or of earth movement is found — examples given by Shepard being west Florida, the Brazilian Highlands, southwest Australia and Ceylon. (An earthquake, accompanied by subaerial surface faulting occurred in

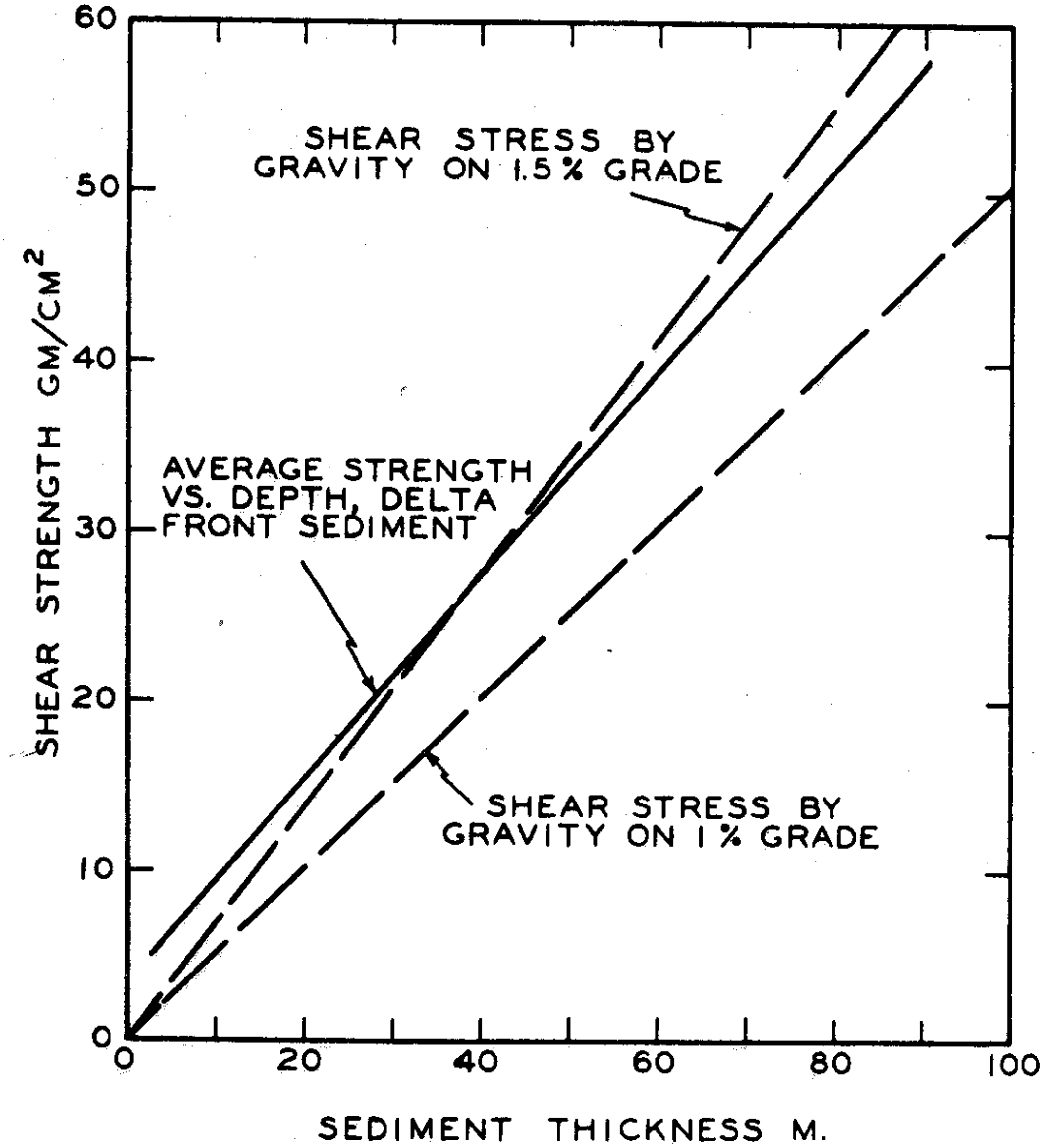
TABLE 2

SLOPE STATISTICS

(Shelf edge to 6000 feet water depth)

World Average	4° 07'
Near Large River Deltas	1° 20'
Near Coasts	5° 40'
Pacific	5° 20'
Atlantic	3° 05'
Indian Ocean	2° 55'
Mediterranean	3° 34'

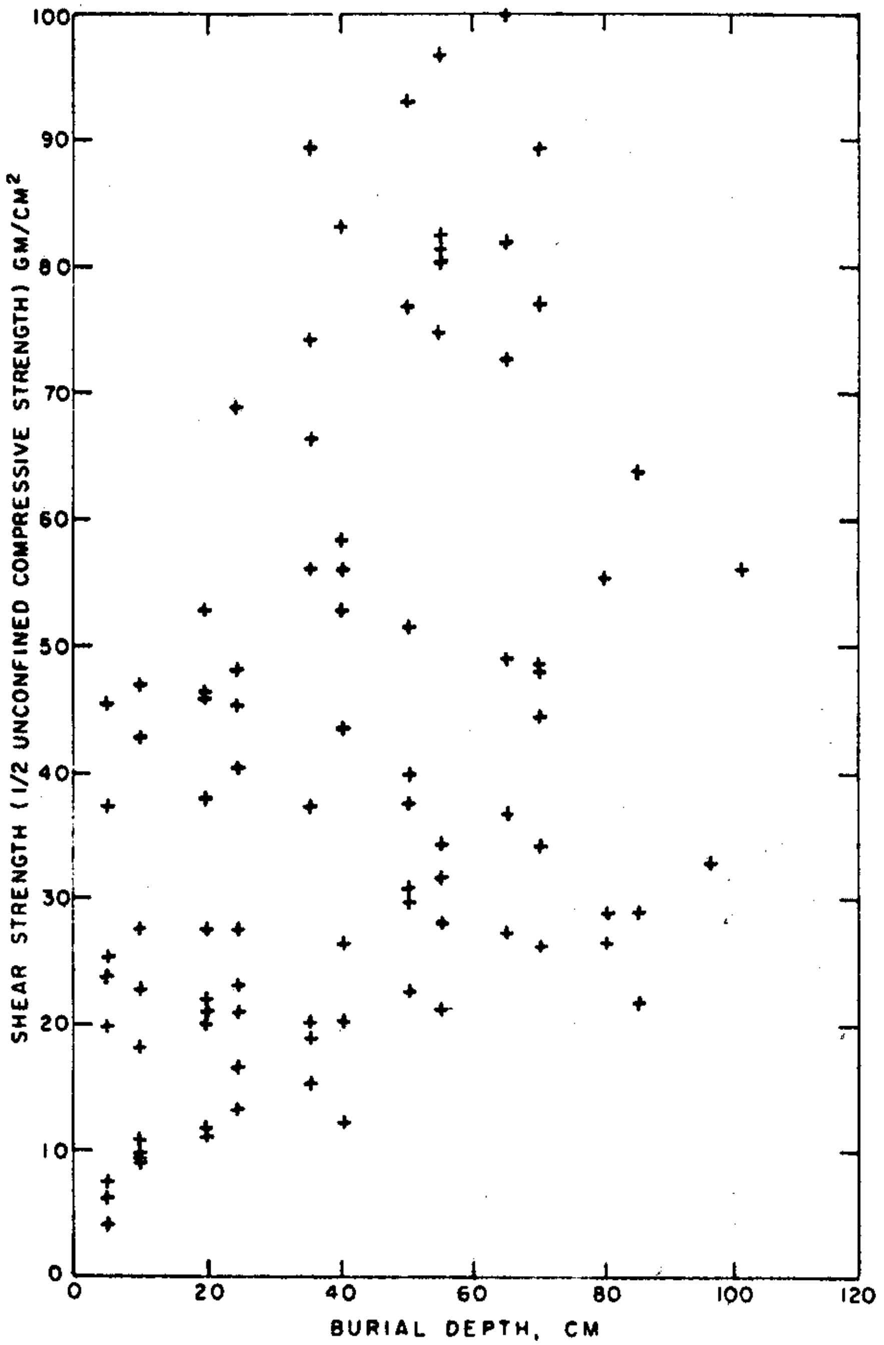
After Shepard (56)



Solid line shows approximate average increase in strength with burial depth of under-consolidated delta front sediments (from data of Fisk and McClelland, 1959). Instability of these sediments is shown by dashed lines indicating shear stress from gravity on very gentle grades of 1 and 1½ percent.

Fig. 10(a) Variation of Shear Strength with Depth

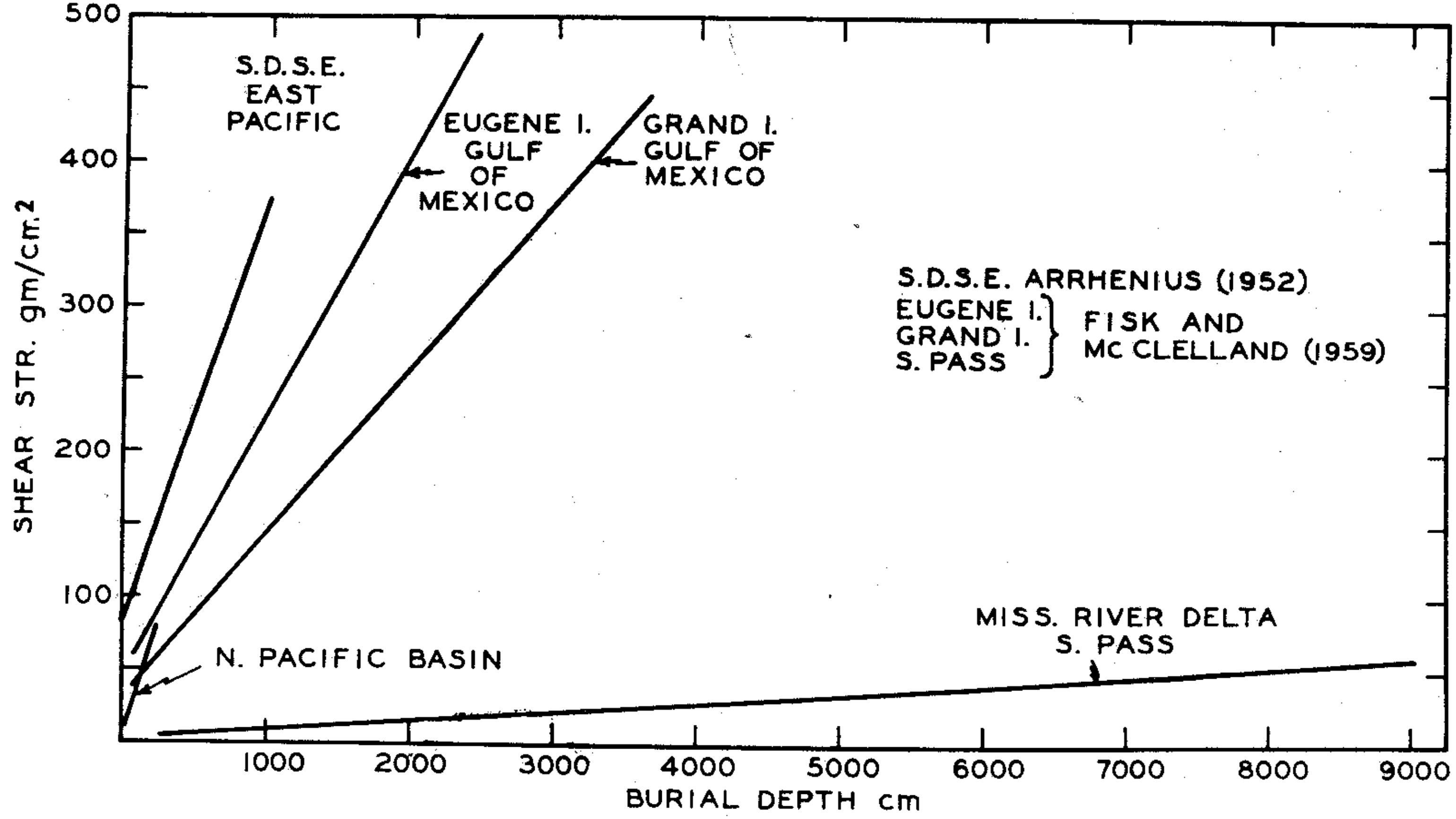
After Moore (42)



Shear strength against burial depth from Atlantic Ocean-floor cores in water depth range of 400–900 fms. Note that values of weakest sediments are comparable to those of the Pacific Ocean-floor samples.

Fig. 10(b) Variation of Shear Strength with Depth (Atlantic)

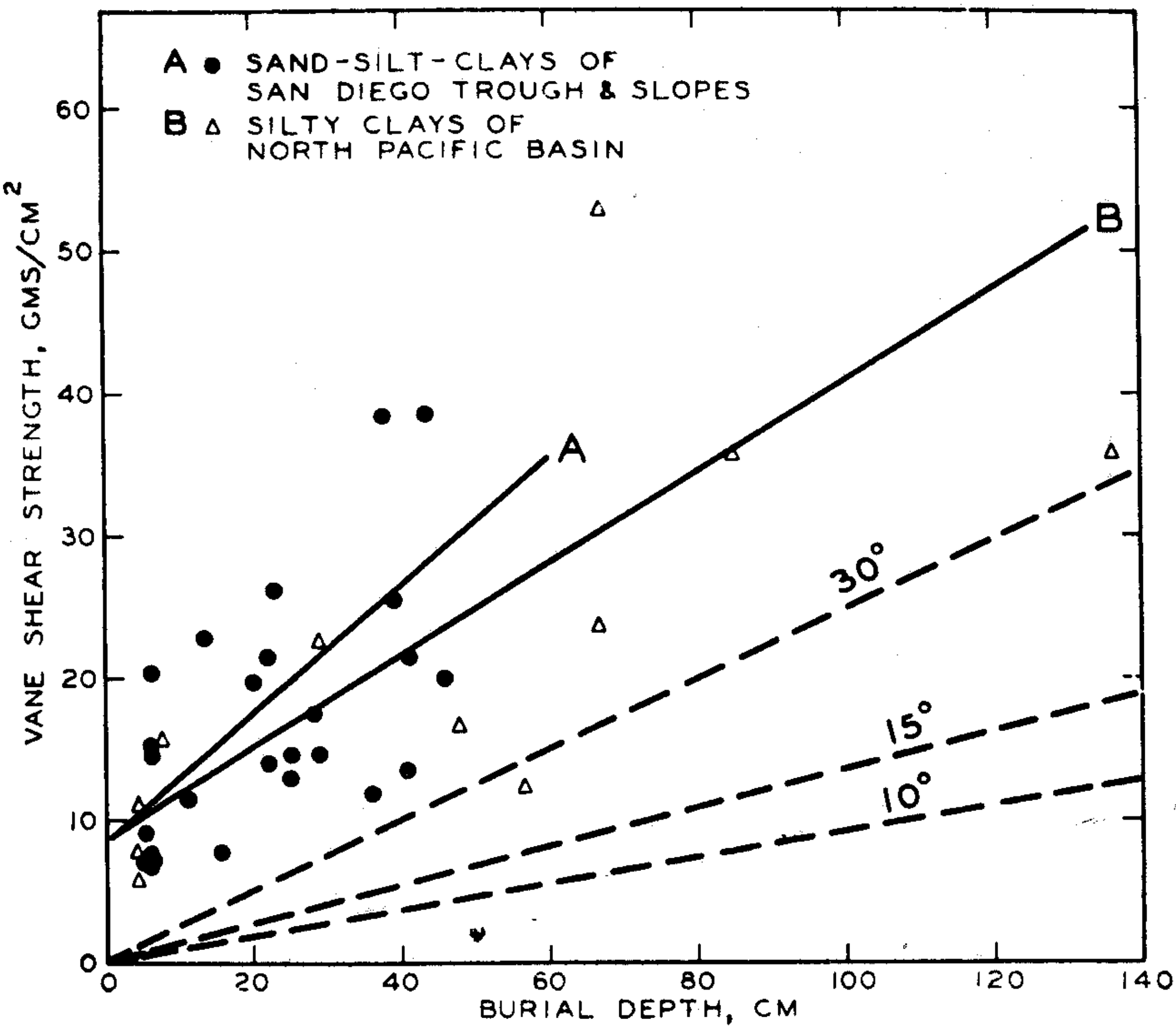
After Moore (42)



Curves of approximate shear strength against burial depth from four areas of strongly contrasting rates of accumulation. Greatest increase in strength with depth is in very slowly deposited deep-sea red clays of North and East Pacific basins; more gradual strength increases are from areas of progressively greater rates of sediment accumulation in northern Gulf of Mexico.

Fig. 10(c) Shear Strength Versus Depth and Rate of Accumulation

After Moore (42)



Least squares lines, A and B, of vane shear strength against burial depth from San Diego area and North Pacific basin sediments. Dashed lines show shear stress from gravity with increasing sediment thickness on slopes of 10, 15, and 30 degrees.

Fig. 10(d) Shear Strength Versus Depth (Pacific)

After Moore (42)

Western Australia in October, 1968). According to Shepard, sediments on these ocean slopes consist of about 60% mud (silt and clay sizes), 25% sand, and 10% rock and gravel, the remainder being shells or ooze. These proportions differ from those on the continental shelves where sand is more common than mud although the percentages of rock and gravel are similar.

(b) Classification of Soil Movements

Geological classifications used to describe subaqueous gravity movements are beneficial to the engineer's understanding of these phenomena. Dott (9), following subaerial practice, has classified subaqueous gravity movements into four major categories illustrated in Figure 11. It is important to understand the distinctions among these classes of movement categories to eliminate confusion in the use of such terms as sliding, slumping, flow, and turbidity currents. The term, subaqueous rock fall, as shown in Figure 11(a) refers to a very sudden falling and rolling of essentially undeforming fragments. It probably constitutes only a small proportion of underwater mass movement. True subaqueous sliding, shown in Figure 11(b), should be, according to Dott, applied only to mass movements of blocks of soil along discrete shearing surfaces with relatively minor disturbance to the internal structure of the sliding blocks. The term "slumping" as it has commonly been used for subaqueous deposits is synonymous with sliding, but it has been employed so loosely in the literature that the term is of little merit. When used it should refer only, as above, to sliding of masses along distinct shearing surfaces.

The slides described above involve the movement of a mass of material above a distinct sliding surface, below which the parent rock or soil is essentially undisturbed. Other slides can occur in which an interface is not so clearly distinguishable; these are usually called "flow-slides" or "flows". The material possesses fluid rather than solid characteristics and deforms essentially as a viscous liquid.

Dott distinguishes between the plastic flow of a mass of soil and the viscous flow characteristic of sediment-laden density currents. Plastic mass flow, as shown in Figure 11(c), contorts the existing bedding pattern. On steeper slopes the viscous flow becomes turbulent and develops into a turbidity current as shown in Figure 11(d). The terms plastic and viscous, employed in this context, do not conform to the usage of applied mechanics.

Logically, it would seem that a turbidity current might develop from either a block slide of the type of Figure 11(b), or from a flow slide. In either case a prime requisite would be an unstable material stress-strain relation, in which an *increase* in the strain or displacement occurring would lead ultimately to a decrease in the strength of the material. On a given slope the slide would thus accelerate. In extreme cases, the distortion in the flow will result essentially in liquefaction of the soil.

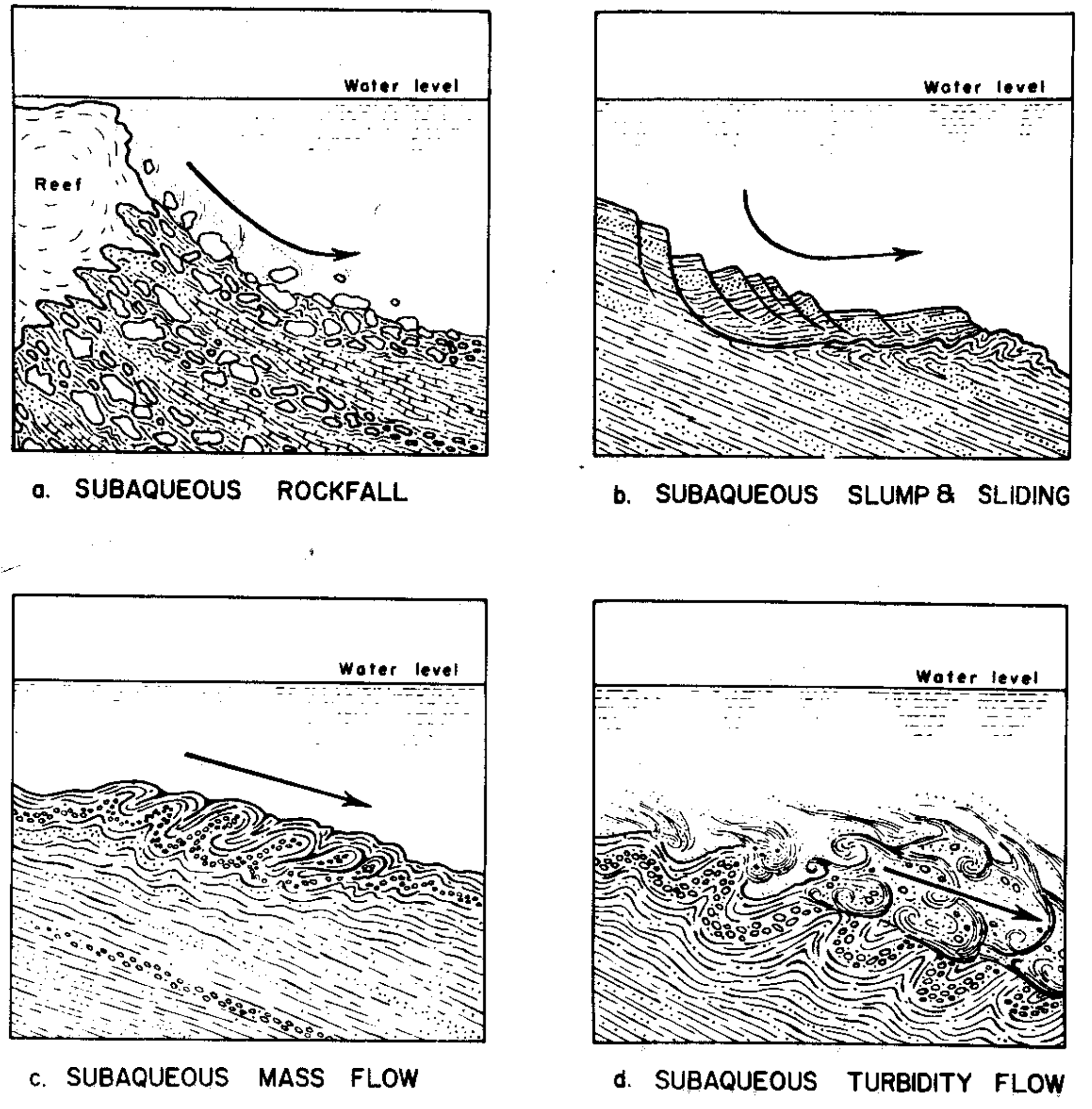
According to Terzaghi (66); known submarine slope failures can be divided into three categories: local minor slumps on steep otherwise stable slopes; vast mass movements of short duration on steep or gentle slopes and intermittent local slumping on gentle slopes. Although Terzaghi apparently believed turbidity currents to be impossible, experimental work of Kuenen (34) showed that such dilution by mixing due to turbulence actually can occur. Plapp and Mitchell (45) have also presented a theoretical analysis of turbidity currents.

The classification systems for soil movements on the ocean floor discussed above are limited in scope and do not approach the sophistication with which geologists have classified terrestrial soil movement. For example, Sharpe (53) classified soil movement into four major categories, each with many sub-classifications. Sharpe's major categories consist of slow flowage, rapid flowage, sliding, and subsidence, and he considers materials to be classified according to their water content. The complete classification system is shown in Figure 12.

A more unusual classification system for landslides was developed recently by Takada (62). He divides soil mass movement into four types, namely: (1) Slide of fluid type; (2) Layer slide type; (3) Protruding slide type of an intermediate layer; and (4) Multi-slide type. The fluid slide involves sliding such that the rate of movement of the soil mass is fastest at the ground surface and tends to zero in the vicinity of the bedrock. A layer slide is the most common type of slide. It is characterized by the movement of a coherent layer of soil. The protrusion of an intermediate layer, as its name implies, is a slide movement during which only a soil layer between the ground surface and the bedrock begins to move. This may be considered the cause of a depression or sinking zone within a landslide area. A multi-layer slide involves movement in which two or more different layers take part in slide movements at different times and at different rates.

(c) Occurrence of Sliding, Flow, and Creep

There exists a large body of literature, written primarily by geologists and oceanographers, concerning the mechanics of various types of submarine sliding. A summary has been compiled by Morgenstern (43) who paid particular attention to earthquake-initiated submarine slumping. The results of Morgenstern's studies are shown in Tables 3 and 4. Morgenstern noted that slumping has been observed or has been inferred to have occurred on a wide range of slope inclinations. One of the earliest indications of this possibility is a description by Heim (25) of a slide that moved into Lake Zug, Switzerland, in 1887. The slope had an inclination of 2.5°. From core studies in the Black Sea, Archanguelsky (1) described slumping on slope angles of 1 to 3°.



Major categories of subaqueous gravity movements modified after Varnes (1958); a. represents elastic behavior; b. both elastic and plastic; c. plastic flow; d. viscous fluid flow.

After Dott (9)

Fig. 11 Subaqueous Gravity Movements

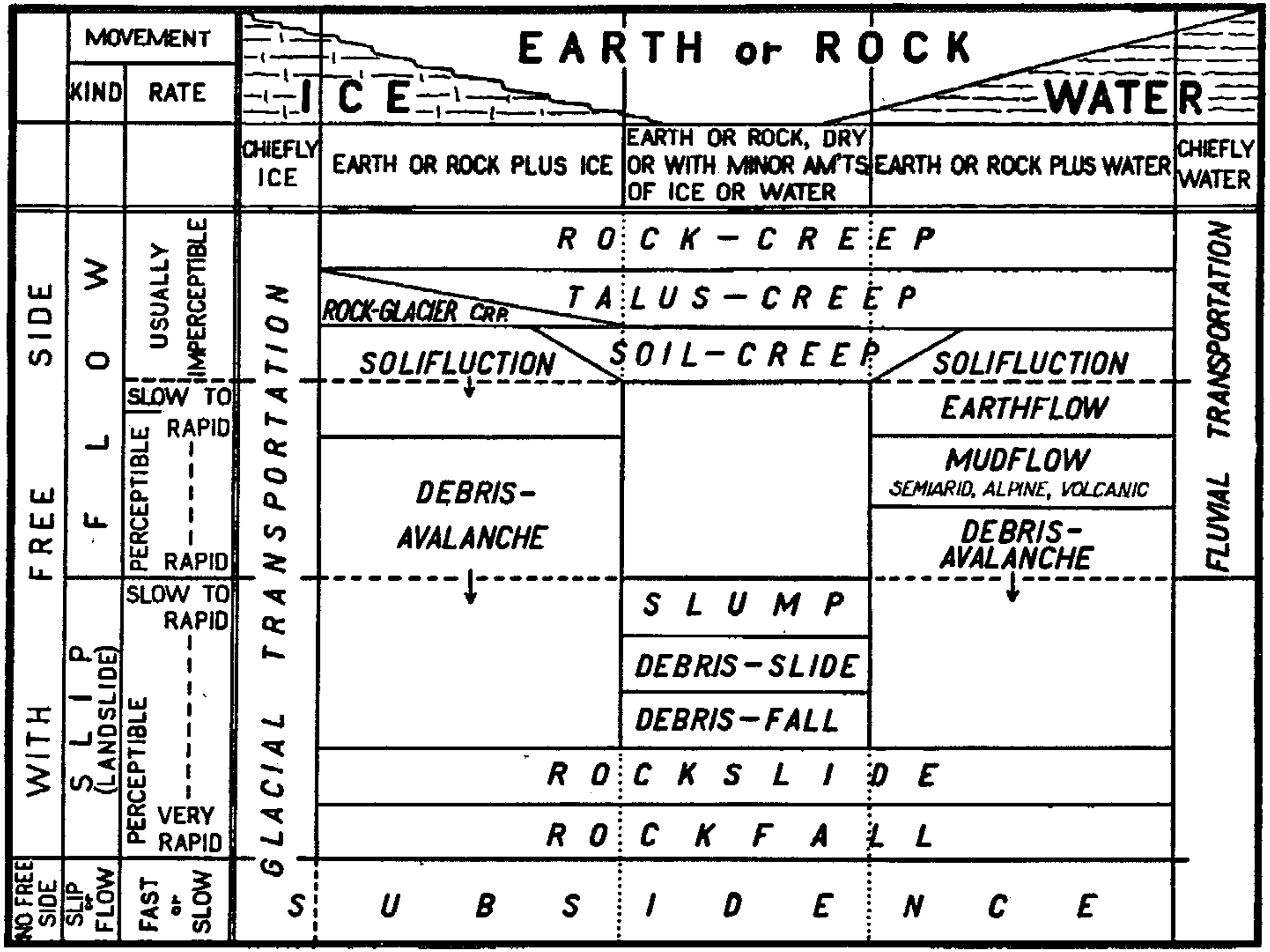


Fig. 12 Classification of Landslides

After Sharpe (53)

Slumping on inclinations of 1° has been suggested by Shepard (55) to account for the delta-front valleys associated with the Mississippi River. Studies in New Zealand by Grant-Mackie and Lowry (17) suggest movement may have taken place on slopes of less than half a degree. Moore (42), on the other hand, seriously questions the possibility of slumping on such gentle slopes. In particular, Moore doubts the existence of sliding on the deep sea floor and normal open continental shelf. He observes that the amount of sliding on the continental slope will vary with the type of sediment, its rate of accumulation, and the topographic features in the regions in which it is being deposited.

Moore's investigations revealed that the kind and rate of sedimentation required to produce unstable slopes occurs only in specialized areas of relatively rapid accumulation, such as river deltas and submarine canyons. He further states that the normal open shelf and the deep-sea are believed to be nearly free of sliding. Moore himself points out, however, that he did not consider the mechanism of progressive sliding which might explain failure of relatively gentle slopes.

The effects of submarine slumping have been observed in various geological strata in many locations. Subaqueous slumps on slopes inclined at steeper angles than those mentioned above have been discussed by Terzaghi (66) and Koppejan, Van Wamelen and Weinberg (32). These include a slope failure in clean sands and gravel in Howe Sound, British Columbia, which probably had an inclination greater than 28°, and the slides which occurred in fine sand along the coast of Zeeland. Original slope angles of 15° are known to exist in the latter case.

One particular geological feature which is generally linked with submarine slumps and turbidity currents and is worthy of particular attention is submarine canyons. Shepard (54, 57, 59, 60) first hypothesized the role of submarine landslides in the modification of submarine valleys. Although the geologic origin of these submarine features is still a debated question, there seems to be little doubt that some type of soil movement, whether it be landsliding or turbidity currents or a combination of the two, plays an important role in the formation and modification of submarine valleys.

Recently, Shepard and Buffington (58) completed a detailed geologic investigation of the La Jolla Submarine Fan Valley. This study revealed terrace-type features in the central portion of the valley. A landslide origin for these terraces seems probable. The importance of sliding in the formation of terrace features was recognized by Sharpe (53) who pointed out four different mechanisms by which terracettes may form (see Figure 13).

Turbidity currents are believed to play two roles in the formation of submarine valleys. First, Heezen (21) believes that these submarine canyons are a direct result of erosion by turbidity currents, and, secondly, Buffington (5) conjectures that the existence of submarine natural levees along the edges of submarine canyons are the products of turbidity currents.

TABLE 3

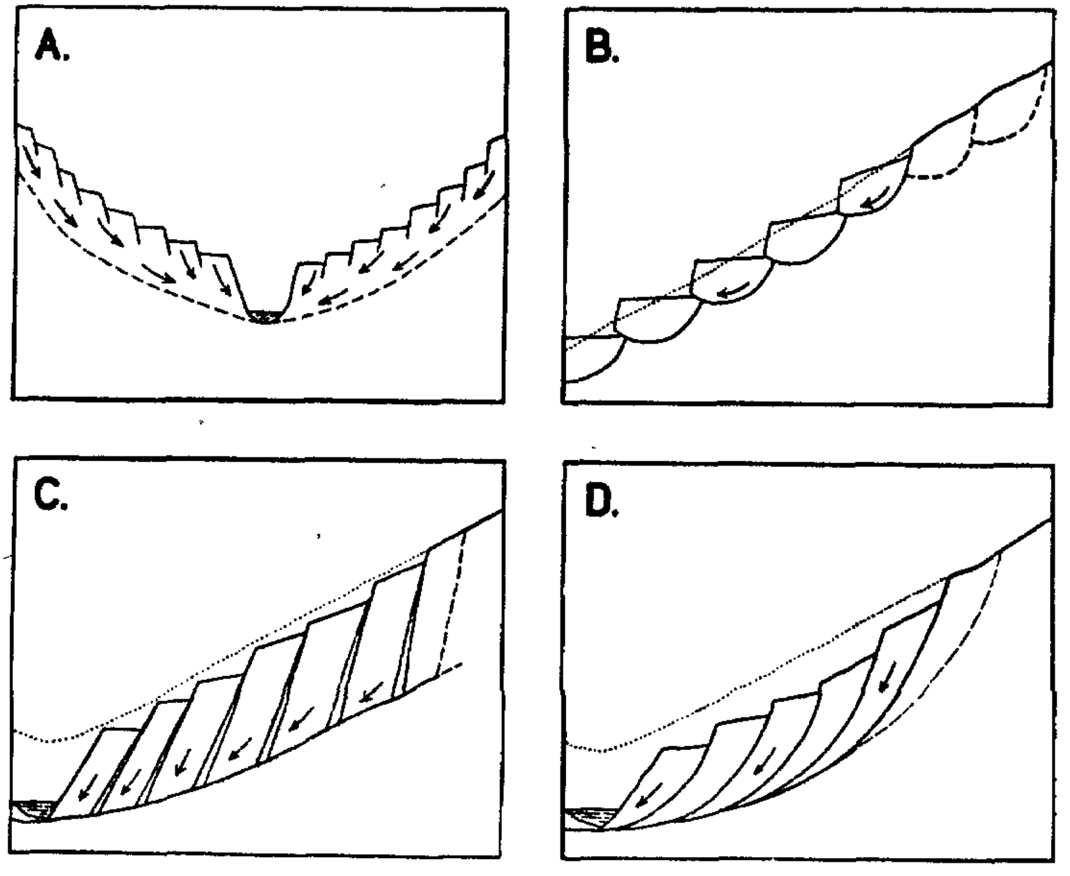
SOME SLUMPS CAUSED BY EARTHQUAKES

Location and Date	Slope Degrees	Magnitude M	Focal Depth km	Within Epicentral Region	Reference
Grand Banks, 1929	3.5	7.2	Shallow	Yes	Heezen and Ewing(1952)
Orleansville, 1954	4-20	6.7	7	No	Heezen and Ewing(1955)
Strait of Messina, 1908	4	7.5	8	Yes	Ryan and Heezen(1965)
Suva, 1953	3	6.75	60	Yes	Houtz (1962)
Chile, 1922	6	8.3	Shallow	No	Gutenberg (1939)
Valdez, 1964	6	8.5	Shallow	Yes	Coulter and Migliaccio(1966)
Aegean Archipelago, July 9, 1956	10	7.5	15	No	Ambraseys (1960) and Admiralty Chart No. 1866 (1951), Royal Hellenic Navy

TABLE 4

VOLUMES OF SUBMARINE SLUMPS

Location	Volume m^3
Magdalena River Delta, Colombia	3×10^8
Mississippi River Delta, USA	4×10^7
Suva, Fiji	1.5×10^8
Valdez, Alaska	7.5×10^7
Folla Fjord, Norway	3×10^5
Orkdals Fjord, Norway	10^7
Sagami Wan, Japan	7×10^{10}



Origin of *terracettes.* (A) Conditions particularly favorable for the formation of terracettes; valley in unconsolidated material, fairly steep sides with toes of slopes actively undercut by deeply trenched stream. (B) Terracettes formed by rotation of superficial blocks as pictured by Ødum. (C) Terracettes caused by slippage of blocks on major slip plane or partially fluent zone. (D) Terracettes caused by typical slump movements resulting from slippage on deep-seated curved surfaces.

Fig. 13 Origin of Terracettes

After Sharpe (53)

Another active investigator in this field is Moore (39), who pointed out that nearly horizontal beds of stiff cohesive clays, alternating with cohesive silts, crop out along a steep wall of the channel in the La Jolla Fan Valley in water depths of about 3000 feet. These exposures are believed to be the result of bilateral channel erosion by turbidity currents. This should result in periodic development of new channel routes. Once current erosion forms these steep walls of outcropping sediment layers, the sliding of clay blocks, local slumping, and the actions of benthic organisms modify them. Turbidity currents which have been active recently are believed to be on a much smaller scale than those responsible for the development of the large morphological features of the overall valley.

The soil movements and the resulting effects upon the ocean floor after the 1929 Grand Banks Earthquake have received a great deal of attention because of the successive breaking of telegraph cables. Heezen and Ewing (22), Heezen and Drake (23), and Heezen and Erickson (24) surmise that these cables were broken by a turbidity current originating as a slump on the continental slope, continental rise, and ocean basin floor, and continuing far out on the abyssal plain, well over 450 miles from the continental shelf. Kuenen (33) estimated that the initial slide was approximately 50 meters thick and that the turbidity current achieved a maximum velocity of approximately 85 knots. At one point the turbidity current is estimated to have had a thickness of 270 meters and a length of 5-1/2 kilometers. The thickness of the deposited bed as a result of the turbidity flow is in the order of one meter and spread over an area of perhaps 100,000 square miles.

Turbidity currents have also been studied in other areas by Houtz and Wellman (28) off the Fiji Islands; by Holtedahl (27) in the Hardanger Fjord, Norway; by Gould (16) in Lake Mead, and by Gorsline and Emery (15) off the southern California coast. It is not clear how a slide develops into a turbidity current. Most investigators (Kuenen (34), Middleton (35, 36)) have studied the behavior of turbidity currents which were artificially generated prior to release.

Less attention seems to have been paid to the problem of the transition process by which a slide becomes turbulent and entrains enough water to become a turbidity current. A block slide in an unstable material may trap water under its leading edge, or toe and thereby initiate a mixing process. A flow slide (and possibly a block slide) may, alternatively, develop enough velocity to generate instability at the soil-water interface, while at the same time the disturbance causes the unstable material to become weakened.

Waves would then develop at the interface, and these would further deteriorate the soil material properties, so that the flow velocity would increase. At some velocities the waves could break, to initiate turbulent mixing of the soil and water. If this is indeed a possible mechanism, it might be difficult to develop it in a laboratory experiment, by reasons of scale.

However, to the authors' knowledge, there have been no attempts to reproduce, in the laboratory or through various experiments in the ocean itself, the mechanism by which turbidity currents are initiated from a previously stable sea-floor slope. In an attempt to shed some light on this problem, and to examine one theory of slide initiation, a series of experiments was conducted by the authors in a tilting flume. The apparatus was sufficiently small (approximately 6 ft x 1 ft x 6 inches in size) that one end could be lifted slightly and dropped to provide an artificial seismic shock to the system. The submerged soil slope was arranged in two layers, a thin sandy-silt layer over a thick layer of loose coarse sand. The thicknesses of these two layers were varied from one experiment to the next, in the range of a few tenths of an inch to an inch or so. The reasoning behind the experiments was that the loose underlying sand, when jolted, would liquefy. It would then settle out relatively rapidly, leaving a thin layer of water sufficient to suspend the overlying fine-grained silt layer. This would allow the silt to accelerate down the slope. The slopes employed were limited by the apparatus to angles of about 10°.

Although the postulated phenomenon did occur, the movement was slow and short-lived; in no case was a turbidity current generated. It was concluded that due to insufficient pore pressures caused by the restrictions of the size of the apparatus and the escape of pore pressure around the boundaries of the silt layer, a turbidity current could not be generated at this scale of experiment. It seems likely, however, that criteria for a more suitable test of the hypothesis can now be developed.

More easily observed are submarine landslides which take place in near off-shore areas. The harbor facilities at Valdez, Alaska were lost in a large submarine slide which developed on a much steeper delta slope during the 1964 Alaskan earthquake (67). Boutsma and Horvat (4) have conducted extensive studies in the Rotterdam Harbor area where seepage forces are a particularly difficult problem in the deepening of harbor channels. Other instances in which seepage forces contributed to slides in near off-shore areas are mentioned by Terzaghi (66).

(d) Classification of Submarine Landslide Initiation Mechanisms

There are many mechanisms which can induce submarine landslides. Table 5 presents a list of these mechanisms divided into two classes, natural and man-made or man-influenced. The natural mechanisms are induced solely through the processes of nature while the man-made mechanisms are usually a direct result of engineering projects constructed by man.

According to Morgenstern (43), the most common mechanism is probably oversteepening of the slope. This may occur due to deposition or possible crustal tilting associated with tectonic movement. Erosion due to water currents or turbidity currents may cause local oversteepening, leading to progressive failure. Slumping is particularly common at the head of

TABLE 5

LANDSLIDE MECHANISMS

Natural Mechanisms	Man-Made Mechanisms
1. Steepening of Slopes	1. Steepening of Slopes
a) Deposition of Material	a) Filling or cutting (dredging)
b) Erosion of Canyons, Submarine Valleys	b) Changed flow regime (harbor works)
c) Tectonic Movements	c) Construction loads
2. Pore or Water Pressures	2. Pore or Water Pressures
a) Seepage	a) Salt water intrusion barriers
b) Ocean waves	b) Changes in seepage pattern
c) Tidal changes	
3. Earthquake Vibrations	3. Vibrations
	a) Pile driving
4. Creep Leading to Failure or Not	b) Blasting
5. Organic Matter Decomposition	

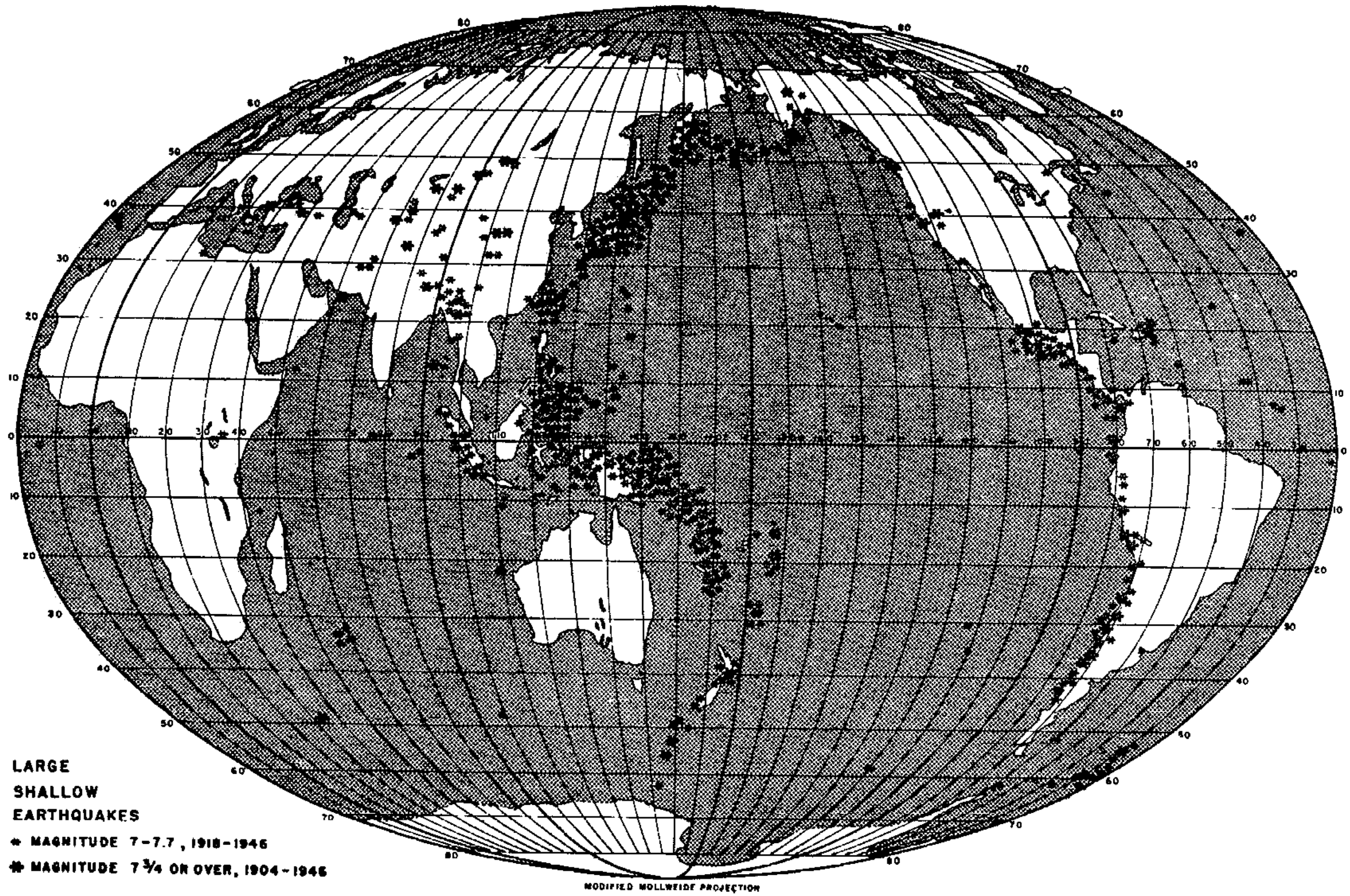


Fig. 14(a) World Map of Shallow Earthquakes

After Gutenberg (18)

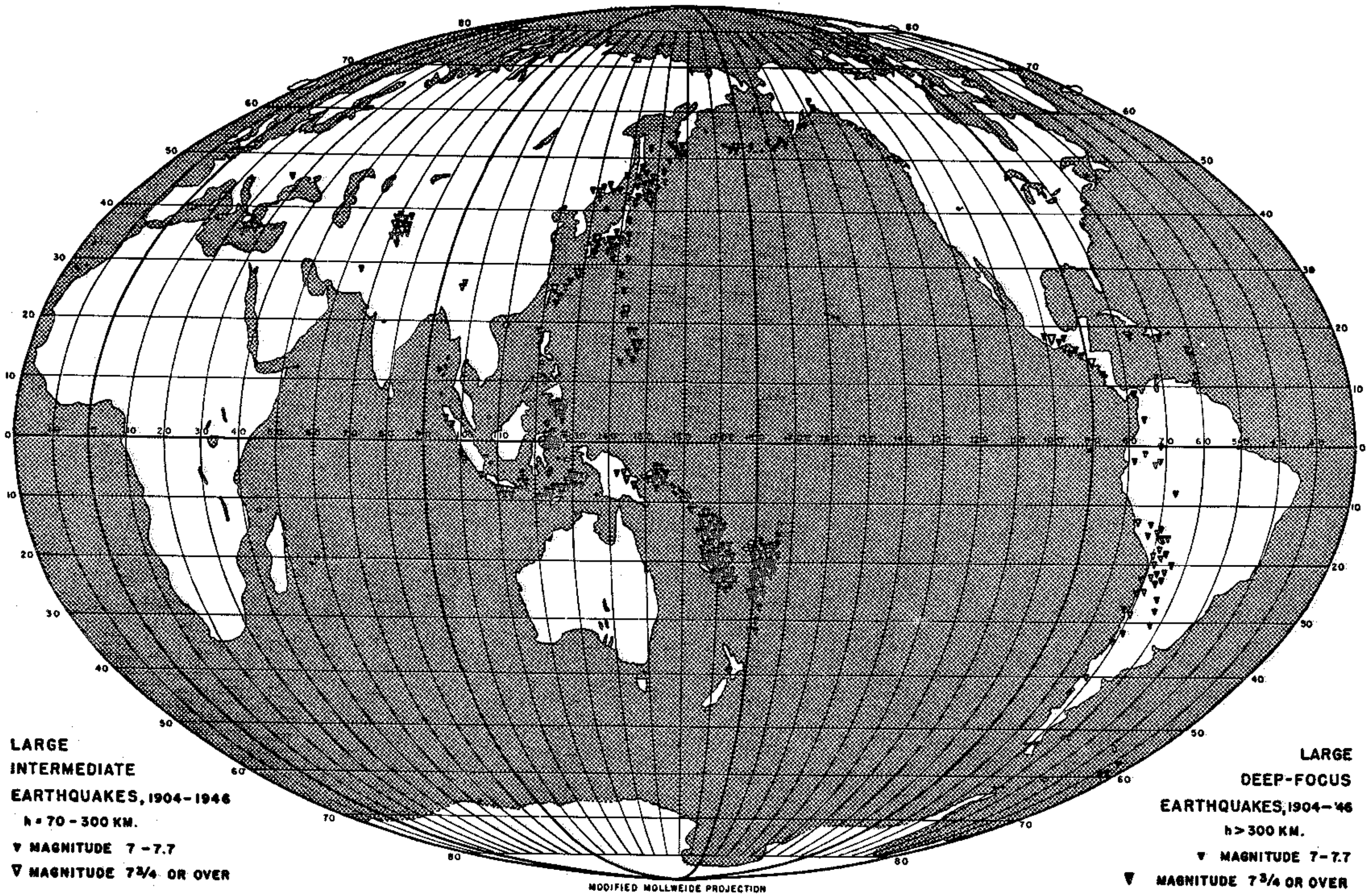


Fig. 14(b) World Map of Deep Earthquakes

After Gutenberg (18)

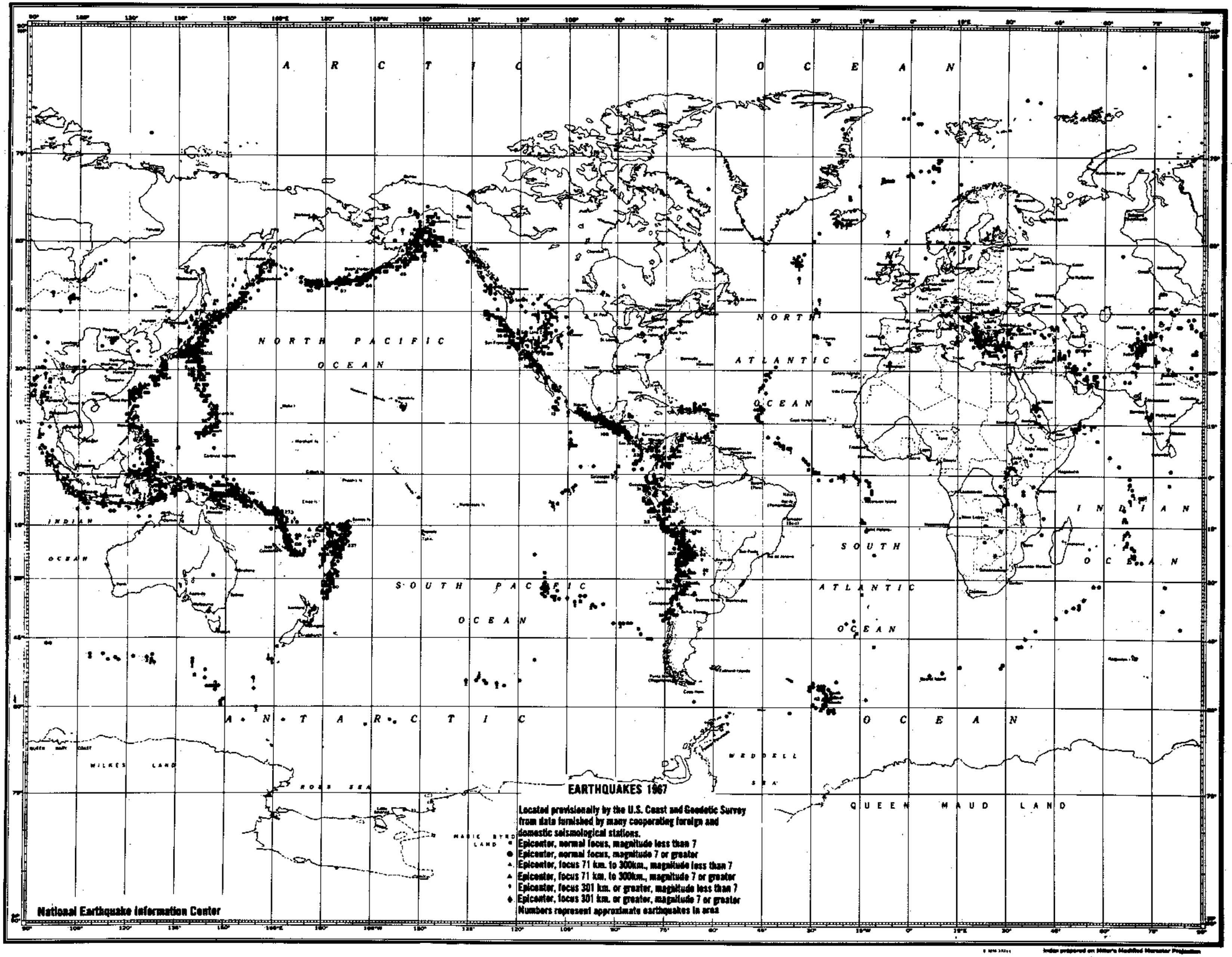
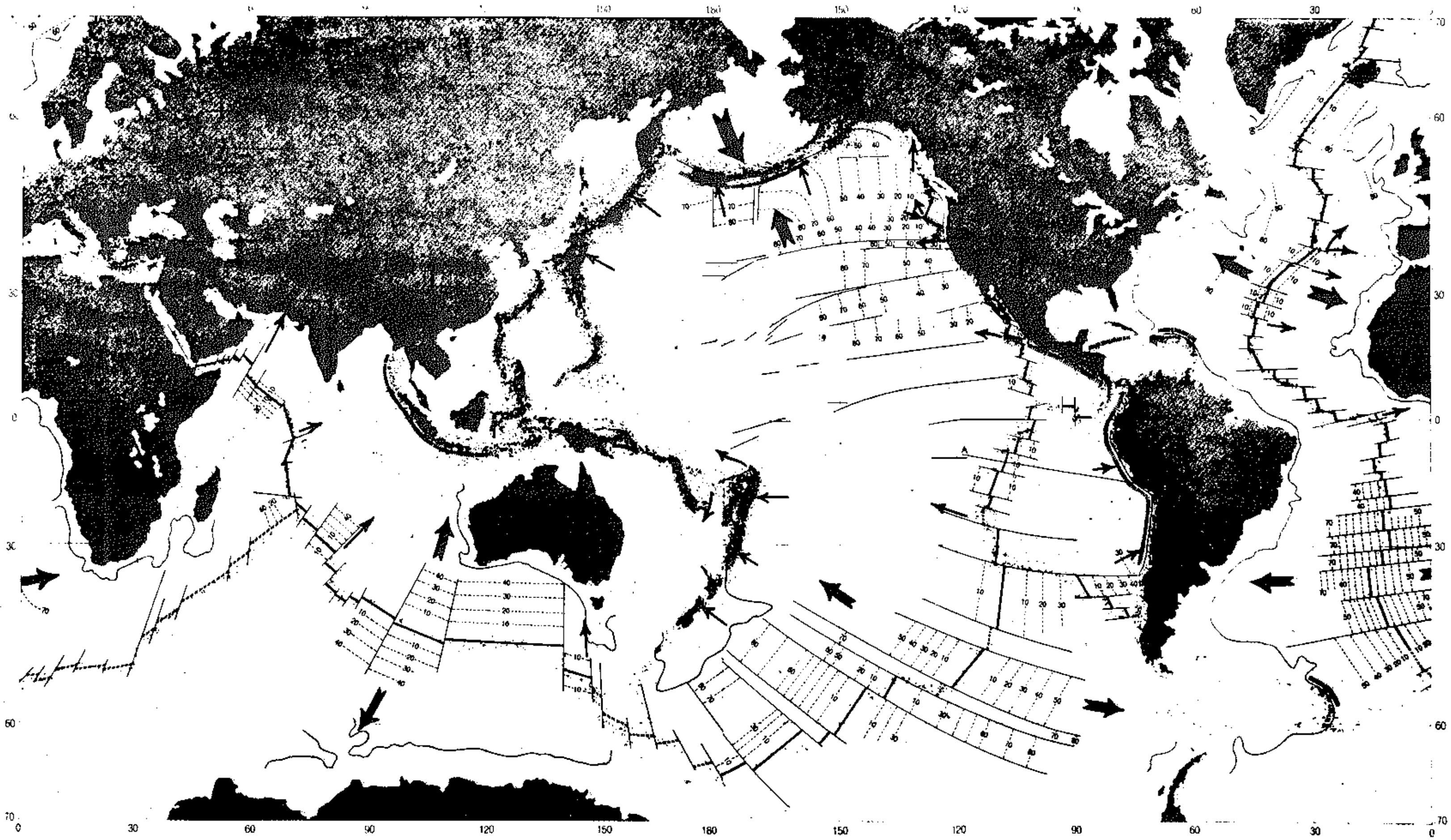


Fig. 15(a) World Seismicity, 1967.



World seismicity and sea floor spreading. Seismic activity (the dots represent the epicenters of some 29,000 earthquakes that occurred in the interval 1961–1967) is concentrated in narrow, continuous belts separating large, relatively stable blocks. Significant activity does occur, however, within the blocks, as shown, for example, by the epicenters in the central and western United States.

Plates of the lithosphere, the thin, near-surface layer of strength, are bounded by the active seismic zones along the ocean ridges, the arcs, and the major strike-slip faults. Magnetic anomalies (numbered, dotted lines) provide information on the rate of divergence of adjoining plates, suggesting that the plates move, relative to an adjoining plate, in the general direction of the large arrows. Small arrows along fault lines perpendicular to the ocean ridges represent a sampling of slip vectors from earthquake-mechanism studies and indicate the horizontal component of the direction of motion of the block on which the arrow is drawn relative to that of the adjoining block, confirming the nature of the faulting required for spreading of the sea floor.

Most seismic activity occurs in zones of compression, where small arrows head toward arcuate-type structures. Note the similarity of the movement implied by the two types of arrows in the regions of compression. (Compression and extension along boundaries of lithospheric blocks after Le Pichon, 1968; earthquake source vector data from Isaaks, Oliver, and Sykes, 1968; epicenters located by ESSA—from Barazangi and Dorman, 1969; magnetic data and other map information based on Heirtzler, 1968).

Fig. 15(b) World Seismicity and Sea-Floor Spreading

From Bullard (80)

submarine canyons and in the vicinity of mouths of large rivers. These are both environments of rapid deposition.

The sediment displaced by a slide in shallow water is deposited at lower elevations and acts as a sudden load on a slope in deeper water, thereby inducing a subsequent failure. Slides in shallow water may be triggered by erosion or rapid drawdown. The latter phenomenon can be particularly important in areas which experience high spring tides.

The occurrence of earthquakes can induce submarine landslides through inertial loading and possibly spontaneous liquefaction of metastable soils. Liquefaction will be given particular attention later in this study. Table 3 refers particularly to slides triggered by earthquake mechanisms. It is important to note that many seismically active areas of the earth are in or immediately adjacent to the oceans. Data compiled by Gutenberg and Richter (18), shown in Figures 14(a) and 14(b) reveal the high seismicity of the periphery of the Pacific Ocean. More recent work has uncovered other oceanic regions which are seismically active, including the mid-Atlantic ridge and regions of the Indian Ocean. A world seismicity map for the year 1967 is shown in Figure 15(a). In these areas, earthquakes most likely play an important role in the initiation and frequency of submarine sliding.

Another tectonic action which may play a role in the formation of large ocean floor slides is sea-floor spreading. A great deal has been said about this phenomenon in relationship to continental drift, (see Heirtzler (26) and Bullard (80)), but its relationship to slide formation remains unexplored.

An inter-relationship between zones of high seismicity and regions of ocean-floor spreading has been observed. This is graphically illustrated in Figure 15(b) which combines world seismicity with lines and vectors of ocean-floor spreading. These observations are based upon magnetic anomalies in ocean floor bedrock. The existence of ocean-floor spreading, therefore, can be determined without the aid of historical seismic records. If the magnetic anomalies characteristic of ocean floor spreading are observed in an area of historically low seismicity, it may be justifiably concluded that the historical seismic record is inadequate. For engineering purposes, regions of ocean-floor spreading should be considered highly seismic. Other long-term tectonic movements, such as crustal tilting, probably do not play a significant role in the formation of submarine landslides. However, the possibility of such mechanisms cannot be totally overlooked and is, in fact, thought to be the cause of ancient sea-floor slumping off New Zealand (17). Some peripheral tilting may accompany the settlement of basins in which sedimentation is occurring.

A mechanism which is often overlooked is that associated with forces involved in the generation of gas from the decomposition of the plant material. Dill (8) has found that this mechanism can lead to significant creep movements in canyonhead areas. This is attributed to the fact that organic material tends to accumulate in the head of a canyon. Finally a mechanism which was suggested by Saito and Uezawa (50) is that of creep failure. It is thought that a slope can withstand a given rate of creep for only a limited time after which failure will occur. This critical time is called the rupture life due to creep and has been investigated empirically on terrestrial slopes. Saito (51) has shown in several cases that the time of occurrence of a slope failure can be reliably predicted when this mechanism exists, and the pre-failure movements of the slope are carefully measured.

Henkel (74) has proposed that sliding of soft underconsolidated deltaic sediments may be initiated by the passage of large ocean waves. He theorizes that oscillating wave forces shear underlying sediments in a cyclic fashion. This gives rise to remolding of the soil with an increase in pore pressure and a decrease in strength. If the process causes sufficient loss of strength, mass movement will occur. Henkel's analysis indicates that shear failure in soft sediments can be induced by this mechanism in water depths up to about 400 feet. At present there is only limited evidence to support this theory. This question will only be answered when data on the actual bottom pressure in the area of interest is available and frequent bottom surveys can be made. Cartwright and others (69, 70, 71, 72, 73) have made pressure observations on the sea bottom with various types of sensors. However, these instruments have only been used to make measurements of oceanic tides, and have not been utilized in the investigation of the slope stability problem.

Because man has only occupied and worked in near off-shore areas, the man-made mechanisms listed are generally associated with shoreline slides. Fills and cuts induce slides by oversteepening. Pile driving and blasting can induce spontaneous liquefaction in much the same way as does an earthquake. Channel modifications have been observed to upset the pattern of off-shore sediment deposition, indirectly causing oversteepening and subsequently submarine landslides. This mechanism induced a large submarine slope failure on the north shore of Howe Sound in southern British Columbia, as discussed by Terzaghi (66). Harbor areas are a special problem in themselves and are often subject to adverse seepage conditions. Boutsma and Horvat (4) have described problems associated with channel construction in the Rotterdam Harbor area. Monney (37) has discussed the seepage of groundwater in aquifers toward the sea. This will give rise to seepage forces (or pore pressures) which will have an unfavorable effect on the stability of immediately off-shore slopes. In some instances, injection wells have been placed along the shore to protect the inland groundwater against seawater intrusion. This injection process may result in off-shore slope instabilities.

(e) The Role of Unstable Soils in Submarine Slope Stability Analysis

The stability of a sedimentary deposit on a given slope depends basically on the shear strength of the deposit and the rate of increase of this strength with the depth of burial. A slope fails as soon as the average shearing stress along the potential surface of sliding becomes equal to the average shearing resistance along this surface. If the deformation which follows develops an increased strength in the soil (strain-hardening) the failing mass comes to rest. The material strength-deformation property is called stable. If, on the other hand, deformation or distortion of the soil diminishes its shearing resistance, it is said to exhibit an unstable behavior, and the initial movement accelerates to form a rapidly flowing mass of soil. A particularly dangerous situation therefore exists when soil is subject to a rapid loss of shear strength with deformation.

As an extreme case, certain soils lose almost their entire strength upon distortion or disturbance. They are considered to possess a metastable or unstable structure and the process of loss of shear strength is called liquefaction.

A number of investigators, including Morgenstern (43), noted that liquefaction occurs most commonly in saturated loose sands and silts whose grain structure, when stressed, collapses to a lower density, and is therefore, for a short time, incapable of sustaining any applied load. Since such a material has essentially zero shearing strength in the collapsing state, the mass of soil and water becomes a fluid with a unit weight equal to that of the original soil. Diminution of the pore water pressure and settling of the soil grains to form a new stable structure proceed simultaneously. If the seepage velocity due to the resulting hydraulic gradient is sufficiently high, solid particles can be carried with the flow. As this mechanism suggests, unstable soils will show a very large rate of volume decrease during drained shearing tests in the laboratory. Some typical stress-strain relations for stable and unstable soils are shown in Figure 16.

Terzaghi (66) has discussed the microscopic origin of unstable soils. In a typical soil structure shown in Figure 17, a sinking particle of buoyant weight Q arrives at the surface of the sediment and tends to roll or slide into the most stable position. This tendency is resisted by the adhesion which acts at the point of first contact between the particle and the sediment. The resisting couple, M_c, produced by adhesion is independent of the grain size, whereas the overturning couple, M_q, varies in proportion to the fourth power of the diameter of the grain. Therefore, the probability that the particles will assume stable multi-point contact positions decreases with decreasing grain size. If the unstable soil structure should be disturbed by static- or vibration-induced shearing stresses, which occur more rapidly than the excess pore fluid can be expelled, a rearrangement of particles occurs in which they temporarily lose contact with one another to form a liquefied soil.

Terzaghi observed that as the particle size drops below about six microns the effect of vibrations on the soil decreases. This fact has been interpreted to indicate that adhesion forces become more important in the soils' behavior, since the number of contact points (each with the size-independent adhesive force) per unit volume increases with decreasing particle size. If the grain size further decreases, the effect of the vibrations becomes less and less, and the sediment exhibits increasing cohesiveness. Test results show that the sediments most sensitive to shock or vibration are those with a grain size between 20 and 6 microns, or sediments with a high percentage of grains of

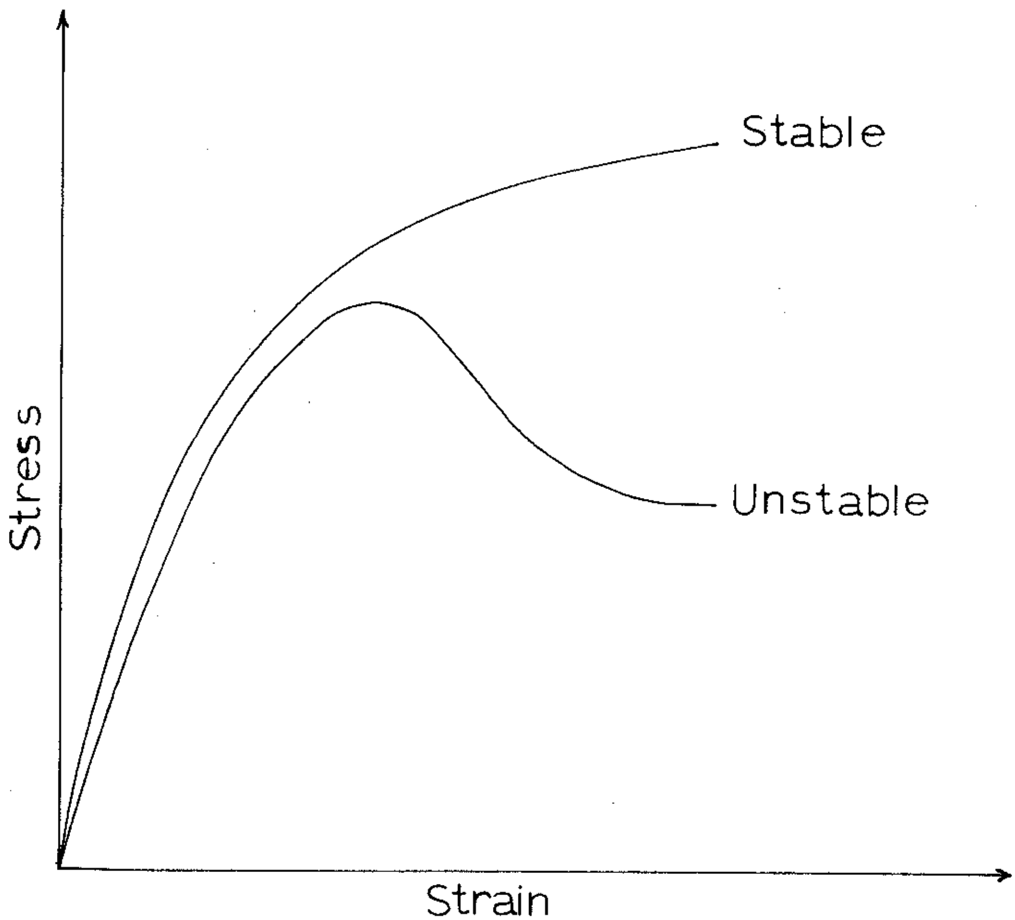


FIG. 16

Stress-Strain relations for stable and unstable soils

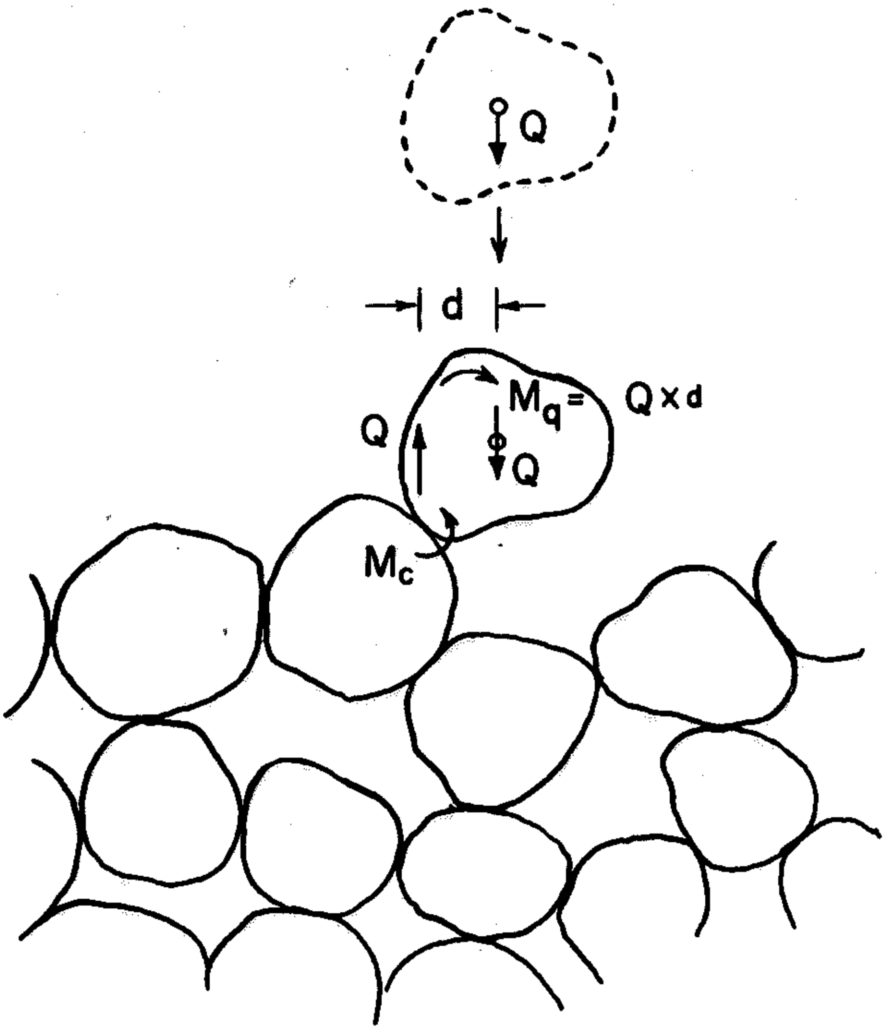
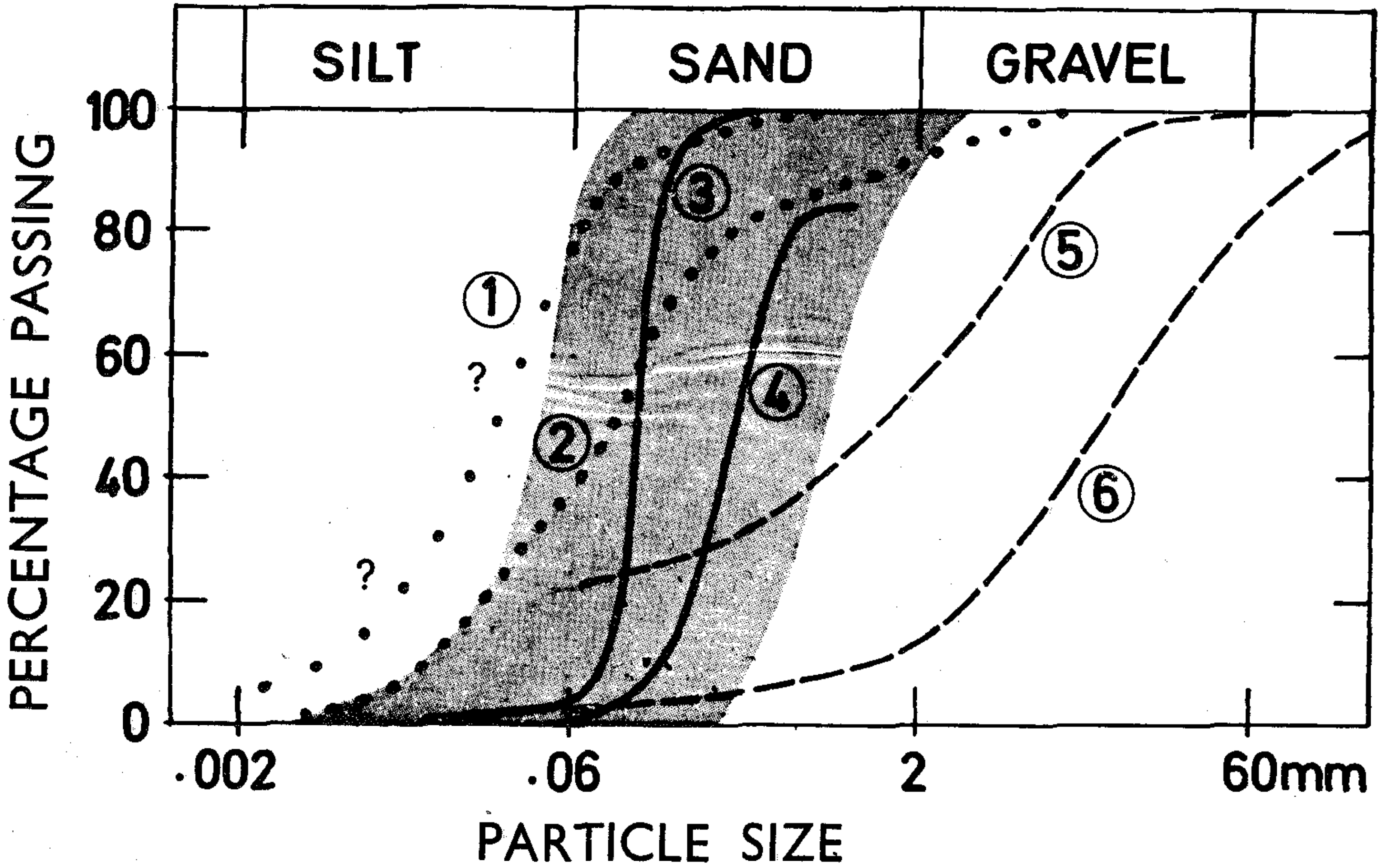


FIG. 17

Origin of metastable structure. Adhesion P at point of first contact between sinking particle and surface of sediment prevents particle from rolling into stable position.

After Terzaghi (66)



Plot of grain size distributions for materials prone to flow slides or liquefaction.

Fig. 18 - Grain-Size Distributions for Liquefiable Soils

After Hutchinson (29)

that size. According to soil mechanics nomenclature, grains of this size are in the silt-size range.

Universal agreement does not exist as to the range of grain-size distribution for unstable soils. The discussion above indicates that liquefaction will only occur in fine-grained soils, but not all the evidence supports this hypothesis. Hutchinson (29) has summarized some of the available data on grain-size distributions of liquefiable soils (see Figure 18). The shaded region on the diagram represents the most commonly accepted limits for grain-size distributions of metastable soils. The lower limit is that of Glossop and Skempton (14) and the upper is that suggested by Watanabe (68). Both curves 3 and 4 represent submarine slides on the Dutch coast at Zeeland and Ravenswaay as reported by Koppejan (32). Curves 1 and 2 represent liquefaction failures of fly ash heaps at Jupille, Belgium and South Wales, England; as reported respectively by de Beer (6) and Evans (11). All of these first four curves fall within or close to the shaded region.

Curves 5 and 6 represent the upper and lower limits of grain size in a catastrophic slide at Aberfan, South Wales. The soil, which consisted primarily of shale and mudstone fragments, has a grain-size distribution significantly coarser than the upper limit of the shaded region. Hutchinson cites other flow slides in similarly coarse material to support his belief that the conventional grain-size distribution limits are inadequate.

A criterion for unstable soils suggested by Terzaghi and Peck (65) and described by Morgenstern (43) as applicable to submarine landslides is the following:

Effective size, $D_{10} < 0.1$ mm

Uniformity coefficient, $U < 5$

This criterion excludes the type of soil which flowed at Aberfan as well as the fly ash material associated with curve 2 of Figure 18.

Aberfan:	D_{10}		0.1 mm + → 0.8 max
	U	$\approx$	18
Welsh fly ash	D_{10}	$\approx$	0.015 mm
	U	$\approx$	10

On this basis Hutchinson asserts that a uniformity coefficient less than 5 is not a criterion for liquefaction and that the effective size of 0.1mm may be exceeded by some unstable soils.

The dramatic effect of liquefaction on shear strength is shown in Figure 19, which represents a fine-grained cohesionless sediment with a metastable structure. Initially the excess hydrostatic pressure u_w is equal to zero. Therefore, the shearing resistance, like the overburden pressure, increases in simple proportion to depth as indicated by the straight line marked s_o. However, as soon as the soil particle structure collapses, as a result of an earthquake or some other disturbance, the excess hydrostatic pressure u_w increases almost instantaneously from zero to a value close to the total overburden pressure. The corresponding degree of consolidation U

where
$$U = 1 - \frac{u_w}{\gamma_s z},$$

γ_s = submerged unit weight of soil decreases from unity to a value close to zero and the shearing resistance also almost vanishes. As a result, the unstable deposit temporarily assumes the character of a liquid with a low viscosity. If the surface of the deposit is sloping, the material, soil and water, flows down slope. However, in a time, depending on the grain size, the soil settles out to form a solid once more, and the material usually comes to rest within a relatively short distance from its original location. Considering the phenomenon as a consolidation process of excess pore pressure dissipation in a layer of fine sand or silt a few tens of feet thick, it will take in the order of minutes for the soil mass to become solid once more. The resulting deposit of the slide material has the shape of an alluvial fan. On the basis of this short duration of liquefaction, Terzaghi (66) has argued against the development of turbidity currents by this means. However, Morgenstern (43) points out that, after the Alaska earthquake of 1964, sand spouting occurred for a duration of five to ten minutes, and it is likely that excess pore pressures existed within the sediment for longer than that. It is also common experience that sediments that have been liquefied during an earthquake remain extremely soft for some time.

It would seem that a critical question that arises with respect to the generation of turbidity currents from a liquefied soil mass with a sloping surface refers to the velocity that the mass of soil can attain underwater before the soil solidifies. For the case of a less dense, less viscous liquid moving over a more dense, more viscous liquid, Keulegan (79) has established a stability parameter θ which determines the conditions under which the interface becomes unstable. This parameter is given by the equation

$$\theta = \frac{\left[\nu_2 g \, \Delta\rho/\rho_1\right]^{1/3}}{v}$$

where ν_2 is the kinematic viscosity of the lower liquid, ρ_1 is the density of the upper liquid, $\Delta\rho$ is the density difference between the two liquids, g is gravity and v is the relative velocity.

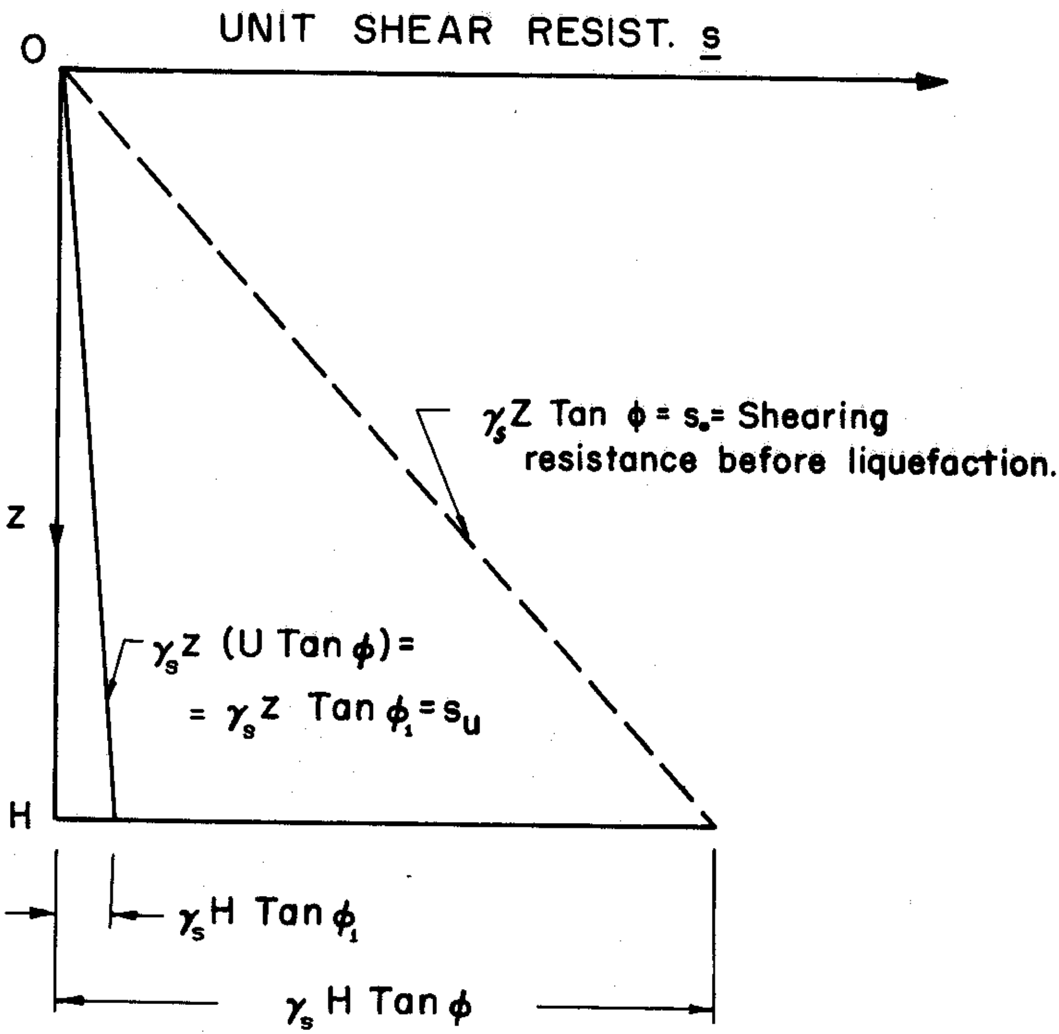
The flow is stable when the value of θ is greater than 0.2, unstable when it is less. When instability occurs, waves form at the interface between the liquids and interfacial turbulent mixing begins. Thus, if the velocity of a liquefied mass of soil on the ocean floor exceeds the velocity at which the soil/water interface is stable, the surface of the flow will become turbulent and mixing will occur. The liquefied soil will be converted into a turbidity current. The velocity of this current which is less dense than the original soil mass will be less than the steady-state velocity of the liquefied soil. Whether the turbidity current continues to flow for some distance, or settles out itself depends on whether its velocity exceeds the stability limit. It is possible that, in the case of turbidity currents which travel long distances, the slope, the density of the current, its velocity and the turbulent mixing which occur, reach equilibrium conditions. Ultimately the current must stop and yield its load of sediment when the slope becomes too flat to sustain it.

Inserting the appropriate values of various terms in the stability equation (the viscosity of a liquefied soil can only be estimated) leads to the tentative conclusion that the surface of a typical flow would become unstable at velocities exceeding a few feet per second. Morgenstern (43) has attempted to calculate the velocity attained by a soil mass liquefying on a slope, and it appears that unstable velocities can be reached. Since model tests are carried out on the same materials as those involved in the natural phenomena, the same velocity holds at which instability occurs. However, in the model, the dimensions are such that liquefaction cannot occur for long enough for these velocities to be attained, unless a special selection of materials can be made.

A phenomenon which is closely related to liquefaction is underconsolidation. In underconsolidated sediments excess hydrostatic pressures are caused by a lag between soil accumulation and corresponding consolidation. The extent of the excess pore pressures which develop are a function of the rate of sedimentation, the depth of the deposited material, and the coefficient of consolidation. The excess pore pressure existing at any level in the stratum subtracts from the total stress at the level to give a reduced effective stress and therefore diminished shear strength. This mechanism is discussed by both Terzaghi (66) and Morgenstern (43). Moore (40) pointed out that shear strength studies off the Mississippi delta show that the rapidly deposited fine sediments of the delta front are extremely unstable. The observed low shear strength of sediment deposits off large rivers carrying predominantly fine-grained silt and clay-sized materials can be attributed to underconsolidation. This is discussed in more detail in the next section.

II. SLOPE STABILITY ANALYSIS OF SUBMARINE SLOPES

Various mechanisms of slide formation in soils have been discussed earlier in this report. Such slides can occur in what are known as "infinite" or "finite" slopes. An infinite slope is one in which the slope continues at the same angle for an uninterrupted distance which is great compared to the thickness or depth of the sliding layer. A finite slope is terminated by a surface



Relationship between depth $\underline{z}$ and shearing resistance of fine-grained cohesionless sediment with metastable structure before ($\underline{s}_o$) and after liquefaction ($\underline{s}_u$).

Fig. 19 Shear Strength Versus Depth Before and After Liquefaction

After Terzaghi (66)

angle change at its extremities, and the potential sliding mass is roughly equidimensional in slope length and thickness.

Although the analysis of sliding in both of these classes of slope will be discussed here, particular attention will be paid to the infinite slope; the characteristics of which are more relevant to ocean floor processes both in terms of the scale of sea-bed slopes, and the lack of detailed existing knowledge of slope profiles and soil properties. On the other hand, the behavior of finite slopes is of interest in connection with construction excavation or filling activities on the ocean floor.

(a) Infinite Slope.

This case was examined by Taylor (63) who showed that the ratio of shearing stress to normal stress on a plane parallel to the surface at any depth below the surface of an infinite slope was equal to the tangent of the slope angle. Thus, the shearing stress increases linearly with depth below the surface. For the slope to be stable, the shearing strength must increase with depth at at least the same rate as the shearing stress. Stable slopes will therefore occur in all soils where the friction angle is equal to or greater than the slope angle. In a cohesionless material steepening of the slope angle beyond the angle of internal friction causes surface sliding, unravelling, and flow to develop. Sliding of a slope of cohesionless material is likely to occur as a result of some process leading to oversteepening of the slope, such as tectonic tilting or erosion of the foot of the slope. Earthquakes may also lead to slides.

In a cohesive material with an angle of friction less than the slope angle, the shearing stress on a plane parallel to the surface will always become equal to the shearing strength at a particular depth, which is referred to as the "critical" depth. The material above this depth will be unstable. If the slope is steepened, the material will be unstable at a shallower depth. When excess pore pressures are present in the soil, the shearing strength at a particular depth is reduced and the depth to the shearing plane is similarly reduced. Slope failures may be generated in cohesive materials both by the mechanisms cited above, and by depositional rates which accumulate soil to thicknesses greater than the critical value for the slope angle and the material properties. As a depositional rate is increased for a given soil, pore pressures may be developed in the mass at a rate greater than they can dissipate. The soil strength is thereby prevented from building at a rate proportional to the increasing thickness of overlying material, and failure may occur at very small slope angles. The development and maintenance of excess pore pressure depends on the rate of deposition, the coefficient of consolidation of the soil and the drainage conditions at the underlying boundary.

This problem has been examined by Morgenstern (43) based on a solution by Gibson (13). Some of Morgenstern's results are shown in Fig. 20 for a soil layer which has grown at a uniform rate of sedimentation to a depth of 49 ft (15 m).

It can be seen from the figure that for rates of sedimentation less than 0.05 cm/year all soils are essentially fully consolidated as they are deposited. Rates ten times higher than this are required to produce a substantial degree of underconsolidation in silty clays and silts. Excess pore pressures are therefore likely to be significant only at deltaic rates of deposition.

Since soils usually have friction angles, with respect to effective stresses, greater than 10°, and are generally between 20° and 35°, it follows that slope angles up to this value will be stable in all regions where slow deposition is occurring. Since, as seen earlier, abyssal slopes are very flat, sliding under fully drained (no excess pore pressures) conditions does not seem to be a mechanism of importance in deep water. It was found by Moore (42) that most sediments in the deep sea can be deposited stably on quite steep slopes to substantial thicknesses. It is implied that deposition takes place so slowly that consolidation is complete. The work of a number of investigators (31, 38, 40, 19, 47) has demonstrated that fine-grained sea floor soils possess cohesion even very close to the sea/soil interface. This has been confused in the past with a cohesion developed as a result of higher past pressures (preconsolidation) but has been observed to occur even when no previously higher pressure or overburden could have existed at the site. It has been explained by the presence of cementation of various kinds, but may be simply (especially in the cases of weaker soils) due to the interparticle attraction of clay-sized grains in a flocculating environment. In the latter case, an external pressure need not be present; the grains come closer together and adhere as a result of the interparticle attractive forces.

When consolidation tests have been carried out on these fine-grained soils from near the surface of the ocean floor, a behavior characteristic of over- or preconsolidated soils has been demonstrated, as for example by Hamilton (19). This is shown in Fig. 22(a) which consists of a conventional consolidation plot of void ratio versus the logarithm of the vertical effective stress in the consolidation apparatus. Normally the knee, or bend in the curve results from the imposition of a past consolidation pressure higher than the value existing in the soil after sampling and at the beginning of the test. This fact is frequently used in the determination of the value of the maximum past pressure. However, in the present case, another interpretation is possible. When particles settle out of the water onto the clearly-defined surface of a bed of soil, the arrangement of grains at the surface must exhibit a finite void ratio, depending on their size, mineral nature, and the nature of the environment. Even if no further grains arrive at the surface, or if the depositional rate is very low some diminution of the void ratio will occur due to small perturbations and readjustments of the grains. Thus, if the void ratio in the deposit is plotted against the increasing vertical effective stress with depth to a linear scale, it would be expected that the initial portion of the curve would appear as shown in Figure 22(b), with a finite void ratio at zero stress. (It is not the point of the present discussion to treat the detail of the shape of the curve near the origin in Figure 22(b); only the fact of the finite void ratio is pertinent.)

A curve of the form of Figure 22(b) will then, by the logarithmic distortion at low stresses, exhibit the levelling-off of Figure 22(a) with the presence of a knee similar in character to that usually arising from a loading-unloading-reloading cycle. Presumably also, at very high pressures, as shown to the right of Figure 22(b) the soil will tend to zero void ratio, and this will be represented on the semi-logarithmic curve of Figure 22(a) as another levelling-off of the recorded curve. The behavior of soils commonly encountered in soil engineering practice occurs in the range of the steeply sloping straight-line portion of Figure 22(a) and it is here that the preconsolidation effect manifests itself. Heretofore, only seldom have soils been investigated in the range of extremely low or high pressures.

It is assumed, therefore, that fine-grained soils will display an initial but small amount of cohesion, in the range of 20 to 200 psf for example in the prodelta Mississippi clays (the lower values corresponding to the higher deposition rates), and that this cohesion, or strength, will increase with increasing effective stress.

In the case where excess pore pressures are developed by rapid sedimentation on slopes, it is of value to examine the problem in more detail than seems to have been done hitherto.

Considering the vertical slice of soil of width b and height z in Figure 21, it can be shown that the normal and shearing components of stress on the base, σ and τ, respectively are given by the equations

$$\sigma = \gamma z \cos^2 i \tag{1}$$

$$\tau = \gamma z \sin i \cos i \tag{2}$$

where γ is the unit weight of the soil and i is the slope angle. For a soil deposit under water, σ will be the effective stress, σ', and τ the shearing stress when the buoyant unit weight, γ', is employed in the above expressions. In general, the shearing strength, s, of a soil may be represented by the equation

$$s = c + \sigma' \tan \phi' \tag{3}$$

where c is the cohesion the soil possesses under zero stress conditions and ϕ' is the angle of internal friction with respect to effective stresses.

If the shearing stress, Equation (2), is made equal to the strength Equation (3), in which the normal effective stress, Equation (1) is inserted, the resulting equation can be solved for the critical depth of material, z_c at which sliding of the soil mass will occur. If, as mentioned above, the slope angle i is greater than the friction angle ϕ', and drainage is complete (no excess pore pressures),

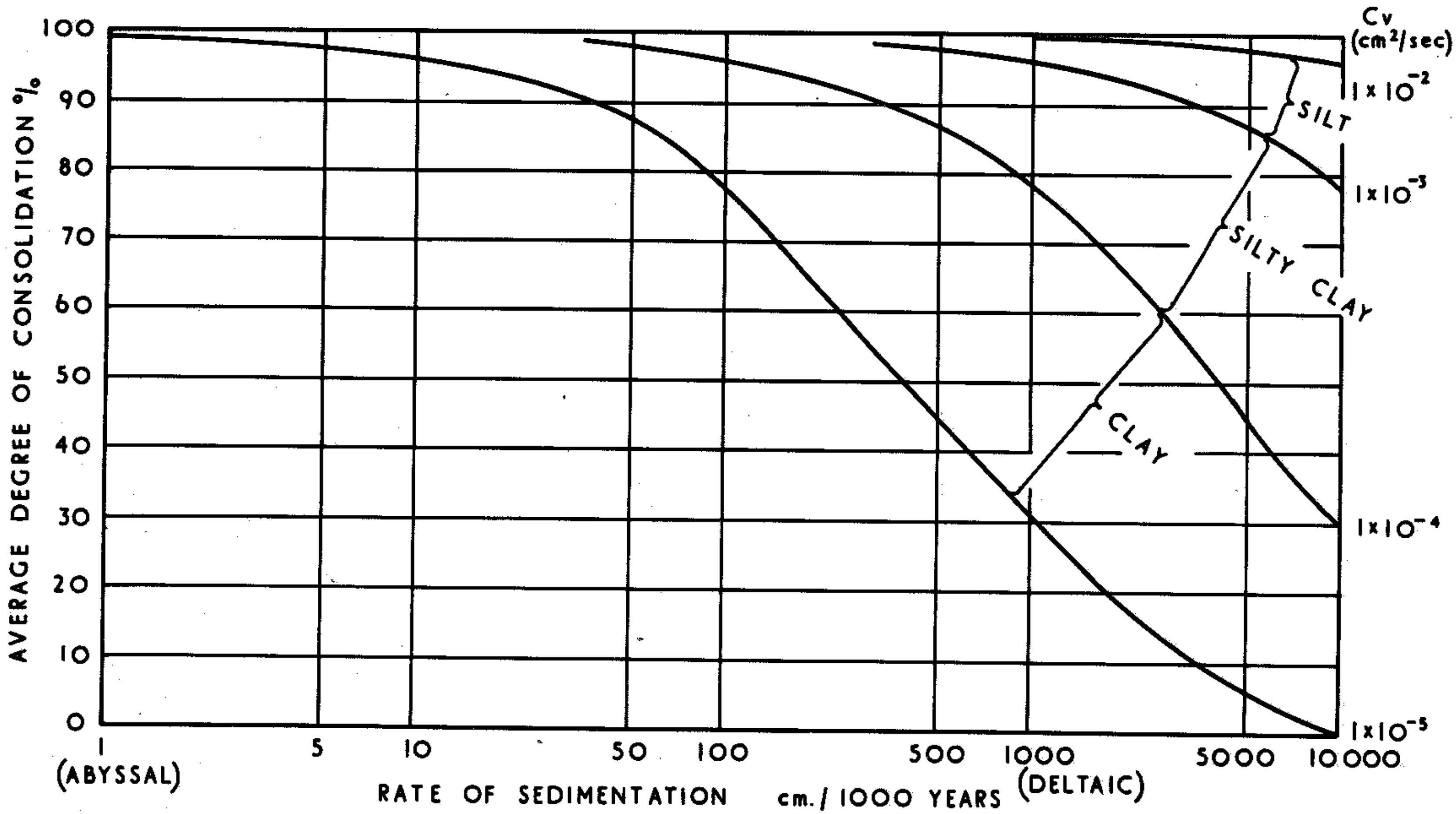


Fig. 20 Sedimentation Rate Versus Degree of Consolidation

After Morgenstern (43)

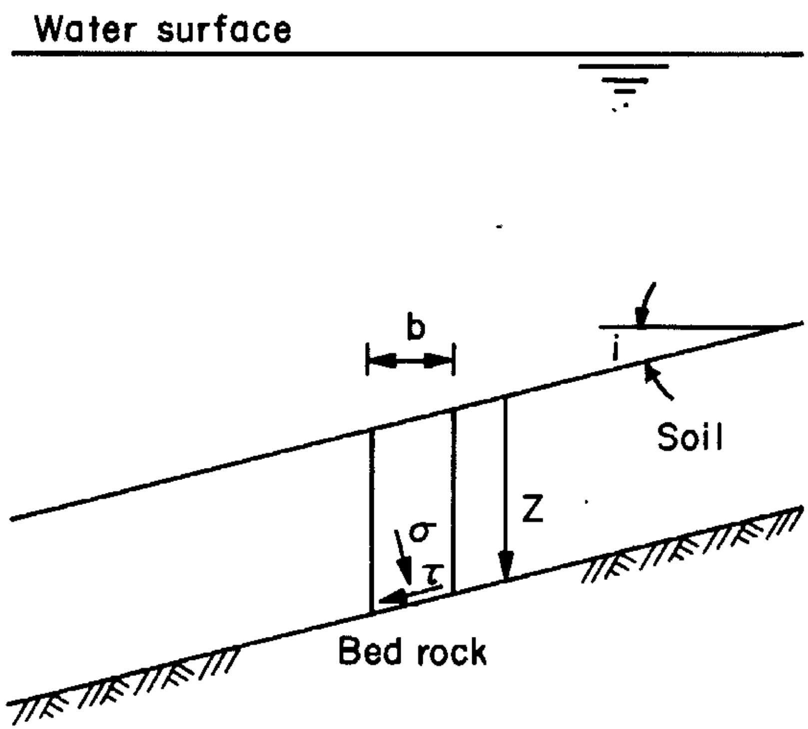


Figure 21

Stresses in Finite Slope

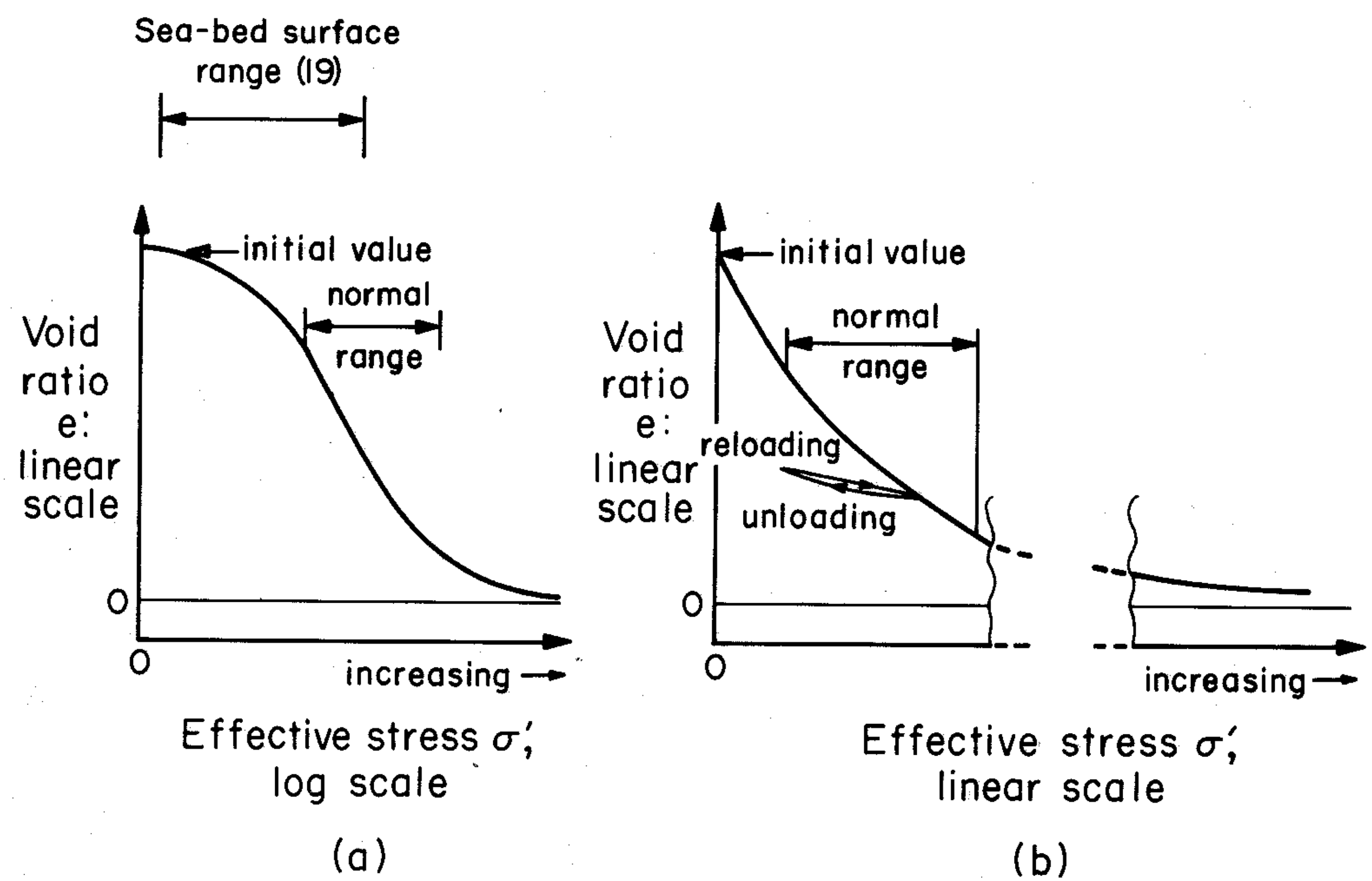


Figure 22

Void Ratio Versus Effective Stress; Log and Linear Scales

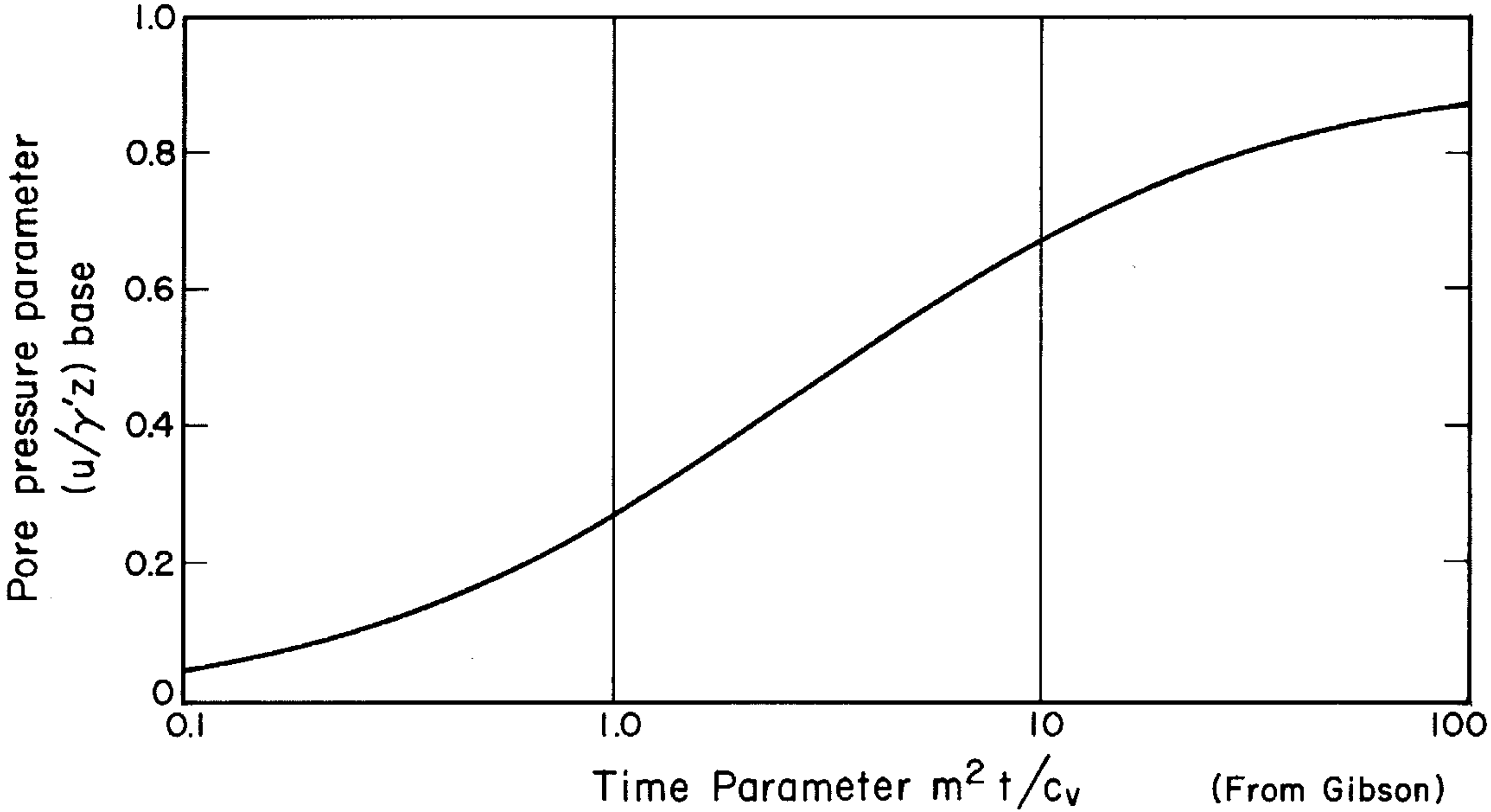


Figure 23

Pore Pressure Versus Time in an Accumulating Layer then:

$$z_c = \frac{c}{\gamma' \cos^2 i(\tan i - \tan \emptyset)} \quad . \tag{4}$$

It is usually convenient to assemble the terms of Equation (4) to give a dimensionless number, $c/\gamma' z_c$, called the stability number (its reciprocal is also used).

In the event that deposition occurs so fast that excess pore pressures are generated in the soil faster than they can dissipate (see Figure 20), then the effective stress σ' in Equation (3) is not given by the effective stress of Equation (1). Instead, the excess of pore pressure u over the hydrostatic pressure at the depth in question must be subtracted from the stress developed by the buoyant weight of the settling soil particles

$$\sigma' = \gamma' z \cos^2 i - u \tag{5}$$

Substituting Equation (5) in Equation (3) gives for the strength of the soil

$$s = c + (\gamma' z \cos^2 i - u) \tan \emptyset' \tag{6}$$

It is convenient also to make this dimensionless by dividing throughout by $\gamma' z$ to get

$$\frac{s}{\gamma' z} = \frac{c}{\gamma' z} + \left(\cos^2 i - \frac{u}{\gamma' z}\right) \tan \emptyset' \quad . \tag{7}$$

In the present discussion the stability of only very flat slopes (<10°) will be discussed. Thus $\cos^2 i$ may be approximated by unity and the thickness of the soil bed normal to its surface and the depth z become closely equal. Equation (7) may be written

$$\frac{s}{\gamma' z} = \frac{c}{\gamma' z} + \left(1 - \frac{u}{\gamma' z}\right) \tan \emptyset' \tag{8}$$

and Equation (2) can be written approximately

$$\frac{\tau}{\gamma' z} = \sin\ i \tag{9}$$

If the excess pore pressure, u , can be calculated at the depth z it may be substituted into Equation (8) to give, with the other parameters known, the strength of the soil. At the same time, Equation (9) will give the shearing stress at the same depth. If the rate of deposition is rapid enough at a given slope angle, the depth will eventually reach a value at which the shearing stress will exceed the strength and the slope will fail.

If the assumptions are made that deposition proceeds at a uniform rate onto an impervious base and that the soil's vertical deformations are small relative to the thickness of the bed at any time, Gibson's (13) solution can be employed (since the slope is also taken to be flat). In this solution, the thickness of the bed z is given by the equation

$$z = mt \tag{10}$$

where m is a constant indicating the rate of deposition, and t is time. Gibson also establishes a dimensionless time variable m^2t/c_v, and obtains a solution for the problem in which $u/\gamma' z$ is given as a function of m^2t/c_v, and dimensionless depth inside the developing soil layer. In the present case, interest is confined to the base of the bed, for which Gibson's solution is replotted in Figure 23. Since the bed thickness (which is a function of time) occurs in both Equations 8 and 9, it is necessary, under given slope, depositional rate, and soil conditions, to solve for the bed thickness at which failure occurs by trial and error. When the critical thickness is obtained, the time interval between, and thus the frequency of failures can be calculated from the depositional rate. In this calculation it is convenient to use the relation m^2t/c_v in the form mz_c/c_v to eliminate the time variable.

As an example, assume a deltaic slope of 5° with a sedimentation rate of 0.1 ft/yr of soil with the following properties: $c = 50$ psf, $\phi' = 15°$, $\gamma' = 20$ pcf, $c_v = 10^{-5}$ cm²/sec (0.34 ft²/yr). Trying $z_c = 25$ ft gives $c/\gamma' z_c = 0.10$, $t = 250$ yr, $m^2t/c_v = 7.37$, so that Figure 23 gives $u/\gamma' z_c = 0.61$ and Equation (8) becomes equal to 0.204. However, the right-hand side of Equation (9) amounts to only 0.0872, so that the trial depth is not great enough. Alternatively, the selected depth would just be stable at a slope angle of $\sin^{-1}(0.204) = 12°$. Several more trials of the problem give a resulting critical depth of about 85 ft, which takes 850 years to develop. Thus, other factors remaining constant, a slide would occur at this location every 850 years. If the soil properties remained constant, but the slope were changed to 12° as above, a slide would occur every 250 years.

Alternatively, keeping all the properties and conditions constant, but increasing the sedimentation rate to 1 ft/yr would cause failure to occur at approximately a critical thickness of 40 ft, so that failure would occur every 40 years in this case.

It is possible to approximate Gibson's solution for pore pressure at the bottom of the layer for both small and large values of the dimensionless time parameters. It is found that the dimensionless pore pressure varies as the square root of the time parameter at small times, and as an exponential function of the time parameter at large times. However, substitution of these forms of the relationship into Equations (8) and (9) does not simplify the calculations for z_c significantly.

Changing the soil properties and slopes in the plausible range for various materials and sites, and carrying out analyses as described above, gives rise to the tentative conclusion that slope failures are unlikely to occur as a result of the mechanism in question more frequently than once in several tens of years at a particular site. They may, of course, be triggered, after some period of accumulation, by an earthquake, low tide, or low sea level during a hurricane or tsunami, the latter of which effectively increase the excess pore pressure in the soil.

(b) Finite Slopes

For subaerial slopes, the case most studied has been the stability of finite slopes, and a variety of analysis techniques have been developed to treat this problem. The most common method, which generally satisfactorily predicts or describes failures which do occur in practice is termed the "method of slices". It is usually used, as shown in Figure 24 in conjunction with the assumption that failure occurs along a circular failure surface through the soil; in this form the most detailed description of the technique has been given by Bishop (3). It has also been applied to the analysis of sliding which may occur along a noncircular failure surface, by Morgenstern (44). A brief description of the method will be given here, for completeness and in order that its application to ocean-floor conditions can be examined.

The determination of the distribution of stresses and displacements in nonhomogeneous soils is a difficult problem to solve, and the method of slices is one of a number of approximate analyses employed in practice to give information on stability. In general, to solve a problem such as slope stability it is required that the equations of equilibrium, the stress-strain relations of the material, compatibility requirements, and the boundary conditions all be satisfied. In the approximate method, only the equilibrium, and, to some extent, the boundary conditions are taken into account, in order to yield a statically determinate solution. Even these conditions are not employed in an exact form.

In the finite slope of Figure 24(a), a failure surface ACB is first selected; it may be circular, or have another form, as in Morgenstern's (44) approach. The region isolated by this hypothetical surface is divided into a number (usually 6 to 12) of vertical slices, of which n is taken to be a typical one. The forces active on the sides and base of n are shown in Figure 24(b), for clarity, and are conveniently divided into water forces and effective or soil forces. Acting vertically downward is the buoyant weight of the slice and the weight of the water in the volume of the slice. In the event that the stability of the slope is to be assessed under earthquake conditions, one method of equivalent static analysis is to add a lateral (and sometimes vertical also) component to both the soil and water weight at a given percentage of the vertical weight. Such a lateral component is shown dashed in Figure 24(b). On the base of the slice, at an angle to the base, and on the left- and right-hand sides of the slice, also at an angle, act the forces deriving from the effective stresses in the soil, B_S, L_{SL}, and L_{SR}. Water forces act also on the base and sides, but normally to the boundaries, B_W, L_{WL} and L_{WR}. Normally, in subaerial slide analysis, the upper surface of the slice is force-free, but for a subaqueous slope, a water force must be added to this surface, T_W.

The water forces can be calculated either for the general case of water moving through the soil by seepage, steady or nonsteady, or in the special case where no water movement occurs. In the latter circumstance, all the water forces balance each other and can be left out of the analysis, which then includes only the soil forces arising from effective stresses. It should be noted that the water forces cannot be eliminated in the case of a static seismic analysis, since the earthquake acceleration acts both on the soil grains and on the water filling the pores.

The direction and magnitudes of the soil forces acting on the sides of each slice are not known, and in the literature (63) much discussion has been devoted to different lateral soil force assumptions by which the analysis can be completed. It has been demonstrated that the effect of different assumptions, ranging to the extreme of assuming the side forces on each slice due to the soil just counteract each other, is only a few percent (44) in the resultant factor of safety (or strength required to resist failure). Consequently, lateral forces due to the effective stresses in the soil are usually omitted from slope stability analyses. Again, it must be noted that the soil and water pressures which act during an earthquake are unknown, and no methods for calculating them have currently been agreed on. Generally the static seismic coefficient method of analysis is employed in this case, using water pressures the same as those employed in the analysis without earthquake effects.

Since the forces acting on each slice are in static equilibrium, they must form a closed force polygon. When the soil side forces are neglected, or if an assumption is made regarding them, the magnitude and direction of the unknown forces can be determined. The unknowns are the normal and shearing component of the soil force on the base of the slice. The shearing component is the force acting to cause failure and it is resisted by the soil's cohesion along the failure arc plus the normal effective force times the angle of internal friction of the soil, as in Equation (3). The vertical and horizontal force equilibrium of each slice is taken care of by the polygon construction; the problem is completed by considering the moment equilibrium about the center of the circle. A factor of safety is computed by dividing the sum of the resistive moments (soil's strength times circle radius) by the sum of the shearing force moments. This total process is repeated for other circles to establish a circle with the smallest factor of safety. This is then an indication of the stability of the slope in question.

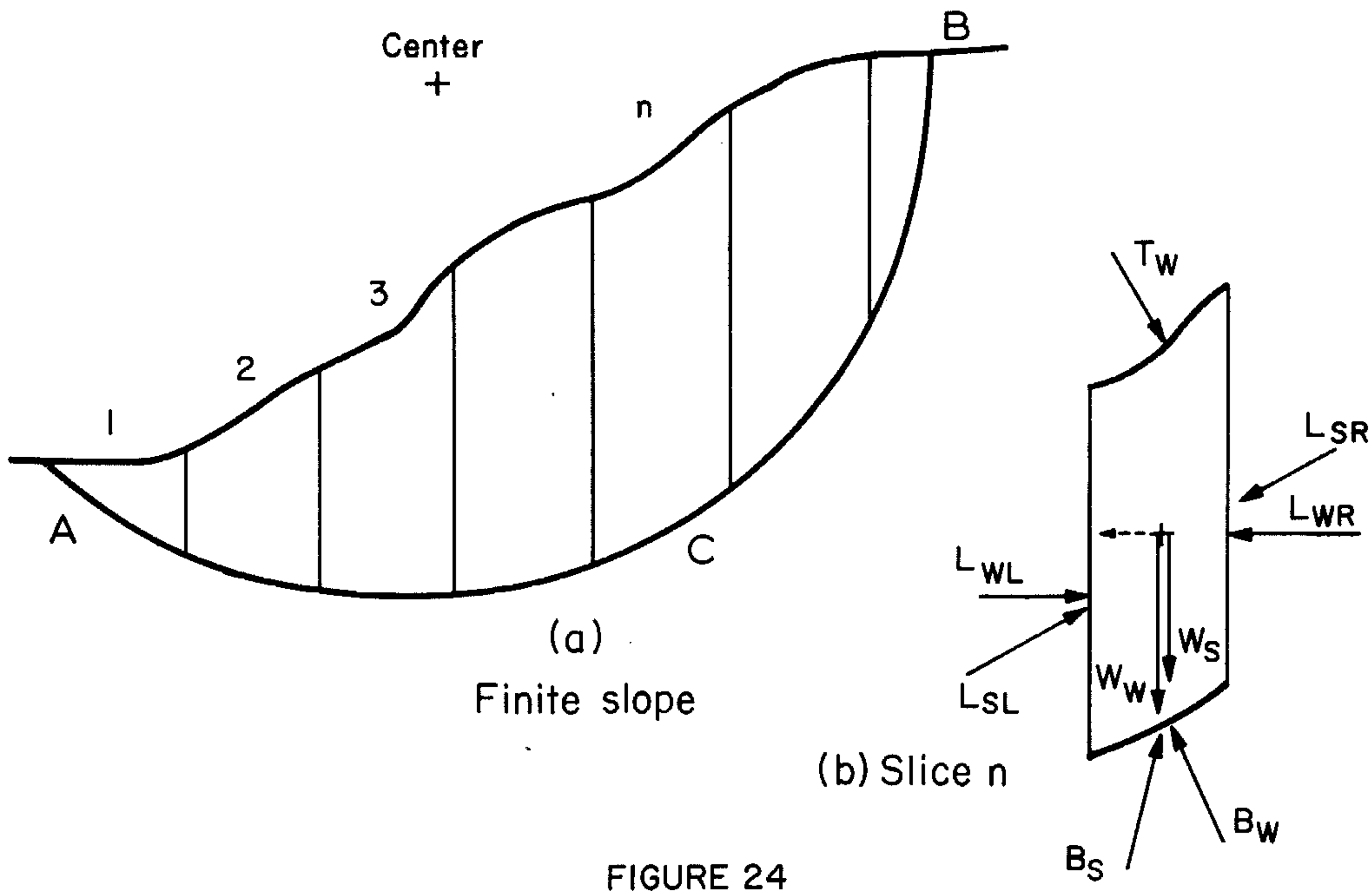


FIGURE 24

Stability Analysis of Finite Slope; Slices Method

Including the water force in the top of the slice, there appears no stage in the preceding analysis which cannot be applied to ocean floor soils, on examination. Except for the general uncertainty about dynamic effective soil and water pressures, the conventional modification of the procedure for earthquake stability analysis can also be employed. It would also appear that another modification of the technique can be applied to the case of the stability of underwater slopes during low tides or tsunami (trough rather than wave) conditions. These circumstances are similar in nature to the effect of the rapid drawdown of a reservoir on the stability of an earth dam. In the case of low tides, the effect is to reduce the water force on the top of the slice (T_W in Figure 24(b)) for a short period of time. Generally this time interval will be too brief for the excess pore pressures thereby generated in the underlying soil to dissipate through the generally fine-grained sediments. The slope will therefore be less stable under these conditions. The degree of instability can be examined by carrying out the analysis with all forces the same as before, except for reduced values of the water forces on the tops of the slices.

In some circumstances, slope instability is manifest by creeping of the slope surface, or sections of the slope surface. The varieties of such movement have been categorized by Saito (50). When this creeping movement occurs in a soil whose strength diminishes at large deformations or distortions (unstable stress-strain relationship) individual sections of the slide offer a progressively diminishing shearing resistance until eventually the whole slide mass moves relatively rapidly. When a calculation of the stability of such a slide is made, employing the peak shearing strength of the soil, it is found that the slope has a substantial factor of safety. Skempton (61) has extensively studied such cases and recommends estimating a correct factor of safety based on the residual shearing strength of the soil. This is obtained from laboratory tests in which the soil is subjected to shearing stresses successively reversed in direction, until the measured shearing resistance reaches a low but constant residual value. Some undersea slopes may fail by this mechanism. The viscous properties of soil and the mechanism of viscous flow or creep by which these slides are initiated have not been studied in detail.

In summary, the procedures which can be utilized to determine the stability of sea-floor slopes are similar to those in common use in subaerial soil mechanics. These two stages in such an evaluation are: (1) site investigation, and (2) stability analysis. The object of the site investigation is to determine the nature of the existing topography, the soil profile, and soil strength properties.

On land, the geometry of the ground surface is ascertained by various surveying techniques. These cannot be employed at sea, and another approach must be used. Methods for delineating slope geometry have been limited to acoustic reflection techniques. The resolution of this technique is sufficient for slope stability analysis (75) provided a suitable ranging system is also employed to give horizontal distances. At longer wavelengths these survey methods have also been used to provide profiles of subsurface structure, (76, 77, 20). Under good conditions, it seems likely that a submarine slope can be described with better accuracy than a subaerial slope, including sub-bottom (subsurface) details of the presence of soil layers and bedrock.

It is important in this phase to determine if any special conditions such as excess pore pressures, expected sea level changes, wave pressures, seismicity, metastability, or rapid sedimentation exist in the area of consideration. Overlooking one of these factors may lead to false estimates of the stability of a given slope.

On land, subsurface information on the large scale is obtained from geophysical and geological investigations. For greater detail, a variety of drilling and samping techniques are employed; the samples obtained are tested for the relevant soil properties in the laboratory. Occasionally in-place tests are conducted in the field in boreholes, or near the surface. Considerable effort is expended, in general, in the attempt to obtain samples as little disturbed as possible, so that the tests carried out will reflect the true in-place properties.

This approach can only be carried out in very shallow water at sea, in deeper water, samples are obtained in core barrels lowered by cable from ships.

Soil properties are determined from these core samples which provide only considerably disturbed samples; <u>in-situ</u> strength measurements, such as the vane shear test, have therefore been attempted (81). This does not mean to imply that core samples have no value. They provide information on sub-bottom soil profiles and yield samples for laboratory examination and testing of the mass properties of void ratio, density, etc.

The methods discussed for stability analysis of submarine slope utilize the same principles and models developed for aerial slopes. In both environments the slope is chosen to be "infinite" or "finite" depending on local topography. The solution to the former case is provided by Taylor (63). Modification of this analysis for underconsolidated sediments is discussed earlier in this section. Finite slopes are most easily analyzed with the "method of slices". This method is described by Bishop (3) for circular failure surfaces and Morgenstern (44) for noncircular or general failure surfaces. One need only add the appropriate water forces to the aerial method of analyses to establish the stability of a submarine slope. In conclusion, therefore, by careful use and appropriate modification of the principles of terrestrial slope stability analysis, the safety of ocean floor slopes can be calculated to a moderate degree of confidence.

III. CONCLUSIONS AND RECOMMENDATIONS

1. Stability analysis methods exist for subaerial slides which can, with some modification, be applied to certain subaqueous slides, in principle.

2. No examples were found in the literature analogous to those numerous cases with subaerial slides where detailed measurements enabled stability analyses to be performed, for confirmation of the methods and assumptions employed.

3. A variety of mechanisms may operate to trigger subaqueous slides, and certain mechanisms may predominate in particular areas and soil types.

4. Although various mechanisms operate in subaerial slides, the major number of slides of geotechnical interest have resulted from the mechanism of oversteepening (or heightening) of a slope. Analysis methods for other mechanisms are less well developed.

5. In subaqueous slides there may be a larger variety of mechanisms at work in general, but there are insufficient numbers of studies of subaqueous slides.

6. Some mechanisms in subaqueous slides are not well understood, for example, creep, generation of turbidity currents, pore pressure effects from a variety of internal and external causes and the role of earthquakes, vibrations and possibly surface faulting.

7. In general, a considerable development is required in techniques of obtaining in situ shearing strength information for analysis purposes.

8. Site investigation methods relating to subsurface profiles, layering and soil properties are not well developed.

9. There is little experience on the effects of modification of the ocean floor by construction; for example, the stability of the side walls of an excavation and the stability of fills.

10. Rapid site hazard assessment must presently be limited to restricting construction from certain specific areas, on a general rather than specific basis.

RECOMMENDATIONS

A. General

   1. Both controlled field and laboratory experiments are required to develop the knowledge basic to an understanding of subaqueous slide mechanisms and analysis. It is emphasized that controlled field

experiments are needed; slides must be generated in areas where the *in situ* soil properties have been thoroughly examined. The slides may be generated by a number of techniques including excavation, excess pore pressure development by means external or internal to the soil, and possibly vibration.

2. Site investigational methods require improvement.

3. As more field results become available, analysis techniques may need revision.

4. Rapid site hazard assessment techniques require development.

B. Specific

1. The slope failure experiments must be performed in a variety of soils. For obvious reasons, field experiments should be performed in water as shallow and clear as possible. If natural clay or sand deposits are utilized, slope failures may be induced by excavation of trenches with various slope angles. This will be difficult to do in a controlled fashion underwater, and will probably require special equipment. In particular, a visibility problem will develop where fine-grained sediments are involved, and slope movement monitoring will have to be provided from the water surface. Two methods may be employed.

   (a) Periodic sonar measurement of the slope profile from a vessel whose path is carefully controlled.

   (b) Embedment of sonar "pingers" in the test slope and distance measurement to the pingers from receivers fixed in position.

   In these respects, shallow underwater slope movements may be observed more comprehensively than those of ground surface slides. In the case of sands, the rate and depth of flow of material on a limiting slope may be studied; this can be more satisfactorily performed in laboratory tests than can clay slope failures.

   A number of attempts at inducing offshore sand liquefaction and flow by explosives has been unsuccessful, probably for the reasons discussed earlier in this report with respect to model liquefaction studies. It may be possible to prepare a full-scale soil profile in shallow water for better-controlled liquefaction studies. It is suggested that this profile consist of a loose, medium-to fine-grained-sand overlain to a depth of a few feet by a finer silty sand material, over an area of dimensions in the order of hundreds of feet. Slopes of 5° to 10° should be satisfactory for experimental purposes. Liquefaction may be induced by explosion vibration, or possibly by artificially raising the pore pressure in the underlying layer with injection wells.

In such experiments, continuous pore pressure measurements must be made in the liquefying medium, and the position of points on the soil surface should be continuously monitored. It would be desirable to include underwater movie photography to record the nature of the movement and the growth, if any, of a turbidity current. To facilitate the latter study, explosive charges should be designed to provide no surface venting through the overlying layer.

In the laboratory, the possibility of carrying out liquefaction, flow, and turbidity current generation studies on artificial soils and liquids other than water should be studied following an examination of the required scaling parameters, to see if the full-scale phenomena can be reproduced at the laboratory scale.

2. In controlled shallow-water experiments and at shallow-water field sites, conventional field and laboratory soil mechanics tests can be employed to give information on the mechanical properties of the soil. In deep water, however, efforts need to be made to develop techniques and instrumentation both to retrieve undisturbed samples for laboratory testing and investigation, and to measure the relevant properties of density, porosity and shear strength components in place. Even the best present methods of deep-water ocean floor sampling fall far short of the best subaerial techniques in degree of sample disturbance. The techniques of hydrostatic and gas driven coring appear to be promising and require detailed examination. The problem of obtaining samples from depths of up to at least 50 ft below the ocean floor may be amenable to these techniques. At some sites, measurements of subsurface pore pressures are required; at present, satisfactory equipment for this purpose seems to be lacking.

   Relatively precise <u>in situ</u> measurements to date have been limited to several attempts to employ automatic vane-shearing devices. These are limited both to cohesive soils and as to depth they can penetrate in the soil. They have the disadvantage of being complicated and therefore lacking in reliability in the ocean environment. The measurement of in-place soil strength by various types of penetration devices has been attempted. This technique is capable of extension to both cohesionless and cohesive soils. Possibly, with modifications to employ hydrostatic pressure as a driving agent, penetrations to depths of 50 to 100 ft could be attained. In-place densities can be obtained by gamma radiation devices, which have only seen limited use at sea. Examination of the employment of such equipment, perhaps in association with penetrometers to give depth-density profiles, might be fruitful. Such a development would require ocean-floor equipment expensive enough to make its retrieval mandatory.

   No attention seems to have been paid to the possible deployment on the ocean floor of self-contained burrowing or boring vehicles* (possibly

*The senior author holds (jointly) a patent on such a device, originally developed for planetary exploration from a landed spacecraft. (U.S. Patent 3375885).

independent or cable-connected to the surface) in which soil-property measuring equipment might be installed.

Although sonar and seismic techniques have been employed to delineate the ocean floor surface, and the sub-bottom profile at geophysical scales, there do not seem to be any reports in the open literature about the use of sonar to determine sea-floor soil properties, and profiles at a scale of tens of feet. The geophysical work is essentially qualitative. There would seem a promising area of investigation in examining the possibility of determining quantitative properties of ocean-floor soils by a quantitative employment of sonar. A possible technique would be to emit a sonar signal sweeping a predetermined frequency range, and to compare the frequency and intensity of the returned signal with that emitted with reference to the nature of the sea bed. The actual frequency range employed would determine the depth of sonar penetration into the soil. A combined sonar-soil mechanics investigation would prove of value.

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