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THE STORAGE OF OIL IN AN EARTH RESERVOIR

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I. INTRODUCTION

This paper describes some of the theoretical and experimental studies relative to the storage of fuel oil in earth reservoirs; it also describes the design and construction of such a reservoir in Amuay, Venezuela, for the Creole Petroleum Corporation. This paper should be of interest, not only because the Amuay Oil Reservoir is the first of its kind, but also because it illustrates that soil is a very cheap construction material which can be successfully used for certain industrial applications.

The market for fuel oil is very seasonal, with demands the highest during the months of November, December, and January. To meet the heavy requirements during these months, a refinery must stockpile during the preceding slack months. During these slack months, fuel oil is naturally being produced because of the relatively heavy demands for the lighter fractions of the crude oil, primarily gasoline.

Because of newly developed markets, early in 1955 the Creole Petroleum Corporation found it needed by September 1, 1955 additional storage capacity for 3,500,000 barrels (147,000,000 gallons) of fuel oil. There was not time available to design, fabricate and construct steel tankage, the conventional manner of oil storage. Creole requested, therefore, that ESSO Research and Development consider the use of a reservoir to store the oil. ESSO studied questions such

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as, the danger of fire from lightning, and the chances of the formation of rain water-oil emulsion. The author was retained to study problems of seepage.

Based on the favorable results of the studies by ESSO and the author, Creole decided to go ahead with its idea. Foundation exploration at the proposed reservoir site disclosed the presence of very weak and soft clay. The writer was then retained to design, and to aid construction supervision of the dam and reservoir floor.

As described in the following pages, the reservoir was built during the period between the middle of June and the first of September. It has now been filled and emptied; its performance was excellent. Not only was it built more rapidly than possible with conventional storage, but it resulted in an extraordinary financial saving. The earth reservoir (including pumping and piping facilities) cost $0.27 per barrel of storage compared with $2.00 per barrel of steel tankage.

The very favorable results with the Aduana Oil Reservoir led to the decision to construct another oil storage reservoir, twice the capacity of the first, at Amuay during the spring and summer of 1956. The use of soil in conjunction with concrete to store more volatile oil fractions at Maracaibo is also being considered.

II. The Permeability* of Soil to Various Fluids

A. *General*

Three main questions to be considered when studying the permeability of soil to various fluids are:

1. What is the effect of the pore fluid on the structure of the soil?
2. What is the effect of the permeant on the permeability of the soil in one-phase steady flow?
3. What is the pressure required to initiate flow in a multiphase pore fluid when the phase being replaced is a wetting one and the replacing phase is a non-wetting one?

*The soil engineer is usually concerned with the permeability of soil to water and expresses his permeability as a superficial velocity of flow per unit of gradient, normally in units of cm/sec or microns/sec. A more fundamental expression of permeability employs a pressure differential rather than a head differential and considers the viscosity of the permeant and is equal to the soil engineer's k multiplied by the viscosity divided by the unit weight. The following are simple relationships between the various expressions of permeability:

1 Darcy = 1.055 x 10-2 cm2

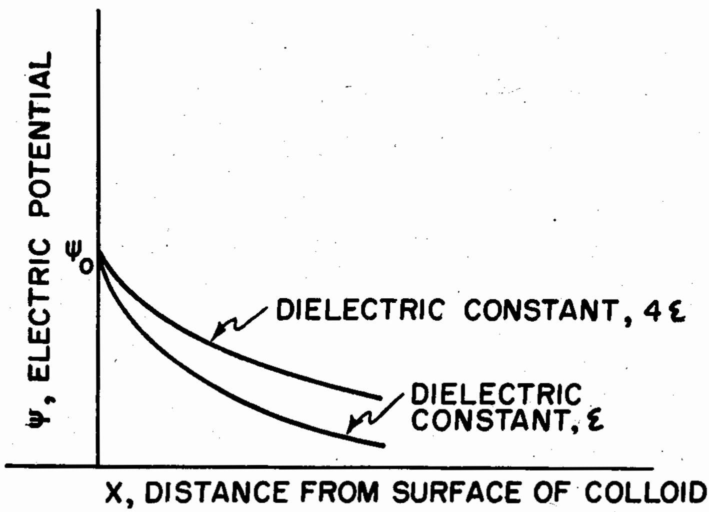
for water at 20°C, K in cm2 = K in cm/sec x 1.028 x 10^{-5}

K in cm/sec = K in Darcy's x 1.028 x 10^{3}

These three questions are treated separately in the following paragraphs.

B. *The Effect of Pore Fluid on Soil Structure*

The clay-size soil minerals carry net negative charges—no positively charged soil mineral colloids have been reported. The pore fluid surrounding the soil colloid contains positively charged ions which counterbalance the negative charge of the colloid so that the entire system is neutral. The electrical potential in the pore fluid decreases from a maximum value of ψ_0 at the surface of the soil colloid as is illustrated in Figure 1. The potential-distance plot in Fig-



EFFECT OF DIELECTRIC CONSTANT OF PORE FLUID ON DOUBLE LAYER OF COLLOID

FIG. 1.

ure 1 indicates the size of the double layer of the colloid which is made up of the negative charge on the soil particle plus the counter ions in the pore fluid. The thickness of the double layer around a soil colloid is of considerable importance on the behavior of the soil colloid since it is a measure of the amount of immobilized pore fluid around the particle and, more importantly, of the repulsive force between the particle and an adjacent one.

Overbeek (1949) has studied the properties of the diffused double layer at great length. His equations based on the Gouy-Chapman Colloid Theory show that the thickness of the diffuse double layer depends on the dielectric constant of the dispersion medium (along with other variables). He has shown, for example, that the x(distance from surface of colloid) for any given potential varies directly with the square of the dielectric constant as illustrated in Figure 1. He has further shown by his theoretical equations that an increase in dielectric constant increases the repulsive force between adjacent colloids.

The soil engineer is well aware of the fact that an increase in ion concentration in the pore fluid usually causes the flocculation of soil colloids—this is one of the contributing factors to the deposition of river-borne sediments when the river reaches a salt water body such as the ocean. In a similar fashion, a decrease in dielectric constant of the pore fluid causes a decrease in the repulsive force between particles or, in effect, an increase in the net attractive interparticle force, with the resulting tendency toward flocculation. The effect of the dielectric constant of the pore fluid on the structure of a soil is illustrated in Figure 2. In this Figure are shown three test tubes, each containing the same weight of kaolinite but with a different pore fluid—Tube a containing benzine, b alcohol, c water. All three tubes were shaken and at the same time sedimentation permitted; after a couple of minutes the photograph in Figure 2 was taken. The kaolinite in the dispersion medium with the lowest dielectric constant, benzine, flocculated very rapidly; the kaolinite in the medium with the highest dielectric constant was still dispersed when the picture was taken; and that in the alcohol was midway between the two extremes. At a long period of elapsed time, kaolinite in the benzine will settle to a loose porous sediment and the kaolinite in the water to a denser one; the degree of particle orientation will be much closer to parallelism in the water.

The effects of soil structure on strength are well known to the soil engineer; this structure is the reason that a marine clay can have a very high strength in the undisturbed state and essentially no strength when remolded. The effect of soil structure can be just as startling on permeability as on strength. For example, data on two clays (Lambe, 1955) in Table I show that the permeability of the soil can vary over 50 fold with structure even though the density

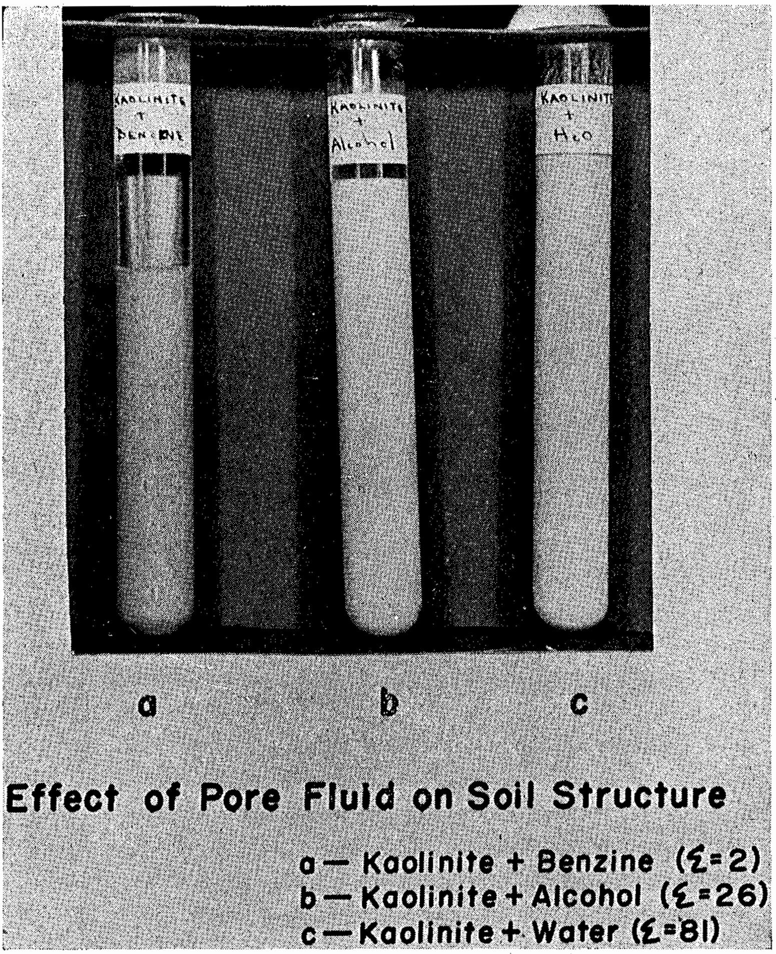


Fig. 2.

and degree of saturation of the compacted samples are the same. The samples with the lower permeability have structures of adjacent particles being more nearly parallel.

Theoretical considerations, supported by experimental data, show that the properties of the pore fluid, especially dielectric constant, can have a very pronounced effect on the structure and engineering behavior of fine grained soils. While the largest effects are, naturally, realized when the sediments are deposited in different pore fluids, a

TABLE I

Soil	Dry Density or Void Ratio	Degree of Saturation	Permeability cm per sec
Jamaican Clay	115 lb per cu ft	Approx. same	4×10^{-6}
	116 lb per cu ft		7×10^{-8}
Virginia Sandy Clay	1.3	100%	2.7×10^{-4}
	1.3	100%	1×10^{-3}

change of pore fluid in a given sedimentary deposit can cause changes in structure. Research in the Massachusetts Institute of Technology Soil Stabilization Laboratory has shown, for example, that leaching compacted samples of dirty sands with different pore fluids can affect changes in permeability as large as 100 fold by changing the structure of the soil. In summary, when considering the effects of permeant on soil permeability, we must give thought to the effects of the permeant on the structure of the soil.

C. *Effect of Permeant Characteristics on Steady-State, One-Phase Fluid Flow in Soil*

The soil engineer is well aware that the permeability of a soil varies directly with the unit weight and inversely with the viscosity of the permeant. As was noted in the preceding section, the polarity of the pore fluid has an influence on the thickness of the diffuse double layer of the soil colloids. We would expect, therefore, that the more polar the permeant, the thicker the immobilized layer of fluid and the lower the permeability of the soil. This expectation is borne out by experimental data, as illustrated, in Table II from the work of Michaels and Lin. These data show that when a sample of kaolinite was molded in water and compacted to a void ratio of 0.94, the permeability measured successively to the three permeants, water, acetone, and nitrogen with each permeant replacing the preceding one, that permeabilities of 0.040, 0.047, 0.067 were obtained. In other words, the permeability of the least polar (nitrogen) was almost 70% higher than the most polar (water). The effects of the permeant on the permeability caused by the thickness of immobilized fluid are very small, however, when compared to the effects of the permeant on the structure of the soil. This is illustrated by the right column of data in Table II which are the permeabilities to each of

TABLE II*

Permeant	Dielectric Constant	Permeability of Saturated Kaolinite in $10^{-10}cm^2$ Molded in Water	Molded in Permeant
Nitrogen	1	0.067	1.1
Acetone	21	0.047	0.31
Water	81	0.040	0.15
		Void Ratio = 0.94	Void Ratio = 1.43

*From Michaels, A. S. and Lin, C. S., "The Permeability of Kaolinite", "Industrial and Engineering Chemistry" 46, 1954.

the permeant when the sample was compacted in that permeant. These figures show that the effect of the permeant on permeability by giving different structure to the soil is far more important than the effect of the immobilized fluid.

In summary, we can say that the polarity of the permeant has a measurable influence on the permeability of the soil; this influence, however, is relatively small compared to that of the permeant's influence on structure.

D. *Multi-Phase Fluid Flow*

Petroleum engineers have devoted considerable effort to the study of multi-phase fluid flow in rocks and sands for application to oil production. See, for example, "Porosity, Permeability, and Capillary Properties of Petroleum Reservoirs" by Charles D. Russell and Parke A. Dickey, in *Applied Sedimentation,* edited by Parker D. Trask, John Wiley and Sons, 1950, for a general discussion of this subject from the viewpoint of a petroleum engineer and see Volume 192, Transaction of the American Institute of Mining and Metallurgical Engineers, 1951, for a number of good papers on the subject. Figure 3 is a plot of multi-phase fluid flow data on a sandstone core. Figure 3a is a plot of degree of water saturation versus air pressure required to force water out. This curve, a type familiar to soil engineers and soil scientists, indicates the pressure necessary to force water from the stone core against the capillary forces. Thus, a pressure of 1 psi or a head of about 70 cm of water is necessary for air to push out any of the water; regardless of the pressure, the degree of saturation does not fall below a value of 5%. This remaining water is termed "connate water" and corresponds to the soil engineer's "capillary water". Figure 3b shows the relative permeability,

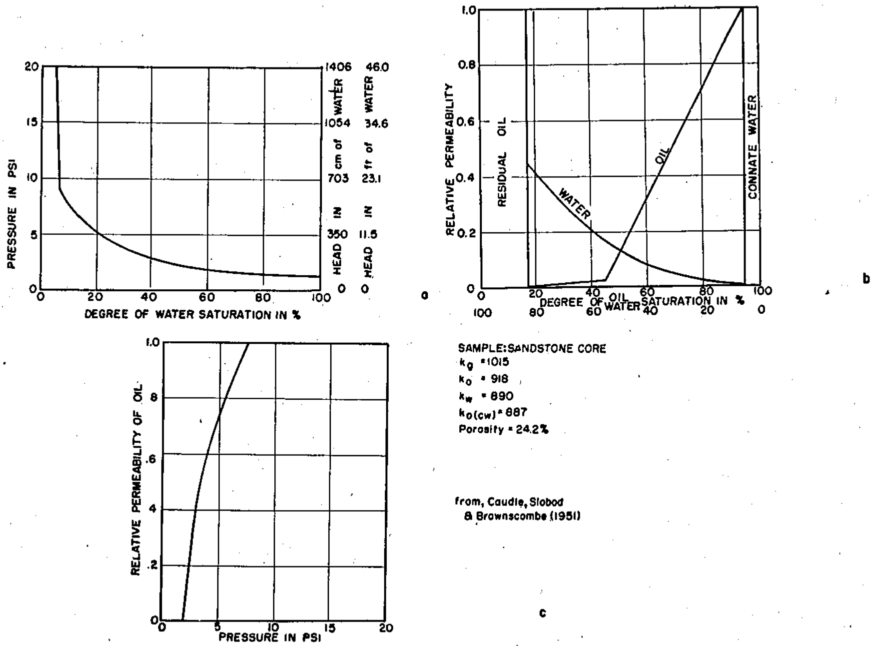


FIG. 3.

that is, the permeability measured at that degree of saturation divided by the permeability to water at full saturation, as a function of degree of saturation. The plots indicate that for this particular core:

1. The permeability to water of an initially oil-saturated core never reaches more than about 44% of its maximum value because of the residual oil.

2. Essentially, the full value of oil permeability can be reached on the initially water saturated core.

Listed in Figure 3 are the measured values of permeability to the various fluids when each is the only permeating fluid. The permeability to gas is 1,105 millidarcys, permeability to oil 918 millidarcys, and permeability to water 890 millidarcys; these data illustrate what has already been pointed out, namely, that the absolute permeability increases with a decrease in permeant polarity.

The reason for the relatively high value of residual oil as indicated in Figure 3b is apparently as follows: The soil, which is preferentially wet by water over oil, draws water into the ends of voids, thereby sealing in the entrapped oil. This phenomenon is essentially the same as that which occurs when water flows into a dry soil; gas is sealed in pockets so that the degree of saturation is considerably below 100%. It can reach 100% by the water eventually dissolving the gas.

Figure 3c is a combination of the data in Figure 3a and 3b; it shows a plot of pressure versus relative permeability of oil. Until the pressure is in excess of 1 lb/sq in, there is no flow into the initially water-wet core. As the pressure is increased, the permeability rapidly rises until a pressure of 7½ gives the full relative permeability of the oil. The cross plot in Figure 3c implies that the capillary water has the same resistance to expulsion by oil as by air. These two resistances are unequal as discussed later in this section.

The sandstone core for which the data in Figure 3 are applicable has two characteristics which makes permeability behavior more simple than normally encountered by the soil engineer. By being a cemented sandstone, it cannot change in volume or structure; further, it is a relatively free draining material. The saturation capillary head of 70 centimeters corresponds to what would exist in a uniform fine sand; a normal silt would have a saturation head of about 200 centimeters.

The particular multi-phase flow in which we are interested with our present problem of the retention of organic fluid by wet soil is that of the replacement from the soil of the wetting phase, water, by the non-wetting phase, oil. Since the soil engineer has had so much experience with the replacement of water by air in his drainage studies, we can profit by comparing water replacement with oil by water replacement by air. Figure 4 shows two capillary tubes which were initially filled with water and then raised until the capillary head h obtained an equilibrium condition. The tube on the left contains water on the bottom and oil on the top, while the tube on the right has water on the bottom and air on the top. The capillary head, h, is

$$h = \frac{2\gamma}{R\ d \cos \alpha}$$

where,

γ = interfacial tension
R = radius of tube

d = density of water
α = contact angle between the wetting phase and the tube.

The contact angle between water and glass is essentially zero, and it is thought that the contact angle between water and soil is zero. For the case shown in Figure 4, if the contact angle between the water and tube wall is zero, then the contact angle between the oil and wall of the capillary must also be zero. For the system shown in

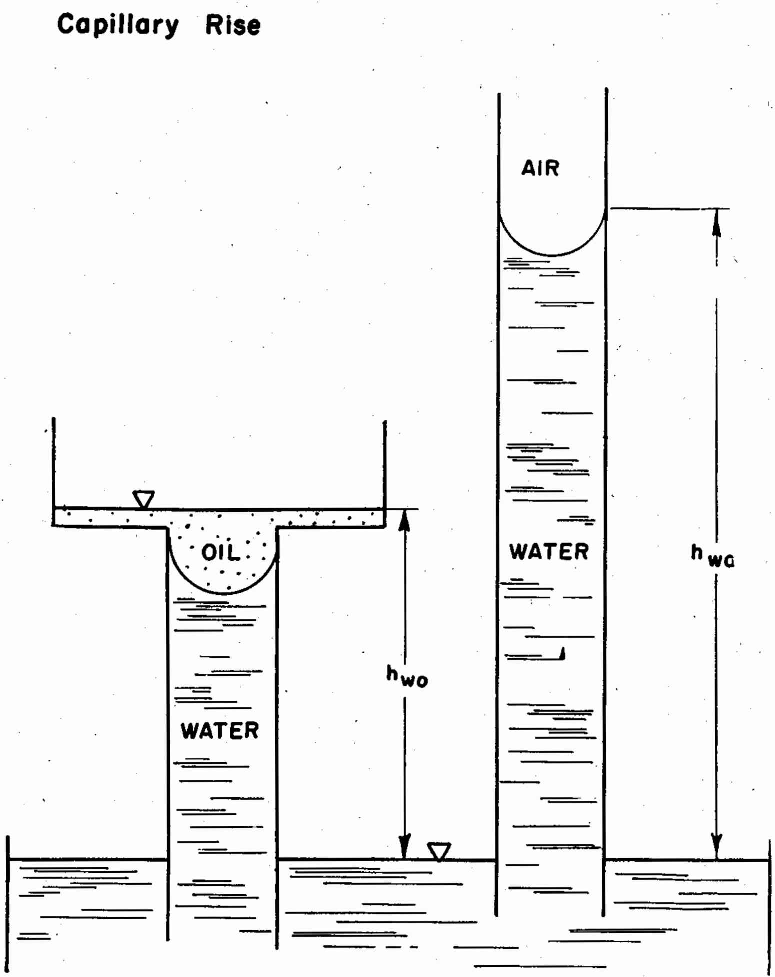


FIG. 4.

Figure 4, then the height h varies as the interfacial tension. The interfacial tension of a water-air boundary at 70°F is equal to 72 dynes/centimeter; the interfacial tension of an oil-air boundary at 70°F for the oil used in the following study is equal to 36 dynes/centimeter. The following expression has been found to be approximately true:

$$\gamma_{wo} \approx \gamma_{wa} - \gamma_{oa},$$

where,

γ_{wo} = interfacial tension between water and oil
γ_{wa} = interfacial tension between water and air
γ_{oa} = interfacial tension between oil and air

Employing these numbers, we find then that $h_{wo} = h_{wa} \times \frac{36}{72}$.

If these approximate relationships between interfacial tensions in the capillary tubes hold in soil systems, we can find the pressure required to push water out of soil with oil by simply multiplying the pressure required for air to push water out by ½.

In Figure 5a is shown a diagrammatical setup for measuring the pressure required for oil to push water out of a soil sample. The sample of soil is compacted into the mold and a layer of oil placed on top of the wet soil and then to the top of the oil air pressure is applied until a movement of water in the burette indicates that water is being forced from the soil sample. The pressure required to just start forcing the water out by oil is shown in Figure 5b as h_{wo}. By using increasingly larger pressures and measuring the degree of oil saturation at equilibrium under each pressure, we can obtain the entire curve shown in Figure 5b. While this is a simple sounding procedure, it is actually one which requires considerable effort to perform.

As already noted, the pressures required to initiate flow of a wetting phase by a non-wetting phase from a fine grained soil can be considerable. The air pressure required to force water from a clay has been approximated on a number of clays by the method illustrated in Figure 6. In Figure 6a is a plot of water content versus void ratio for an initially water saturated sample of clay which is slowly dried; at each water content a measurement is made of a volume, thus permitting the continuous plot to be obtained. The void ratio-water content plot is a straight line until some point, illus-

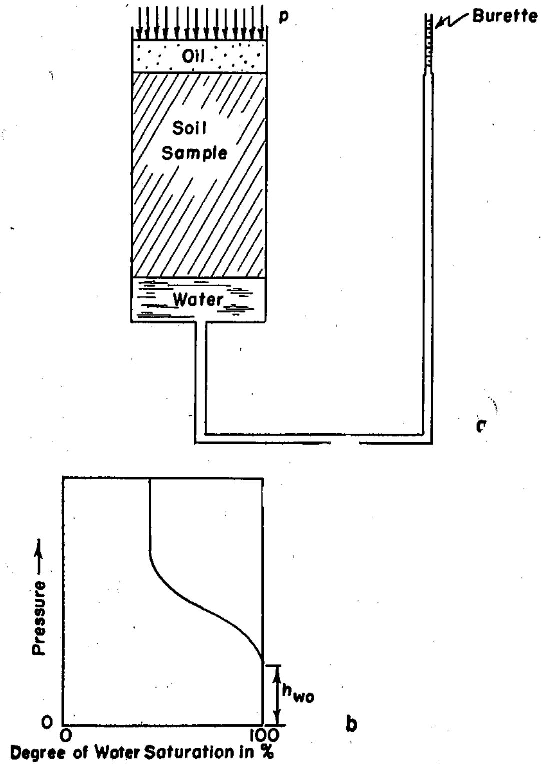


FIG. 5.

trated by e, is reached below which the void ratio continues to decrease but not at so rapid a rate. At e, the degree of saturation is 100 while at zero water content, the degree of saturation is, of course, zero. If from a plot like Figure 6a, void ratio e is measured, it can be used along with a standard consolidation test as illustrated in Figure 6b to obtain the equivalent externally applied pressure to give the same void ratio. Tests on several clays have shown that values as high as 200 to 300 lb/sq in of external pressure are required to give the same void ratio that can be obtained by drying without changing the degree of saturation as shown in Figure 6a. This means that an extremely large pressure (200 to 300 lb/sq in) is required to push water out of a saturated clay with air. If our approximate ratio of 1/2 holds, 100 to 150 lb/sq in of oil pressure is required to force water from the saturated clay. For our particu-

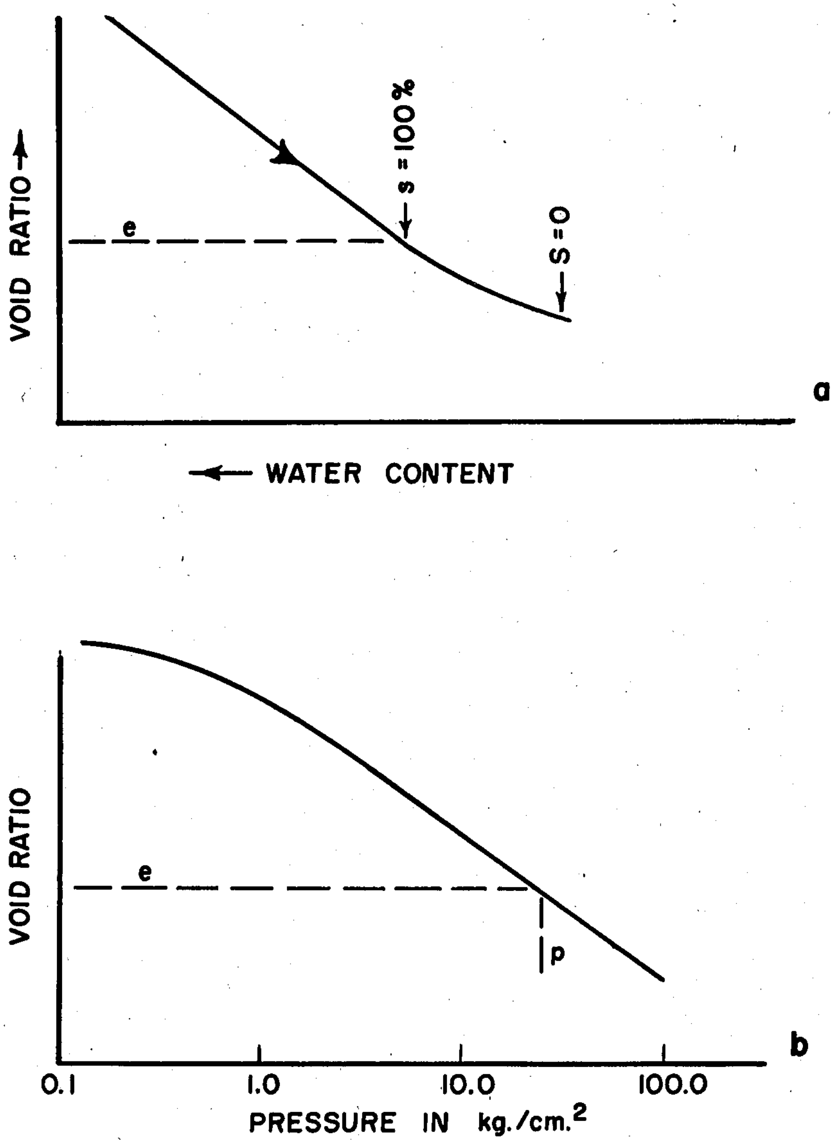

VOID RATIO
e
s = 100%
S = 0
a
WATER CONTENT
VOID RATIO
e
p
b
0.1
1.0
10.0
100.0
PRESSURE IN kg./cm.2

FIG. 6

lar problem of retention of oil by clay blanket, an oil depth in excess of 200 feet could be put in the reservoir with zero leakage.

E. *Practical Significance of Theoretical Considerations*

The preceding theoretical discussion of the permeability of soils to various fluids brings up several important and significant practical points. To design and construct the most effective clay blanket to line an oil retention reservoir, the following facts should be considered:

1. The blanket should be processed and compacted with a high polarity molding fluid, namely, water rather than oil since a more dispersed and impermeable soil structure will result.
2. The normal theoretical expression, the Carman-Kozeny equation, for relating permeant and permeability does not take proper account of all permeant properties. The polarity of the permeant has an influence on the permeability of the soil—higher permeability for a lower polarity permeant.
3. There is a critical minimum pressure which must be obtained before oil will replace water in the soil. The more dispersed and the higher the density of the soil, the higher is this minimum pressure since the smaller is the effective void wherein the oil-water interface exists. Since the permeability of soil to water is related to the same pore size, the lower the water permeability of the soil in one-phase fluid flow, the higher would be the pressure required to force water out of the soil by oil.

III. Description of the Aduana Reservoir

In the preceding section, theoretical consideration was given to the retention of oil by earth-lined reservoirs. This theory was employed in the design of several different oil storage reservoirs of various types involving different types of oil. The remainder of this paper is concerned, however, with the design and construction of the first of the oil storage reservoirs; this reservoir, the Aduana Reservoir, was built in Amuay, Venezuela, during the summer of 1955. Amuay is approximately 250 air miles westnorthwest of Caracas, the capital of Venezuela. Oil is transported via overland pipeline from the wells at Maracaibo to Amuay for refining.

The Aduana Reservoir is used to store the refined fuel oil prior to shipment via tanker to various parts of South America, Europe, and the United States. The Reservoir was formed by an earth dam, of dimensions to be presented later, joining a natural hill. A general picture of the Reservoir is given by Figure 7 which consists of 4 photographs of the dam and reservoir (taken by Creole). The area of

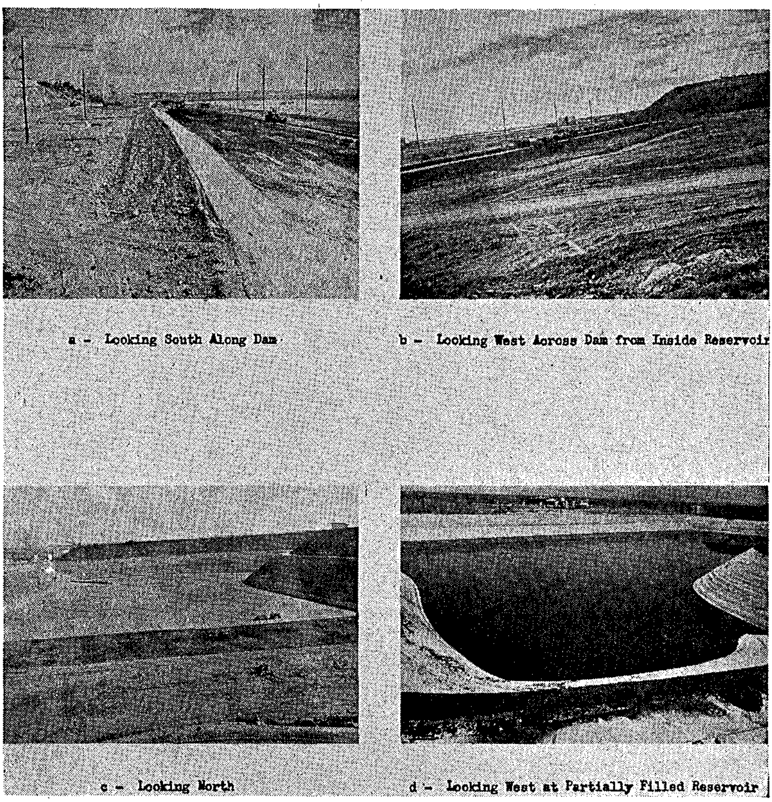


FIG. 7.—ADUANA DAM AND RESERVOIR.
(*Photographs by Creole*).

the floor enclosed by the base of the dam and the natural hill is 40,000 square meters; and the area enclosed by the crest of the dam and the natural hill is 60,000 square meters. The capacity of the Reservoir is approximately 4,000,000 barrels or 173,000,000 gallons.

The oil stored in the Aduana Reservoir is heavy fuel oil, having properties shown in Figures 8 and 9. These figures show that the fuel oil has a viscosity ranging from 4,000 to 400 times that of water and a surface tension measured at an oil-air interface of approximately half of the interfacial tension between water and air.

A. *Description of Aduana Soils*

LopVal of Caracas, Venezuela, made 19 drive-sample borings of 15 meters depth on the Reservoir site. These borings showed that

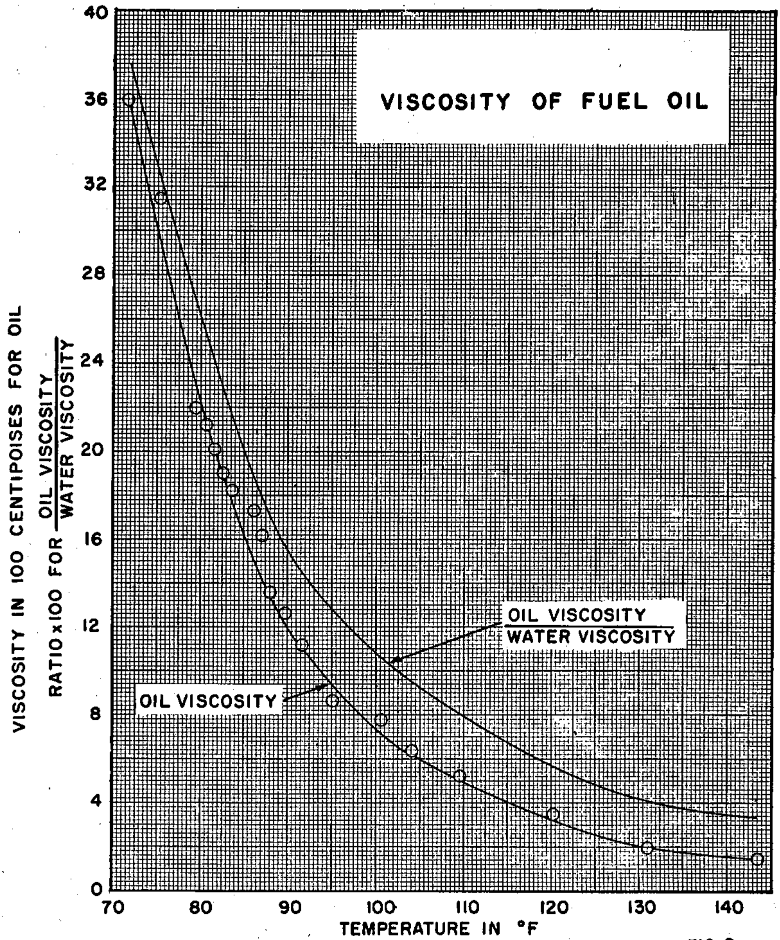


FIG. 8

the presence of a strong crust of about 5 feet in thickness, underlain by either a stratum or series of pockets of very weak clay of about 5 feet in thickness, which in turn was underlain by considerably stronger soils. A number of large diameter (2 feet ±) 8-foot deep holes were then drilled with a mechanical auger by Creole. Based on the drive-sample borings and the auger holes, several test pits were located and undisturbed samples taken. Since borrow for the dam was obtained from the floor of the reservoir and from excavation of the natural hill, samples were obtained at these locations for evaluation for borrow material.

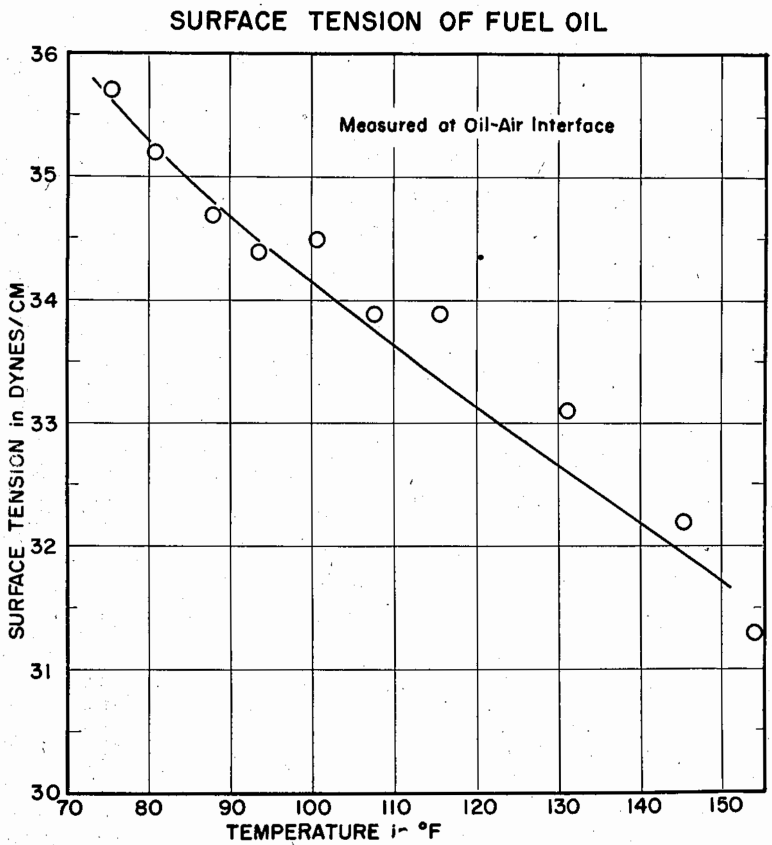


FIG. 9.

Table III summarizes classification and index tests on the soils involved in the Aduana Reservoir. Figure 10 shows a distribution of particles of these soils.

The compositional data shown in Table III for several of the soils were obtained by means of differential thermal analysis, x-ray diffractions, ethylene glycol retention measurements and potash determinations. The mineral contents are accurate to ± 5% and the iron oxide content is accurate to ± ½%.

A most interesting and not insignificant question arose—are the very soft foundation clays (Sample D in Table III) and the strong overlying clay crust (Sample A) similar except that the crust has been dried? Compositional analyses of the soils and microscopic

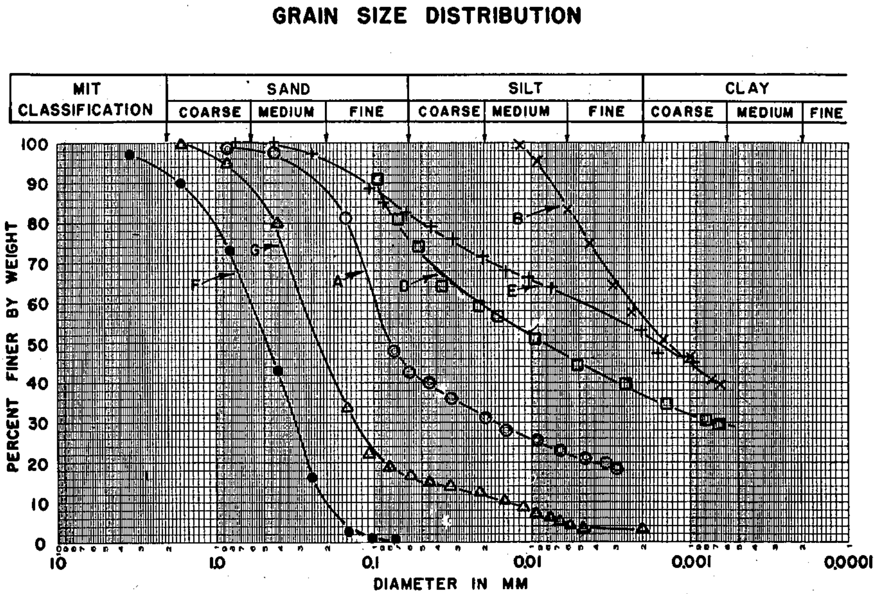


Fig. 10.

study of the coarse portions of the soils suggest the strong and weak clays are of different geological origin. The sand grains of the weak clay are rounded to well-rounded; the sand grains of the crust are angular to sub-rounded. The sand portion of the crust contains more pyroxenes and amphiboles, relatively non-resistant minerals, than does that of the weak clay. These facts indicate Sample D is an older and/or more intensely worked sediment. Investigations disclose that considerable spoil from the dredging of Amuay Bay had been dumped at the side of Aduana Reservoir some years ago. It is thought that the soft, normally-consolidated foundation clay is probably dredge spoil and the overlying crust a recent sedimentary deposit placed on top of the dredged spoil. This geological question will be studied further when the presently proposed dredging in Amuay Bay is initiated and samples of the dredge spoil can be studied and compared with the soft foundation clay.

IV. Laboratory Permeability and Capillary Tests

A. *General*

The success of any fluid storage project is predicated on a reservoir or other container which can contain the fluid in question without

TABLE III—DESCRIPTION OF ADUANA SOILS

Sample Desig-nation	Sample Location	Specific Gravity	% by Weight Finer Than			Atterberg Limits					Composition of Portion Passing No. 200 Sieve in % by Weight
			1.0 mm	0.07 mm	0.002 m	Liquid Limit	Plastic Limit	Liquid-Plastic	Shrinkage Undis-turbed	Shrinkage Remolded	
A	Undisturbed Clay from Test Pit No. 3	2.72	100	47	16	33	18	15			Quartz = 20 Dolomite = 25 Kaolinite = 15 Montmorillonoid = 13 Fe_2O_3 = 6
B	Undisturbed Clay from Aduana Cliff	2.81	100	100	57	66	24	42			Quartz = 15 Kaolinite = 25 Illite = 30 Montmorillonoid = 5 Fe_2O_3 = 3
C	Undisturbed Clay from depth 1.50 m adjacent to Boring 11	—	—	—	—	30	16	14	15.2	14.8	
D	Undisturbed Clay from depth 2.50 m adjacent to Boring 17	2.67	100	80	37	37 34 29	19 18 17	18 16 12			Quartz = 20 Calcite = 25 Kaolinite = 15 Illite = 10 Montmorillonoid = 18 Fe_2O_3 = 3
E	Typical Dam Fill—taken form Compacted Lift	2.80	100	84	52	53 49	17 18	36 31		11.7 12.2	
F	Typical Filter Sand	—	80	1	0						
G	Typical Soil—soil rejected	2.82	96	18	3	21	18	3			

excessive leakage. The feasibility of the earth reservoir to store oil depends on whether or not the soils available at Amuay could be used to make a blanket relatively impermeable to the fuel oil. The laboratory tests described in this section showed that a reservoir could be made of the soils available with essentially no leakage of fuel oil.

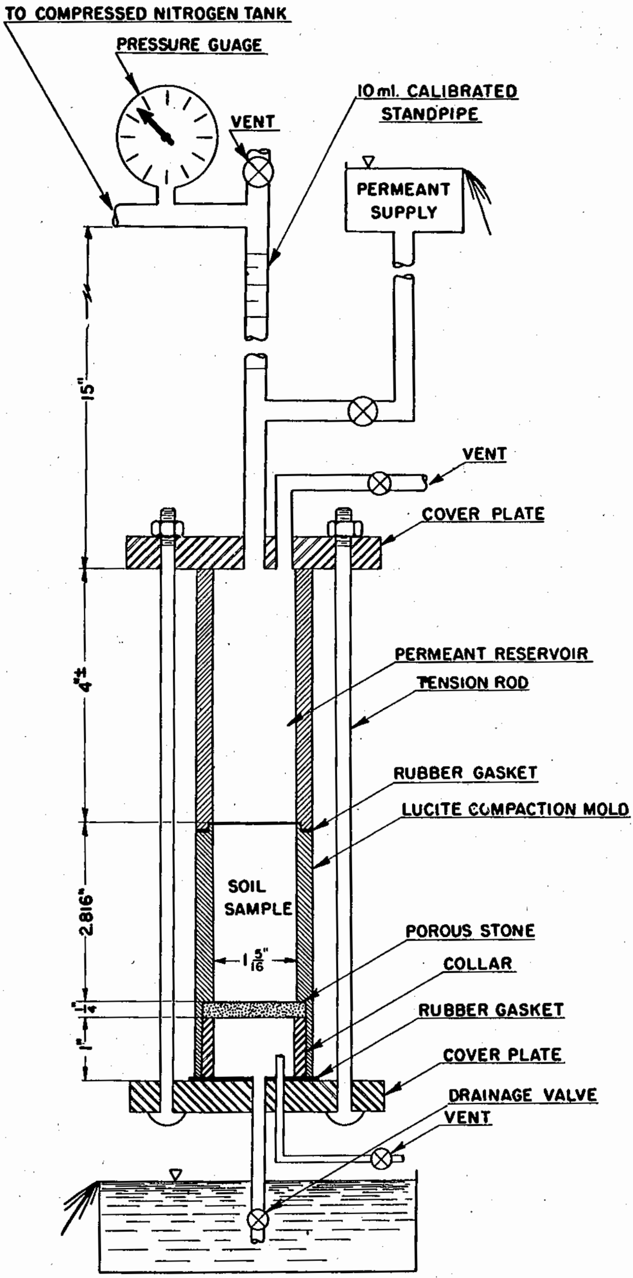
The theoretical considerations in Section II of this paper showed that the lower the water permeability of a compacted soil the lower the oil permeability of that soil and the higher the pressure required for the oil to force water out of the clay against the water-soil capillary forces. To facilitate the laboratory testing, measurements of water-permeability were made first; the results of these tests were used to guide the measurements of oil-permeability.

B. *Test Procedure*

The major laboratory test employed consisted of constant head permeability measurements on compacted samples using the setup shown in Figure 11. The soil was first brought to the desired moisture content, mixed (with or without chemical dispersant, depending on the test), equilibrated for at least 24 hours, and then compacted in the sample mold with a compactive effort approximately equal to that provided by light field equipment. The mold was mounted in the setup, the permeant reservoir and standpipe then filled with permeant, and finally, the permeability measured under any desired applied pressure. Unless noted, all tests presented in this paper employed a pressure of 25 psi, which is equivalent to a 57½ foot head of water or a 60 foot head of oil. Readings of time and permeant level in the standpipe permitted the computation of permeability.

A slight modification in the apparatus as shown in Figure 11 was necessary to measure properly the oil permeability of the compacted samples. Because of oil clinging to the sides of the burette as the oil-air interface dropped, accurate readings of permeant level in the calibrated standpipe were not easily obtained. A burette was therefore connected to the drainage valve and the fluid level measured as it rose in this exit burette.

Two soils, A and B (see Table III for a description of them) were available for construction of a blanket on the bottom of the reservoir and the face of the dam. For the following reasons, A was a better blanket material than B. B is more plastic than A and existed in hard lumps; these characteristics indicate serious problems



PERMEABILITY TEST SET-UP FIG. 11

with handling B in the field. A is better graded and has more silt and sand particles, and therefore, has more resistance to soil particle movement by forces from seeping fluids. Its lower plasticity and better grading show soil A is less expansive than B; this characteristic means A will be less likely to crack from a reduction in moisture. A was selected over B and the tests described in this section were run on A.

C. *Results of Water Permeation*

The results of the compaction-permeability tests are presented in Figure 12. They show two significant facts:

1. A very low permeability can be obtained by properly processing and compacting Soil A. A permeability as low as 3 x 10^{-9}cm/sec was obtained; 1 x 10^{-6}cm/sec is commonly considered an acceptable upper limit for blanket material.
2. The molding water content which gave the minimum permeability was approximately 18%. As has been noted on other soils, the molding water content for minimum permeability was above that for maximum dry density.

Because of the availability of sea water and the scarcity of fresh water at the Amuay Refinery, consideration was given to the use of sea water as the molding fluid for the blanket compaction. Theoretical considerations and previous experimental investigations* have shown that the sodium form of a soil is the least permeable. One would expect, therefore, that the use of sea water as a molding fluid, would result in a lower permeability of the compacted clay than would the use of fresh water. The following section of this report points out, however, that dispersants which produce sodium soil far more efficiently and completely than sea water, had only a minor beneficial effect on the permeability of the compacted clay. This fact suggests that the sea water should result in little, if any, improvement. Plotted in Figure 12 are the results of two tests run on clay samples compacted with sea water (points are indicated by triangles). These points bear out our theoretical expectation, namely, there was little difference between the permeability of the soil compacted with sea water to that compacted with fresh water; what difference existed, however, was for soil compacted with sea water to have lower permeability.

*Lambe, T. William, "The Improvement of Soil Properties with Dispersants", Journal of the Boston Society of Civil Engineers, April 1954.

COMPACTION-PERMEABILITY TEST RESULTS

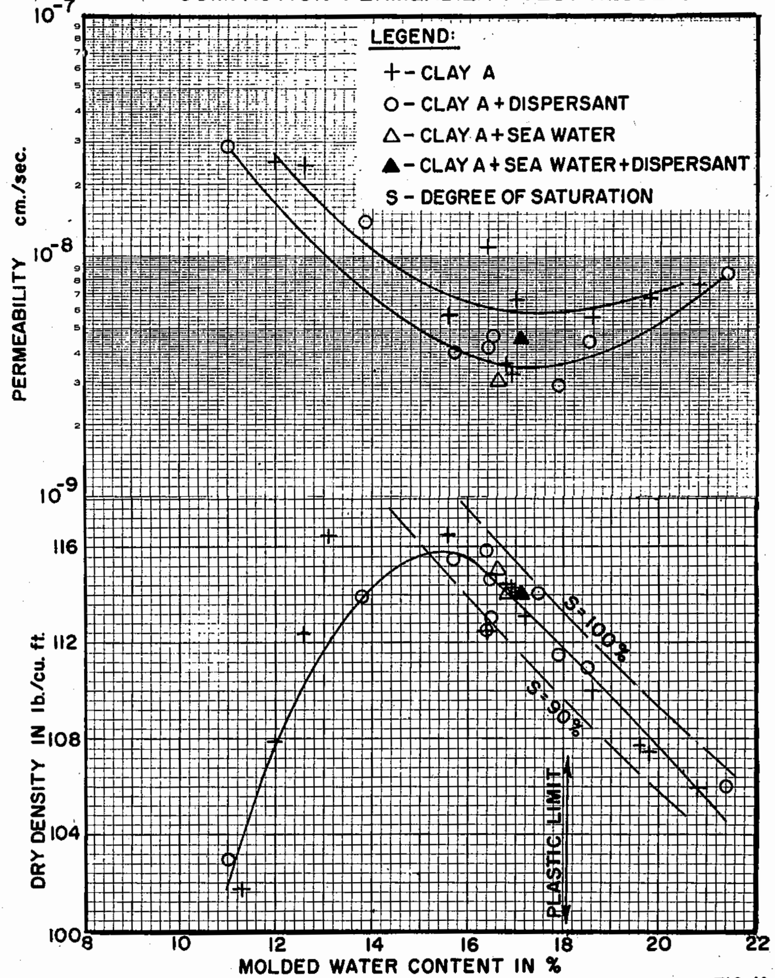


FIG. 12

D. *Results of Dispersant Treatment*

Extensive theoretical, laboratory, and field work* have shown that addition of dispersants to a fine grained soil permits it to be compacted to a higher density and lower permeability than the untreated soil. Experience has shown that a liquid limit test is usually a reliable indicator of the response of a soil to a dispersant. Liquid limit determinations were, therefore, made on Clay A with various concentrations of various trace additives. The results of these tests, given in Table IV, show that none of the trace additives had any significant effect on the liquid limit of Clay A, and in fact, the sea

TABLE IV—EFFECT OF ADDITIVES ON THE ATTERBERG LIMITS OF CLAY A

Test No.	Additive	Concentration in % Dry Soil Weight	Liquid Limit in %	Plastic Limit in %
A 1	None	—	32.6	18.1
A 3	Tetra sodium pyrophosphate	0.1	32.9	18.0
A 6	Tetra sodium pyrophosphate	0.3	32.3	16.3
A 7	Versene	0.1	36.5	16.6
A 8	Daxad 21	0.1	32.7	—
A 9	Marabond	0.1	31.7	—
A 10	Marasperse C	0.1	31.0	—
A 11	Marasperse N	0.1	30.8	—
A 12	Sodium Acid pyrophosphate	0.1	31.2	—
A 13	Sea water	20.0	30.8	—
A 14	Arquad	0.27	32.8	—

water was as effective as any of the added chemicals. Even though the screening tests on the trace additives were not particularly encouraging, several compaction-permeability tests were run on Clay A treated with various dispersants. These results, plotted along with the untreated test results in Figure 12, show that none of the dispersants had any measurable effect on the compacted dry density of Clay A. The permeability results in the upper part of Figure 12 show that the dispersants did, however, reduce the permeability at any given molding water content by a factor of two; that is, the dispersants cut the permeability in half.

Soils which are as unresponsive to dispersants as Clay A are not common. The poor response is caused by the relatively dispersed structure of the untreated clay and by the relatively high amount of ferric iron oxide present in Clay A. These polyvalent ferric cations neutralize the potential of the dispersant to sequester the exchangeable ions on the soil. The neutralization of the sequestering and/or chelating ability of the dispersant by the free polyvalent cations may

*Lambe, T. William, "The Permeability of Fine Grained Soils", American Society for Testing Materials, June 1955.

also explain the favorable effect of sea water as compared to the trace additives. Since sea water contains no anionic groups which are potent removers of polyvalent cations from the soil, the sea water is normally a relatively ineffective dispersing agent as compared to the trace materials listed in Table IV.

The limited response of Clay A to dispersants is a most interesting and important academic question but is of little practical significance in the present problem. The reasons for the limited practical importance are the facts: (1) the clay without dispersant has an extremely low permeability, and (2) sea water has a modest beneficial effect.

E. *Capillary Tests*

To confirm experimentally some of the theoretical deductions discussed in Section II, capillary tests were run on the setup illustrated in Figure 5. In each test of the first series a sample of beach sand (Soil F, Table III) was compacted to a void ratio of 0.49 in a wetting fluid. The degree of saturation in all tests was in excess of 95%. The pressure required to cause steady flow of the pore fluid from the soil was measured.

Figure 13 is a plot of applied pressure versus drainage for the three systems studied. The curves show that as the pressure is increased, the drainage increased until the critical pressure is obtained, at which point a marked increase in drainage occurs. This pressure is close to that required to initiate continued drainage or steady permeation of the non wetting phase into the soil. The critical pressure for each system is:

Symbol for Head	Wetting Fluid in Soil	Non Wetting Fluid Forcing Out Wetting Fluid	Pressure to Initiate Continued Permeation, in inches of water
h_{wa}	water	air	9 to 10
h_{wo}	water	oil	4½
h_{oa}	oil	air	6

These results confirm our theoretical predictions reasonably well since,

$$\frac{h_{wa}}{h_{oa}} = \frac{10}{6} \approx \frac{\gamma_{wa}}{\gamma_{oa}} = 2; \text{ also } h_{wo} + h_{oa} = 10\frac{1}{2} \approx 10 = h_{wa}.$$

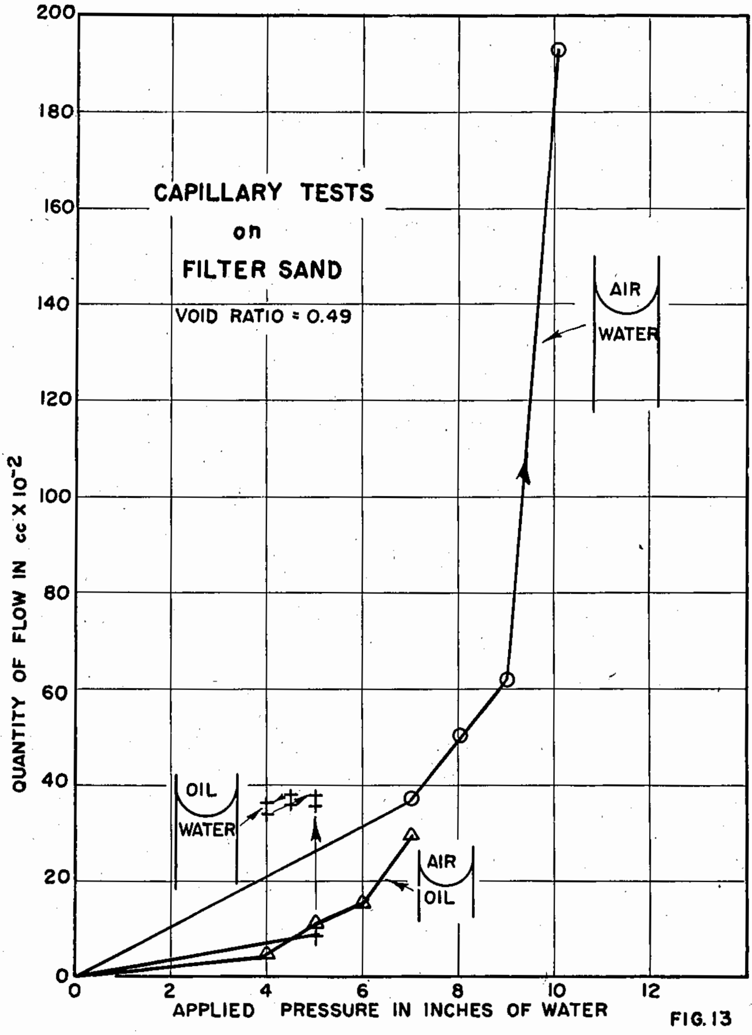


FIG. 13

To determine the pressure required to initiate oil permeation into a water wet clay, several samples of typical fill material used in the Aduana Dam (Soil E, Table III) were compacted at optimum moisture content to maximum density and then subjected to oil under pressure.

Figure 14 shows the sample from Test 28, in which 25 psi was applied, and Test P-7, in which 40 psi was applied. As the photos show, there was no penetration of oil into the clay except for a little

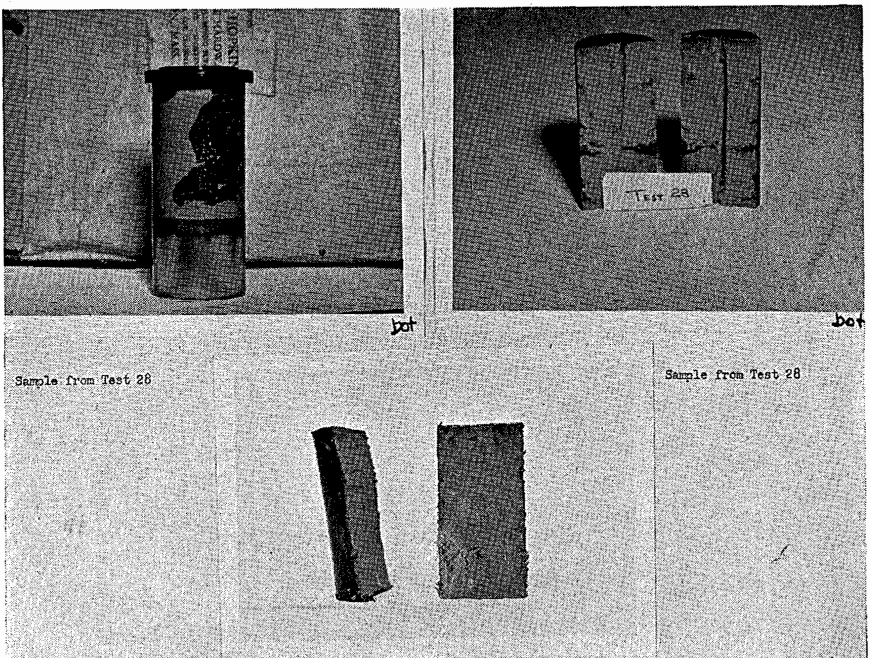


FIG. 14

in the zone between compaction lifts. Extensive flow of oil between the soil and permeameter did, however, occur. Tests 28 and P-7 were confirmed by other tests; in no tests was any oil penetration into properly compacted wet soil obtained.

These results mean that no leakage of oil will occur into a properly compacted blanket of Aduana clay until the pressure exceeds, probably by a substantial amount, 40 psi. This pressure corresponds to a depth of oil equal to 95 feet which is considerably greater than any depth of oil expected in any of the oil retaining structures.

G. *Conclusions from Laboratory Tests*

The following conclusions were drawn from the laboratory tests:

1. The Aduana clay is an excellent material for the construction of a compacted clay blanket.

2. To form the blanket, the clay should be thoroughly mixed, sea water added to bring the water content to approximately 18%, and then compacted to a high density. High moisture content, thorough mixing to break up clay lumps, and a high degree of compaction are important. The mixing and

compacting for the laboratory tests were done by hand and represent a goal for field processing. Careful field work under adequate supervision should give field results as good or better than those obtained in the laboratory since the better field compaction (see Section VI) should offset the superior laboratory mixing.

3. Sea water is a better molding liquid than fresh water.

4. The dispersants investigated gave only modest beneficial effects. Their use was recommended only if work in the field indicated that the dispersants facilitated the processing of the clay and if the supply of blanket clay was low.

5. The clay blanket should be kept moist at all times. After placement of the clay blanket, the surface should be covered with four to six inches of gravel to reduce evaporation and to give structural protection to the clay blanket. When the reservoir is empty, spraying with water or ponding of water is desirable and may be necessary if the climate is particularly hot, dry, and windy. Should the blanket dry out, it should be thoroughly wet, preferably with sea water, prior to permitting the oil to come in contact with the blanket.

6. The pressure required to initiate the penetration of oil into the wet clay blanket far exceeds any pressure to be expected from operation of the reservoir. No leakage of oil will occur from the reservoir, therefore, if the blanket is properly constructed and maintained.

V. Strength of Soils Under Dam

A. *Samples Used in Tests*

LopVal of Caracas made 19 drive-sample borings, each approximately 15 meters deep. In each meter of depth, LopVal took a 30 cm-long sample in a 2-inch diameter spoon; the number of blows* with a 136 kg. hammer "freely" dropping 35 cm to drive the spoon a distance of 30 cm was recorded.

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*The penetration test used in the United States records the number of blows of a 140 lb. hammer "freely" dropping 30 in. required to drive a 2″ in diameter and 2′8″ in length spoon a distance of one foot. Even though the LopVal procedure employs the same driving energy per unit of penetration (350 pounds) as the United States method, it is more severe driving because of the heavier weight. Another important difference between the United States and LopVal tests is caused by the unequal lengths of the sampliers. The United States samplier, 32.0 long, has almost three times as much lateral area as does the LopVal one, 11.8 inches long.

For these two reasons—more severe driving and shorter samplier—the Lop-Val test should show a lower penetration resistance than the United States test in the same soil.

Because of several theoretical and practical considerations, however, no common driving test should be used other than as a rough approximation of soil strength. The unreliability of such testing, especially in weak soils, is illustrated by the following Aduana results: Sample D with a 3-blow penetration resistance was actually a little weaker than Sample C which showed a resistance of zero blows.

A thorough study of the boring logs plus the visual examination of the subsoil exposed by Creole's auger holes showed:

1. The critical area to be covered by the dam was in the vicinity of Station 2 + 00.

2. The weakest part of this critical zone was a stratum, or series of pockets, of very soft clay extending from depth 1.5 m to 3.0 m.

This section shown in Figure 27 was used in the stability and settlement analyses. Settlement measurements of the dam during construction showed that maximum foundation consolidation occurred near Station 2 + 00 — encouraging evidence that this was indeed the zone of weakest subsoil. Even though all of the borings in the critical zone did not show the very weak clay, the stability studies are based on a stratum of weak clay underlying the entire width of the dam. Since a local slip failure could occur, to base the design on a continuous zone of weak soil was thought necessary.

Two large (approximately 1 foot cube) samples of the weak clay were cut by hand from pits and sent to the Massachusetts Institute of Technology Soil Engineering Laboratory for tests. One of these undisturbed samples, D, came from a depth of 2.50 m. adjacent to Boring 17; the other, C, from a depth of 1.50 m. adjacent to Boring 11. LopVal showed a driving resistance of 3 blows at the location of D and 0 blows at that of C. Actually, D and C had approximately the same strength with C a little stronger. Since Sample D was in better condition, weaker, and at the exact zone used for the stability studies, most of the strength testing was done on it.

An undisturbed sample, A, of the crust overlying the soft clay was obtained for the permeability studies described in Section IV of this paper. A specimen from this sample was tested for strength.

The drive-sample borings indicated the soils underlying the weak clay consisted of medium to stiff clays (greater than 15 blows per foot). No samples of these deeper and stronger soils were taken for laboratory tests. Examination of these soils in the auger holes showed them to be far stronger than the weak clays tested.

B. *Methods Used to Measure Soil Strength*

Three laboratory tests were employed to measure the strength of the undisturbed foundation soils, namely: (1) Unconfined Compression test, (2) Triaxial Compression test, (3) Triaxial Compression test with measured pore water pressures.

1. *Unconfined Compression Test*

In the unconfined compression test, a cylindrical soil sample, approximately 3.5 inches long and 1.4 inches in diameter, is failed by an axial load. No lateral load is applied to the sample. Readings of load versus deformation permit the computation of stresses and strains. One half of the maximum compressive stress is approximately the shear strength of the sample.

In Figure 15 are typical stress-strain curves for unconfined tests on an undisturbed sample (U-1) and on a remolded sample (U-1R).

2. *Triaxial Compression Tests*

In the triaxial test the sample is enclosed by a rubber membrane; a cylinder filled with water surrounds the sample. Any lateral pressure, i.e., chamber pressure, can be applied to the sample by subjecting the water to air pressure. By this means, the sample can be tested under conditions which will exist when load is added to the foundation by the dam. Two sample sizes, 3.5 inches long by 1.4 inches in diameter and 6.5 inches long by 2.8 inches in diameter, were used.

Figure 15 shows a typical stress-strain curve for a triaxial test (T-7). The shear strength, s, can be computed from

$$s = \left(\frac{P}{2\,A}\right)_{max} \cos \phi$$

where $\left(\frac{P}{2\,A}\right)_{max}$ is half of the maximum compressive stress and ϕ is the friction angle of the soil.

3. *Triaxial Compression Tests with Measured Pore Water Pressures*

In the normal undrained triaxial test one cannot determine the stress carried in the pore water and is thus unable to compute the friction angle of the soil. If a pilot is inserted in the sample, however, measurements of pore water can be made during shear. One such test, T-3, was run.

Test T-3 was unusual, not only because of the measured pore

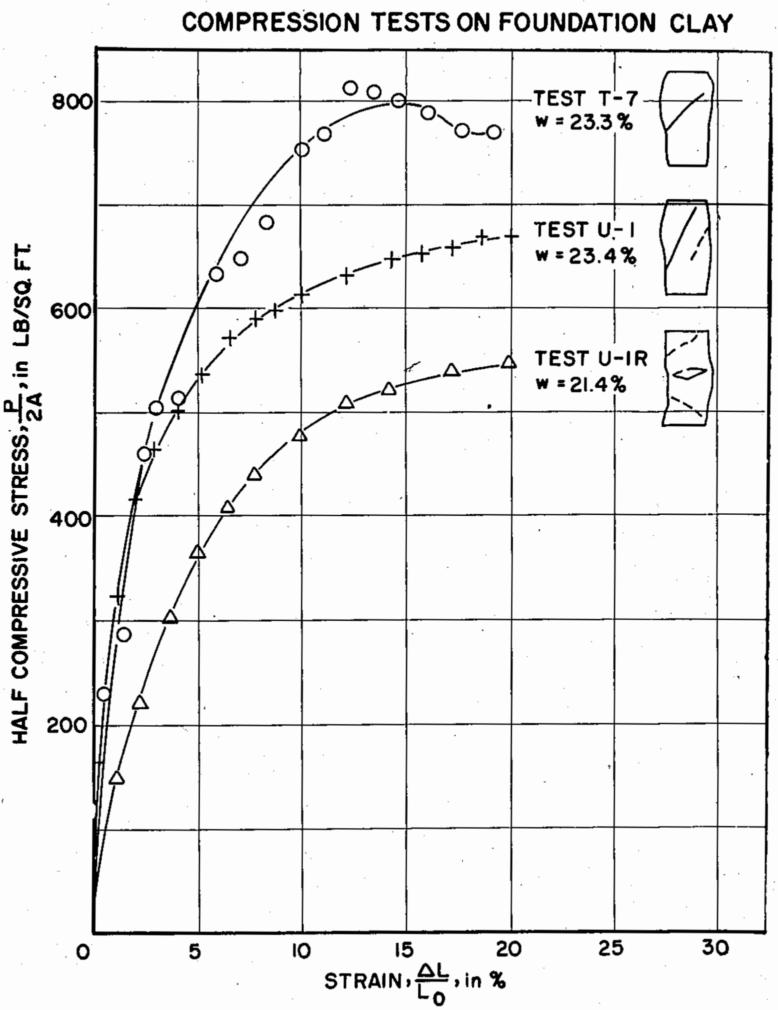

FIG. 15.

water pressures, but also because the chamber pressure was altered four times during the test. By means of this multiple-stage testing, a measure of the strength change with increased confined pressure can be obtained.

The results of T-3 are presented in Table VI and Figure 16.

TABLE VI—STRENGTH OF FOUNDATION SOILS

Sample	Test No.	Type of Test	Consolidation Pressure in lb/sq ft	Half Compressive Strength in lb/sq ft	Shear Strength in lb/sq ft	Strain at Failure in %	Water Content in %	Comments
C — Adjacent to Boring 11, depth = 2.5m	U-1a	U	0	500		3	22.1	
	U-2a	U	0	470		3	20.2	
	T-1a	Qc	1350	2360		16	21.4	
D — Adjacent to Boring 17, depth = 1.5m	U-1	U	0	600		10	23.4	
	U-1R	U	0	540		15	21.4	
	U-2	U	0	650		15	23.2	
	U-2R	U	0	450		15	22.8	
	U-3	U	0	380		2	22.8	Failure around stone, poor test
	U-3R	U	0	515		15	23.9	
	T-1	Qc	607	552	480	20	26.2	
	T-2	Qc	3960	1130	980	17	23.5	
	T-3	Qc	1440	580	505	3	21.0	
	T-5	Qc	5000	1640	1420	6	19.4	
	T-6	Qc	1000	475	415	18	21.6	
	T-7	Qc	2750	815	705	12	23.4	
	T-8	Qc	2750	1250	1085	14	19.0	
	T-8R	Qc	2750	1130	980	19	18.5	
	T-9	Qc	607	730	635	12	23.3	
	T-10	Qc	4040	920	800	3	20.5	
A — From surface crust	U-4	U	0	2020		2	14.6	

Notes: U stands for unconfined compression test.
T stands for triaxial compression test.
R stands for remolded sample.
Qc stands for triaxial test in which sample is consolidated, then sheared with no drainage.
In T, shear strength = half compressive strength $\times$ cos ϕ where $\phi = 30°$.

TRIAXIAL COMPRESSION TEST ON FOUNDATION CLAY

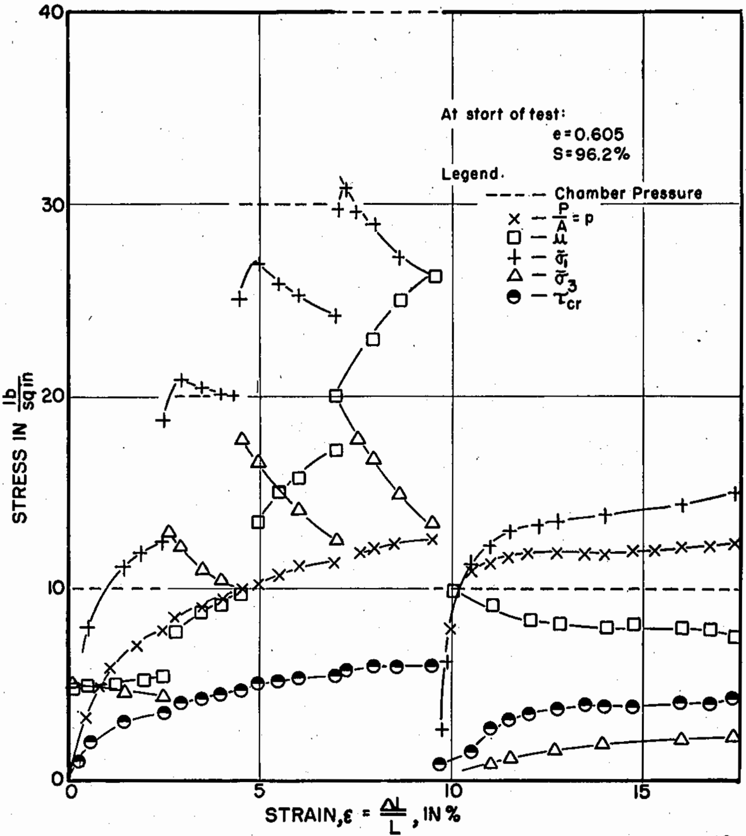

FIG. 16

C. *Results of Strength Tests*

1. *Dependability of Test Results*

In addition to giving a measure of the strength of the weak foundation clay, the laboratory tests brought out several interesting and important characteristics of the clay. Before these are discussed, a few comments on the dependability of the test results are in order.

The clay samples were hand cut from a pit; this procedure is generally acknowledged as the one giving the soil samples with the least amount of disturbance. The samples were air shipped from Amuay to the Massachusetts Institute of Technology. While such a long trip with considerable handling undoubtedly caused some soil disturbance, the generous use of packing material minimized damage. Examination of the samples, the condition of the packing and container, indicated the samples were in excellent condition.

As will be seen in Figures 17, 18, and 19, the data scatter. Scatter is, unfortunately, nearly always present in test results on undisturbed soil. Even the most homogeneous appearing soils are usually nonhomogeneous and nonisotropic. The Aduana clays contained sand lenses and pockets, and occasional rocks. That these interfered with some of the tests can be seen (Table VI) from Test U-3 in which the sample failed around a stone.

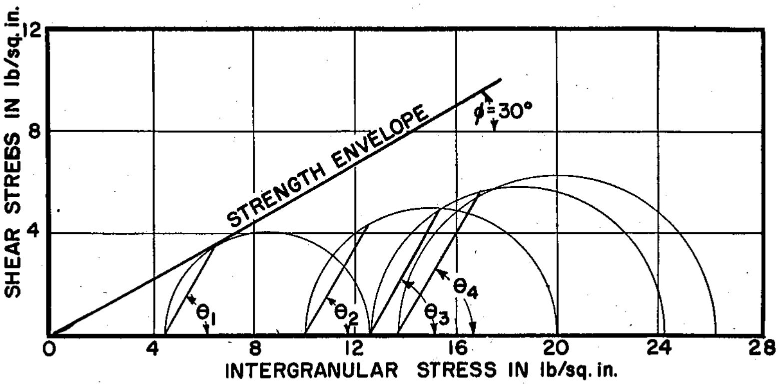

FIG. 17

Commonly accepted test techniques were employed. Only one feature of testing procedure warrants question; this is the time allowed for sample consolidation in the consolidated-undrained (Q_c) tests. Because of the extremely low permeability of the clay, more than the normal time was required for the dissipation of the excess pore water pressures. Incomplete consolidation is probably the reason the strength values at the higher pressures of confinement are lower than anticipated, i.e., fall below the apparent envelope in Figure 18. The lack of undisturbed soil prevented study of the effect of consolidation time.

In summary, the test results are thought to represent accurately the strength of the weak foundation clay. The strength values for consolidation pressures greater than 3000 lbs/sq ft are probably conservative.

2. *Structure of Weak Clay*

All of the test results indicate that the weak clay has a structure* which is insensitive to deformation. This fact means that the clay will not lose a significant portion of its strength from any disturbance or from progressive strain. A comparison of strengths on undisturbed and remolded samples shows the loss of strength on remolding is negligible; as shown by the following test data:

Test No.	Undisturbed Strength in lb/sq ft	Remolded Strength in lb/sq ft	Undisturbed Strength Remolded Strength
U-1	600	540	1.1
U-2	650	450	1.4
T-8	1085	980	1.1

The ratio of undisturbed strength to remolded strength, usually termed "sensitivity", is very low.

Another indication of the insensitivity of the Aduana clay is the absence of a pronounced peak on the stress-strain curves, as illustrated in Figure 15. The strains at peak compressive stress are relatively large for those tests where any peak at all was obtained.

The lack of a "sensitive" structure means that clay will be relatively uneffected by distortions, shocks (such as from an earthquake) or progressive movements; therefore, an unusually large factor of safety against rupture is not required in the dam design.

3. *Strength as Function of Pressure*

As noted earlier, the reason the more complicated triaxial test was employed was that the simple unconfined test could not give information on the effect of applied pressures on strength. The unconfined test approximates, in other words, the in situ strength of the clay at the time of sampling. The triaxial test was needed to indicate the strength of the weak clay during and after construction of the dam.

A series of triaxial tests were run in which different consolidation pressures were employed; for example, the sample in T-6 was consolidated under a uniform confining pressure of 1000 lb/sq ft prior to shear, while the sample in T-8 was consolidated under

*The "Structure" of a clay is the degree of orientation of, and the magnitude of forces between, adjacent particles.

2750 lb/sq ft. The results of the triaxial tests are plotted in Figure 18, the unconfined tests are plotted at zero consolidation pressure.

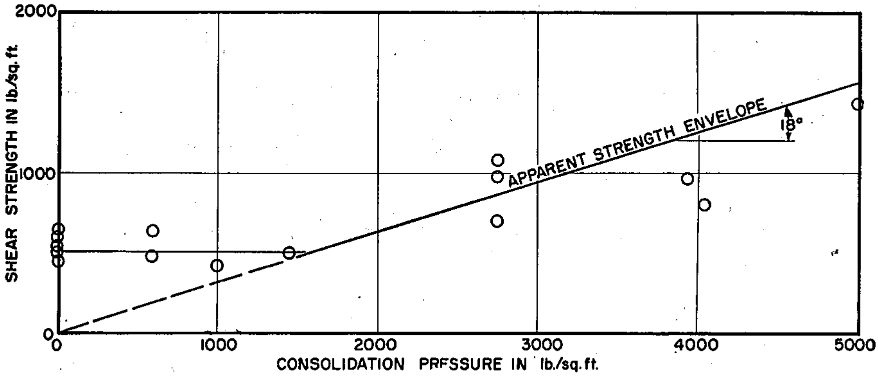

FIG. 18

Sample D came from a depth of 2.5 m, which means it had an overburden pressure ranging from 600 to 1100 lb/sq ft. When the water table existed within a foot of ground surface, as it did below the dam at the time of construction, the pressure was 600 lb/sq ft; if the water table was below the sample, the pressure was 1100 lb/sq ft.

The apparent strength envelope in Figure 18 has a break at a consolidation pressure of approximately 1500 lb/sq ft. The consolidation test data, discussed later in this paper, show a maximum past consolidation pressure of approximately 1400 lb/sq ft. These data indicate that sometime in its history, the weak clay stratum was subjected to a load greater than the overburden existing at the time of sampling. This slight precompression could have been caused by an additional few feet of overburden which had been removed prior to sampling or by capillary pressures resulting from a dropping of the water table to a level considerably below its present location, or by limited dessication.

The apparent envelope shows the magnitude of the clay strength at any consolidation pressure up to 5000 lb/sq ft. Thus, for example, when the clay has fully consolidated under a pressure of 5000 lb/sq ft from the weight of overlying dam, the strength will increase from approximately 500 to 1500 lb/sq ft.

For design purposes, a more convenient form for the data in

Figure 18 is the plot in Figure 19 where the consolidating pressure has been expressed in terms of overlying dam height. This relationship is shown by the right line in Figure 19. The unit weight of

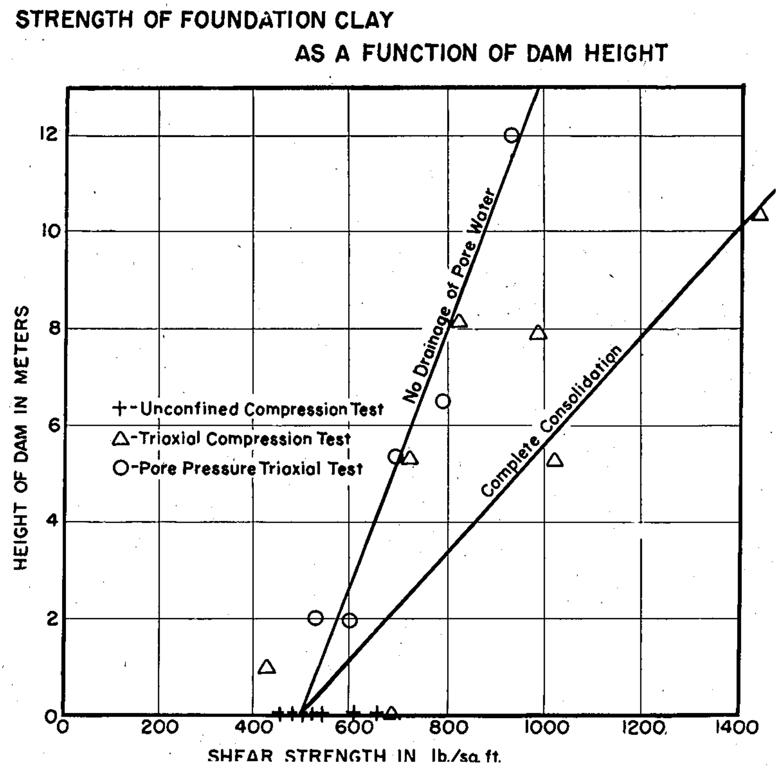

FIG. 19.

overburden employed in the preparation of Figure 19 was the total soil unit weight of 135 lb/cu ft. This use implies that all pore fluid pressures have been taken as zero in the computation of soil pressures. The permeability studies justify this neglect of the effects of seepage.

Unless the dam is to be built at a very slow rate, the strength will not increase along the right line in Figure 19 during construction. What then is the strength at various dam heights before drainage starts? To answer this question, Test T-3, a multiple stage triaxial test with measured pore water pressures was run. In Figure 16, are plotted the data from Test T-3.

The sample in T-3 was consolidated under a pressure of 1440 lb/sq ft. The sample was compressed almost to failure then the chamber pressure was successively changed to 2880, 3320, 5760, and finally, 1440 lb/sq ft with no drainage of pore water permitted. At each pressure, a measurement of compressive strength was made.

Figure 16 shows a very encouraging and important fact—the weak Aduana clay increases significantly in strength with an increase of confining pressure even though no change in moisture content occurs. This strength increase at constant moisture content does not occur in saturated clays, but is common in partially saturated clays like the Aduana. The Aduana clay was over 96% saturated yet behaved as a partially saturated soil.

In Figure 17 the final stress system for each state is plotted. This figure shows the true friction angle of the clay to be 30°—actually a little greater, since complete failure was not reached in the first stage. The critical shear stress, i.e., shear strength, is the shear stress on the plane where the intergranular stress makes the maximum obliquity angle; this plane is at an angle with the horizontal of θ, which is equal to $45° + \frac{\phi}{2}$. The strengths are indicated for the four stages. The true friction angle of 30° was used to compute the shear strength in the triaxial tests, i.e., strength = half compressive strength $\times$ cos 30°.

The results of Test T-3 are plotted in Figure 19 to indicate the strength of the foundation clay at constant water content as a function of overlying dam height.

4. *Strength of Foundation Clay*

Figure 19 is a plot of the strength of the weak foundation clay. The left line represents the minimum strength for any height of dam; the right, the maximum. The lowest strength, below 500 lb/sq ft, exists in the clay prior to construction. When the dam at any location is twelve meters high, the soft foundation clay directly under the twelve meters of dam has a strength between 840 and 1600 lb/sq ft. The percentage of the strength increase from 840 to 1600 which has occurred at any given time is approximately equal to the percentage of the expected settlement that has occurred. The measured settlement at the end of construction was approximately 1/3 of the predicted ultimate; at this time 1/3 of the strength increase from consolidation had occurred.

VI.—Compaction of Dam and Blanket

A. *Compaction Required*

To perform properly, the dam and the reservoir floor must be impermeable enough to contain the stored oil and the dam must be strong enough to resist structural failure. Section IV showed that only a moderate degree of compaction at a high water content was necessary to make the blanket, placed on the floor of the reservoir and the inside face of the dam essentially, impermeable to oil. Because of the weak foundation clay, a high strength in the dam was needed. Table VII is a summary of the specifications for the selection and compaction of soil for the dam and blanket.

In the design computations, a shear strength of 2000 lb/sq ft for the dam was used. The primary object of the compaction control was, therefore, to obtain a dam of this strength. Because of the compressible foundation, the dam should also be able to withstand moderate deformations without cracking. The fill should be placed at a high enough density to minimize settlements within the dam proper. In other words, high strength, high flexibility and low compressibility were goals of the field control of fill placement in the dam. The rest of this section presents laboratory test results and field test data which show that these goals were obtained.

B. *Laboratory Test Procedures*

The test procedures described in the writer's book* were employed in the compaction and strength tests. Each sample was brought to the desired water content and then equilibrated for 24 hours at that water content prior to compaction. During the first part of the test program, an unused sample of soil was employed for each test. After a number of tests showed that compaction data on reused samples were apparently the same as those on fresh soil, samples of reused soil were employed.

On most of the compacted specimens, the resistance to the penetration of the Proctor needle was recorded; the sample was then subjected to unconfined compression tests.

*T. William Lambe, "Soil Testing for Engineers," John Wiley and Sons, 1951.

TABLE VII—DAM AND BLANKET SPECIFICATIONS SUMMARY

Zone	Soil Specifications		Compaction Specifications		Comments
	Particle Size	Liquid Limit	Density	Water Content	
Dam Proper	Max. = 3 inches	Max. = 40%	90% of Max.	from optimum to opt. — 2%	Do not permit cracking from drying. Important not to compact wet of optimum.
Floor Blanket	Max. = 1 inch	Min. = 30%	90% of Max.	from optimum to opt. + 2%	Thoroughly mix soil before compacting. Keep moist.
Up Stream Blanket	Max. = 3 inches	Min. = 30%	90% of Max.	Optimum + or — 1%	Keep moist.

Notes: All maximum compacted densities and optimum water contents refer to modified AASHO compaction test.
All compaction in lifts of 6 inches compacted thickness.

C. *Results of Laboratory Tests*

Figure 20 summarizes the laboratory test data; compacted dry density (the weight of dry soil divided by volume of compacted soil), half unconfined compressive strength, and penetration resistance are each plotted against molding water content. As noted in Figure 20, six compactive efforts* were employed; curves 1 through 4 from tests in the miniature mold ($1\frac{5}{16}$ inch diameter x 2.816 inches long), and curves 5 and 6 from tests in the "standard" size mold (4 inches diameter x 4.6 inches). In order of increasing compaction degree, the tests progress as 1, 2, 5, 3, 4, and 6; tests 1 and 2 give essentially the same results and are, therefore, considered as one.

The curves show:

1. The greater the compactive effort, the higher the maximum density and the lower the optimum water content.
2. Higher strength results from compaction at the lower water contents.

Figure 20 shows characteristically the influence of water content and compactive effort on density, but strength, the most important variable, can be better studied from other plots of the data; these plots are presented and studied in the following pages.

D. *Relation of Penetration Resistance to Compressive Strength*

The compression test is the most dependable method of measuring the strength of cohesive soils. A penetration test is, however, much easier to perform than a compression test in either the laboratory or the field. Many years ago the penetration test lost favor as a means of measuring strength; since World War II, penetration testing, e.g., (CBR, and cone penetration) has become widely used and accepted. In soils without large particles to invalidate measurements, penetration tests can be reliable if they are correlated with the results of strength data on the actual soil under consideration (and consideration is given to the size of the tip of the penetration device).

To determine the relationship between compressive strength and penetration resistance on the Aduana soil, the strength data were cross plotted in Figure 21. This figure shows that at the higher water contents, there is an approximate linear relation between penetration resistance and compressive strength. For the high water con-

*The compaction in the miniature tests was from a squeezing with a spring-loaded tamper; that in larger tests was from a blow by a falling hammer. The type of compactive effort, as well as amount of effort, was, therefore, a variable.

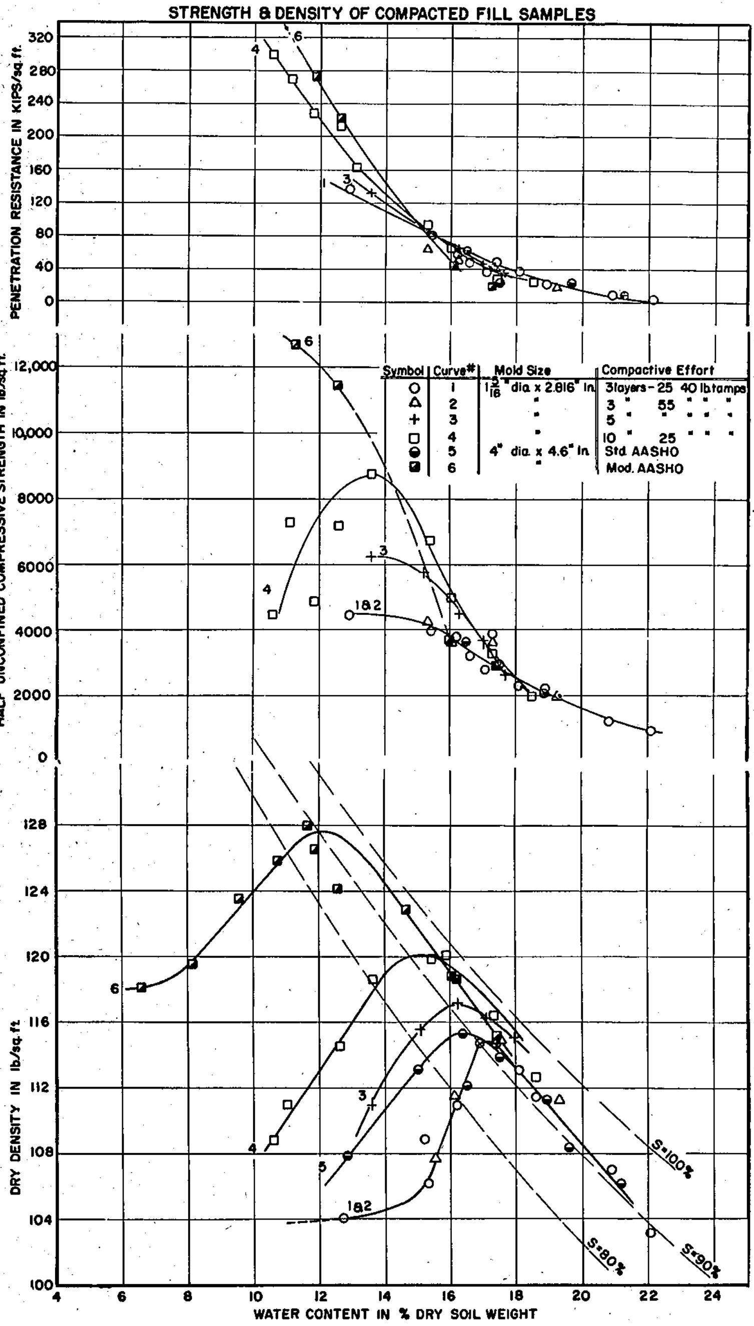
STRENGTH & DENSITY OF COMPACTED FILL SAMPLES
PENETRATION RESISTANCE IN KIPS/sq.ft.
HALF UNCONFINED COMPRESSIVE STRENGTH IN lb/sq.ft.
DRY DENSITY IN lb/sq.ft.
WATER CONTENT IN % DRY SOIL WEIGHT
Symbol | Curve# | Mold Size | Compactive Effort
1 | 1 5/16" dia. x 2.816" In. | 3 layers - 25 40 lb. tamps
2 | 3 " 55
3 | 5 "
4 | 10 " 25
5 | 4" dia. x 4.6" In. | Std. AASHO
6 | Mod. AASHO
S=100%
S=90%
S=80%

FIG. 20

tents, the shear strength (half of unconfined compressive strength) is approximately 0.08 of the penetration resistance; in other words, the penetration resistance is $\frac{1}{0.08}$, which is equal to 12.5, times the shear strength. The semi empirical equation* for the bearing capacity, q, of a round footings on clay of strength, s,

$$q = 7.4 \text{ s},$$

suggests a penetration resistance of 7.4 s. Since the needle resistance is measured at strains which are relatively greater than those at which foundation "failures" are usually defined, the experimental value of 12.5 agrees well with the theoretical value of 7.4.

Cone penetration tests were also run on compacted samples; the cone results were in close agreement with the needle results. The cone resistance—half compressive strength plot was much nearer a straight line, however.

Below optimum water content, the unconfined compression test is not a good indication of strength on the Aduana soil, since the dry soil tends to pelletize and not compact uniformly.

Figure 21 indicated that the penetration test, along with moisture determinations, could be used for the control of compaction of the Aduana clay.

E. *Relation of Density and Strength*

Figure 22 presents density versus strength for a number of water contents. This figure shows a very important but little recognized fact†: While an increase in density results in an increase in strength at a low water content, an increase in density can actually weaken wet soil. This fact means that compaction (past a certain value) of a wet soil is of little benefit and can be detrimental.

F. *Effect of Overburden on Dam Strength*

Tests on the undisturbed foundation clay showed that the strength of the clay increased with an increase of confining pressure, even though no escape of moisture from the soil was permitted. To

*Equation (29:11), page 172, Terzaghi and Peck, "Soil Mechanics in Engineering Practice," John Wiley and Sons, 1948.

†Tests at the Waterways Experiment Station (Charles R. Foster, "Reduction in Soil Strength with Increase in Density", Proceedings of ASCE, July 1953) and tests elsewhere have clearly shown this striking fact.

HALF UNCONFINED COMPRESSIVE STRENGTH vs PENETRATION RESISTANCE

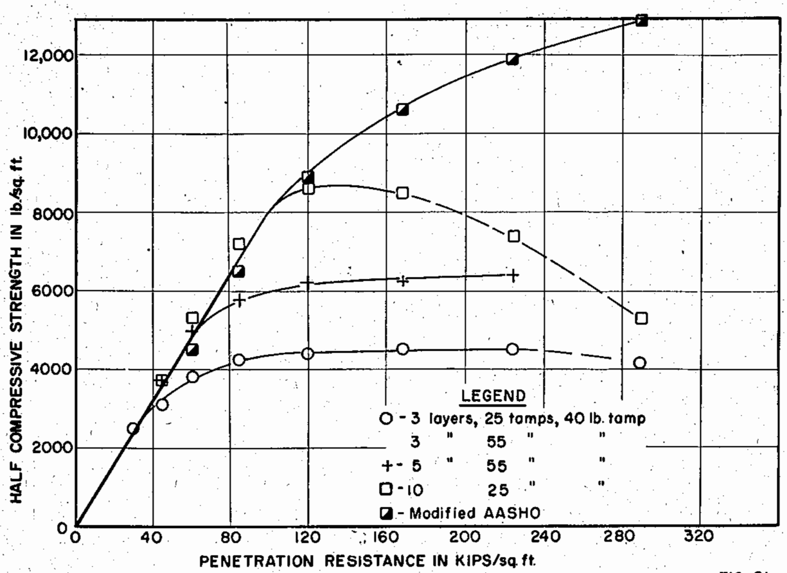

FIG. 21

STRENGTH AS A FUNCTION OF DENSITY

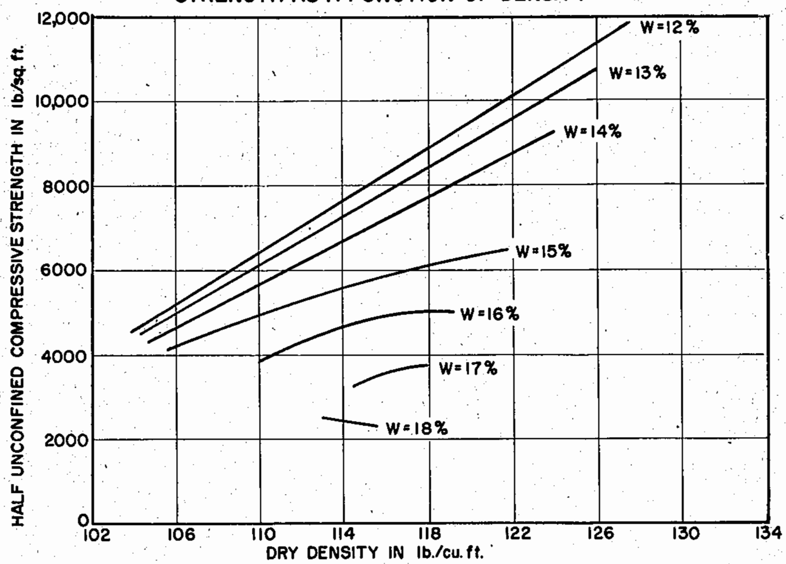

FIG. 22.

determine the effect of confinement without drainage on the compacted fill, multiple-stage, undrained, triaxial tests were run on three specimens—one dry of optimum, one at optimum, and one wet of optimum. The test data are plotted in Figures 23 and 24.

Figure 23 illustrates a characteristic shown in all the strength tests, namely: At all but the low water contents, the strains at failure are relatively large. In other words, the compacted clay has sufficient flexibility to withstand significant deformations without cracking.

COMPRESSION TESTS ON COMPACTED FILL

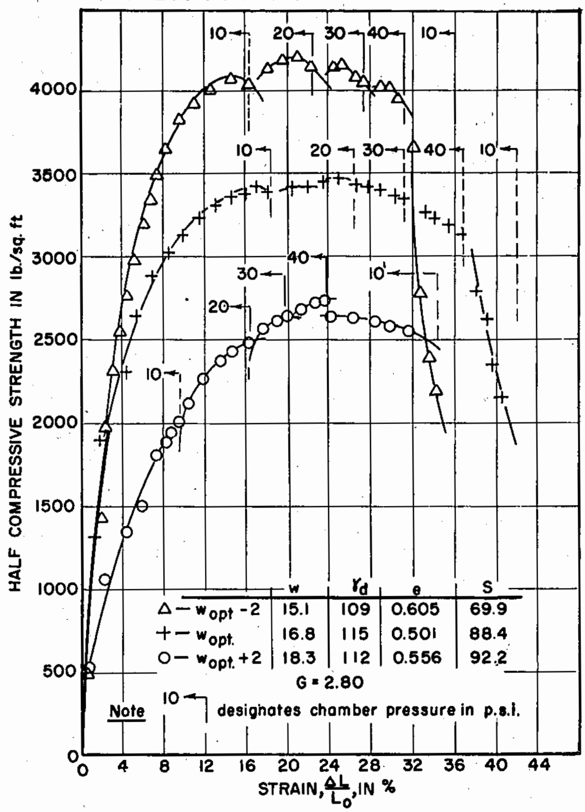

Fig. 23.

Figure 24 shows that the strength of the compacted soil does increase with confinement pressure but that this increase is modest compared to that of the undisturbed foundation clay.

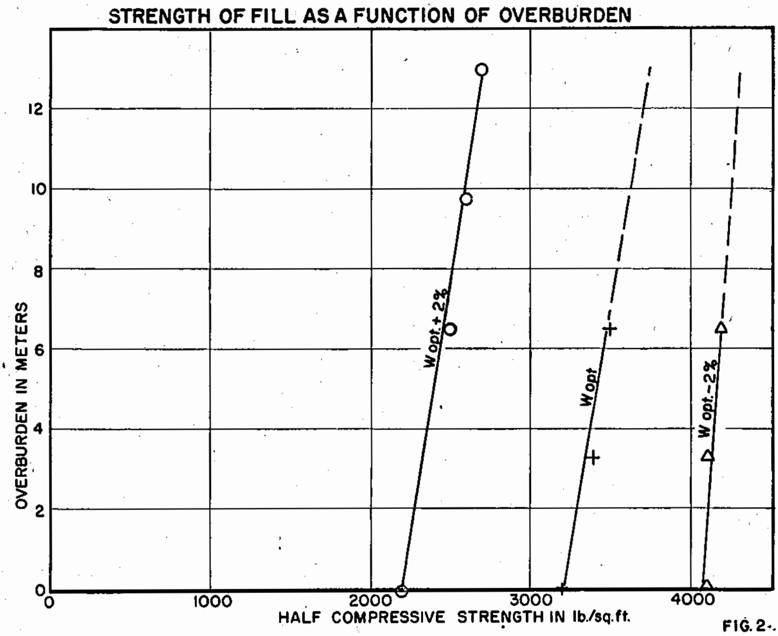

FIG. 2-.

G. *Summary of Laboratory Compaction Tests*

The laboratory compaction test results presented in Figure 20 and discussed in the preceding pages permit a number of conclusions for the Aduana clay:

1. The greater the compactive effort, the higher the maximum density and the lower the optimum water content.
2. At low water contents, an increase in density results in more strength; at high water contents, an increase in density can cause a strength loss.
3. The penetration test is a useful tool to aid in compaction control in the field.
4. The compacted soil undergoes a slight strength increase with increased overburden even before drainage occurs.
5. At water contents near optimum, the compacted soil is very plastic.
6. At all compactive efforts and at all moisture contents less than 18%, the shear strength is greater than 2000 lb/sq ft—the value used in the design of the dam.

H. *Field Compaction*

The Contractor employed two sheepsfoot rollers and one rubber-tired roller. All three of these rollers were small and therefore had to make many passes to give densities approaching those specified. The Contractor's earth moving equipment (14 cubic yard capacity scrapers and "turnapulls") were relatively large and heavy. The tires (containing pressures of 34 to 40 psi) of this equipment were visibly achieving better compaction than the rollers. Most of the effective fill compaction was, therefore, done with loaded earth-moving equipment.

Considerably more efficient compaction could have been obtained with heavier rubber-tired rollers. Rollers having four wheels abreast, each wheel carrying between 12,000 and 25,000 lbs and having tire pressures as large as 150 psi are common in the United States. The use of such rollers greatly reduces the number of required passes and increases the permissible thickness of lift (up to 24 inches).

Creole's engineers made a number of measurements of field density and moisture content. These data, along with the laboratory curves from Figure 20, are presented in Figure 25. Figure 25 shows several interesting and important facts.

The modified AASHO density curve (#6) is the laboratory test that best represents the field compaction employed. The compaction

FIELD DENSITIES OF FILL

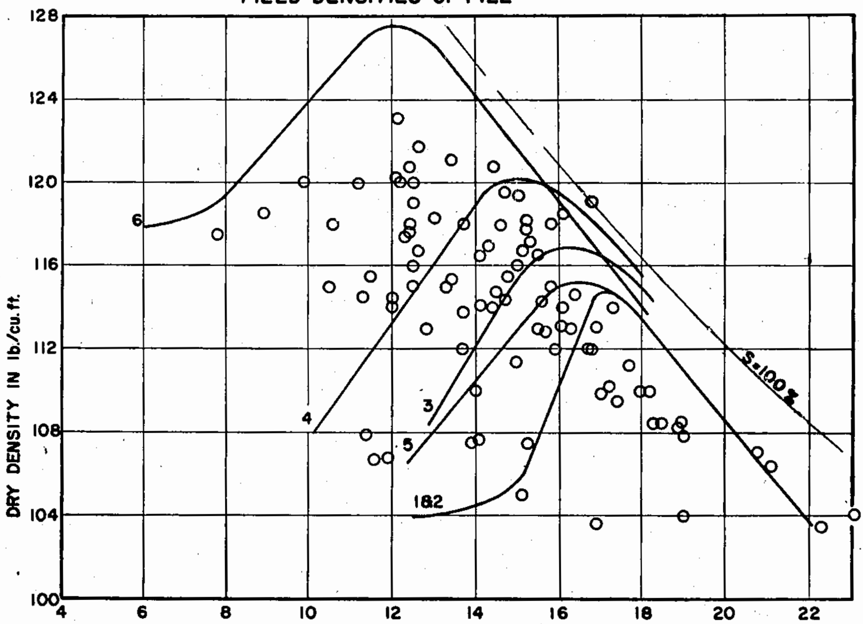

FIG. 25

specifications were rightly based on the modified AASHO test. The other tests indicated optimum moisture contents too great.

Figure 25 suggests that the limit of the field equipment used is a density of 123 lb/cu ft rather than the 127.5 obtained in the laboratory. Based on 123 as the maximum density, the 90% specification means a density of 111 lb/cu ft. In other words, the density and water content specified in Table VII when based on the limit of the equipment are: 111 lb/cu ft or greater; a moisture content from 10 to 12%.

Figure 25 shows:

1. Most of the field densities were greater than 111 lb/cu ft.
2. Most of the fill was placed too wet.

Much of the laboratory data presented and discussed in this paper were not available for field control. The main reason, however, that the fill was placed at such a high water content was that the natural moisture in the borrow soil was considerably above optimum. The rate of construction—7 days of 24 hours per week—was too fast to permit drying of the borrow before placement.

Except for the few points at moisture contents above 20%, all of those in Figure 25 represent fill as strong or stronger than the design value of 2000 lb/sq ft. A number of penetration tests by the writer in the finished dam showed no strengths below 4000 lb/sq ft. The dam, therefore, has more than adequate strength to fulfill its designed function—the storage of fuel oil.

VII—The Design of the Dam

A. *General Considerations*

This section presents the most important features of the design of the dam and reservoir to retain the fuel oil. To perform properly:

1. The dam and reservoir floor must prevent, or limit to a permissible quantity, the amount of fuel oil lost by seepage into the soil.
2. The dam must possess sufficient strength to resist structural failure. As will be seen, the Aduana Dam cannot be correctly analyzed by routine techniques often employed for fluid-retaining soil structures for two reasons. First, the stored fluid is fuel oil and not water for which many of the design conditions were developed. Second, the stratum of very weak clay makes stability analyses based on elastic methods inapplicable.

As shown in Section V of this paper, wet Aduana clay, compacted to the densities specified in Table VII can resist a head of oil in excess of 95 feet (larger than the height of the dam) before oil can force water from out of the clay. This means that there will be absolutely no leakage of oil into the dam or foundation if the blanket and dam face are properly constructed and maintained.

Before the stability of the dam can be studied, the transfer of pressure from the impounded oil in the Reservoir to the dam will have to be considered. This oil pressure becomes intergranular soil pressure at the water-oil interface just as capillary water tensions are transferred to the soil structure. In other words, the pressures from the impounded oil (see Figure 27) at equilibrium condition are similar to those which could be caused by equal pressures being transferred through a footing of no rigidity.

The pore water pressures were taken as zero at all points above the phreatic line (water table) which exists at the bottom of the dam. Actually, the pressures in the pore water above the phreatic line are probably less than atmospheric because of papillary forces developed at the air-water interfaces. If the extreme case existed of minisci being developed at the faces of the dam, the intergranular pressure at any point somewhat above the phreatic line would be much greater than that equal to the weight of the overburden. The strength of the compacted fill would, therefore, be greater than indicated by the lines in Figure 24.

The most critical conditions for the normal earth dam retaining water is "sudden drawdown", that is, removal of all the impounded water before the soil has had an opportunity to gain the strength increase attendant with such a pressure change. Since no seepage will occur through the Aduana Dam and the pore pressures are essentially zero (or below atmospheric pressure), the sudden drawdown condition is impossible. As will be seen, the most critical condition for the Aduana Dam is immediately after construction before any oil has been placed in it. At this time the strength of the foundation and the dam is at a minimum—that is, before any moisture decrease has had time to occur. The effect of the oil, as Figure 27 shows, is actually to help prevent a rupture of the inside face of the dam and to increase soil strength by increasing intergranular soil pressure.

Figure 26 shows a cross-section of the dam as it was built; Table VII summarizes the specifications for the selection and compaction of the dam and blanket. The final design was selected after a number of sections had been studied. The 13-meter height was

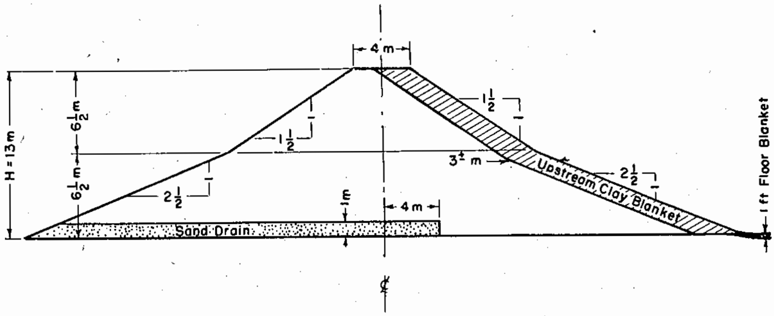

FIG. 26.—CROSS SECTION OF DAM.

the maximum desired by Creole; the 4-meter crust was set as a minimum for proper construction and for vehicular traffic during operation of the reservoir. The final dam section is analyzed in the following paragraph; analyses of the various preliminary sections are not discussed.

Figure 26 shows the presence of a one-meter thick sand drain. The sand drain was placed for two reasons: First, the construction of the dam was started before the laboratory tests and final designs were completed and thus the drain was put in as a precaution; secondly, Creole initially thought that the Aduana Reservoir might someday be used to store water and therefore the sand drain would be desirable. Final laboratory tests and design computations showed that the sand drain is not essential; observations during construction did not detect any moisture coming into the sand drain from the consolidating clay under the dam. A sand drain is not included in the design of the Incinerator Dam to be built during the summer of 1956.

B. *Resistance to Lateral Movement of Dam by Oil*

The strength of the dam and foundation must be enough to prevent the oil in the reservoir from sliding the dam. The lateral force of the oil when the depth is the maximum of 13 meters is,

$$P_L = \frac{1}{2}\ (\text{depth})^2\ (\text{unit weight of oil})$$

$$P_L = \frac{1}{2}\ (42.7\ \text{ft})^2\ (62.4\ \text{lb/cu ft} \times .96) = 54{,}600\ \text{lb/ft of dam length}$$

The strength of dam foundation required to resist this lateral thrust is,

$$s = \frac{54{,}600 \text{ lb/ft}}{\text{dam base} = 184 \text{ ft}} = 297 \text{ lb/sq ft}$$

From Figure 19, we can see that the minimum strength under the completed dam varies from 980 lb/sq ft at the center to 500 lb/sq ft at the edges. For an average minimum streng.h of $\frac{500 + 980}{2} = 740$, the factor of safety against sliding is,

$$FS_{sliding} = \frac{740}{297} = 2.5$$

Sliding of the Aduana Dam by the oil need not, therefore, be given further consideration.

D. *Resistance of Dam and Foundation to Rupture*

1. *Elastic Method of Analysis*

Figure 27 shows the dam section as constructed with the pres-

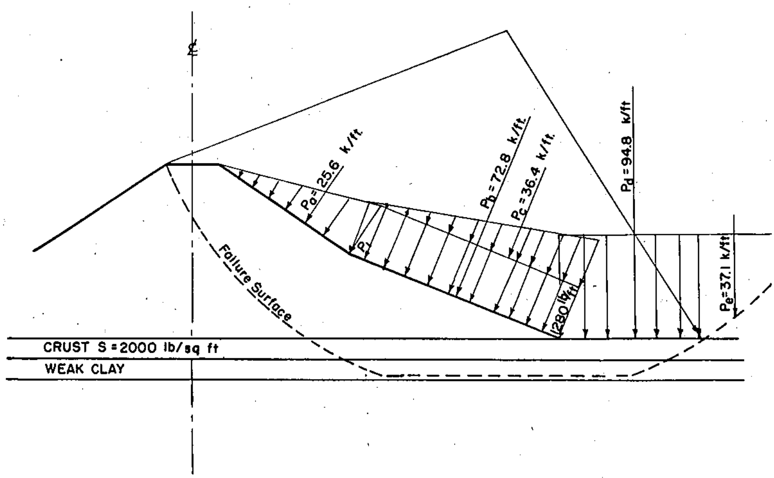

FIG. 27.—FORCES ON DAM FROM FULL RESERVOIR.

sures and forces from a full reservoir of oil. Assuming the theory of elasticity is valid in soils, we can compute the shear stress applied to the weak clay by the dam. Employing Jürgenson's* pressure analy-

*Jürgenson, Leo, "The Application of Theories of Elasticity and Plasticity to Foundation Problems", Journal of Boston Society of Civil Engineers, July 1934.

ses, we find the maximum shear stress in the weak clay caused by the weight of the dam is .22 times the vertical pressure. Thus, the maximum shear, s_{max} is,

$$s_{max} = .22 \times 42.7 \text{ ft (height of dam)} \times 135 \text{ lb/cu ft (unit weight of dam)}$$
$$= .22 \times 42.7 \times 135 = 1270 \text{ lb/sq ft}$$

The value of 1270 lb/sq ft is for the case when the reservoir has no oil. As Figure 27 shows, the impounded oil applies a vertical component of force to the dam. The addition of this vertical force from the oil increases the shear stress in the weak clay by several hundred pounds per square foot.

The shear stress in the weak clay, of the order of 1500 lb/sq ft, is far greater than the available strength, of the order of 500 to 1000 lb/sq ft. This method of analysis indicates that the dam shown in Figure 27 overstresses the foundation and is not safe.

The preceding method of analysis, although often used for embankment design, is inapplicable to the Aduana Dam.* The computation of shear stresses in soil by elastic methods is most questionable, since soil does not behave elastically. The foundation of the Aduana Dam is also far from homogeneous for an infinite extent, as assumed in preceding calculations. This computation is not, however, the major source of error in the analysis. This error is to assume that the overstress of any part of the structure or foundation results in failure of the entire mass. The Aduana Dam will not, in fact, collapse if the weak foundation clay does not start to rupture because strains will mobilize more resistance in the dam proper. The method of analysis presented in the following section of this report considers the stability of the entire dam and foundation—the correct procedure.

2. *Slip Failure*

For the dam to rupture, it must slip along some surface such as shown in Figure 27. If the dam and foundation had the same

*Jürgenson in his classic paper, clearly pointed out that rupture would occur only when the shear stress at *all* points on the potential failure surface had exceeded the shear strength. To reach this fully plastic state required an applied surface pressure somewhat greater than that indicated by the computations based on elastic theory. Jürgenson also indicated that his calculations were based on an infinite mass of homogeneous, elastic and isotropic material. These limitations of elastic-theory stability methods have usually been overlooked in the design of embankments.

strength characteristics, the rupture would occur along a surface which is approximately cylindrical. The critical surface of potential failure of the Aduana Dam lies more in the weak clay than any circle does. The failure surface shown in Figure 27 is approximately the critical one found by trial.

For the dam to rupture, rotation must occur about some point. The moment of those forces, dam weight and oil weight, about the center must be greater than the amount of the resisting soil strength. The critical surface is that where the ratio of resisting moment to actuating moment, i.e., factor of safety, is the lowest.

In Figure 27 the magnitude, direction, and location of the forces on the dam from the oil are shown. Since all but one of the five oil forces give moments which resist failure, we can see that the critical condition is for an empty reservoir. Since as we have already pointed out, the strength of the weak clay increases with consolidation, the most critical condition is immediately after dam construction, since only part of the consolidation has occurred and the reservoir has not been filled. The following analyses neglect, therefore, the forces applied by the oil and employ the clay strength existing prior to consolidation, i.e., the left line in Figure 19.

As a first approximation, we can employ Taylor's chart* of the slip-surface method of analysis to find we need a product of failure surface times shear strength equal to 137 kip feet. For the failure surface shown in Figure 27, we have,

75 ft in the dam and crust where the strength is 2000^+ lb/sq ft and,

80 ft in the weak clay where the strength is 500^+ lb/sq ft, i.e.,

$$\begin{aligned} 75 \text{ ft} \times 2 \text{ k/ft}^2 &= 150 \text{ kip/ft} \\ 80 \text{ ft} \times .5 \text{ k/ft} &= \underline{40 \text{ kip/ft}} \\ & 190 \text{ kip/ft} \end{aligned}$$

and the factor of safety is,

$$F = \frac{190}{137} = 1.4$$

Using Taylor's chart, we cannot properly take care of the slope change on the face of the dam, the irregular failure surface, and the

*Figure 16.27 of "Fundamentals of Soil Mechanics," by D. W. Taylor, John Wiley and Sons, 1948.

fixed crest dimension. These were considered in the slices method shown in Figure 28; the failure wedge was divided into thirteen slices and forces on each considered separately. As noted in Figure 28, allowance is made for a crack (caused by shrinkage from drying or by the tension existing in the top of the dam) extending half the depth of the dam. This analysis gives a factor of safety of 1.45.

STABILITY ANALYSIS OF DAM

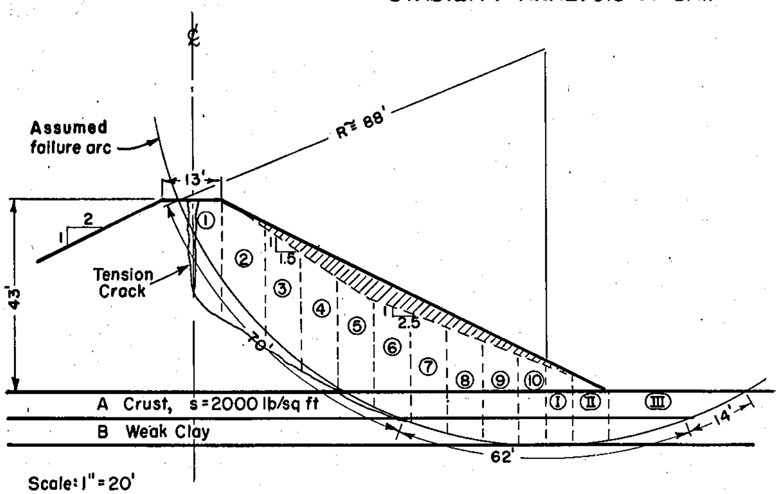

FIG. 28.

Figure 28 shows why a dam section of 1.5 horizontal to 1.0 vertical for the top half and 2.5 horizontal to 1.0 vertical for the bottom half has a higher safety factor than a slope of 2.0 horizontal to 1.0 vertical. Nearly all of the dam removed in changing from the second mentioned section to the first lies to the left of the center of critical surface. In other words, removal of this material reduces the overturning moment. A saving in earth fill also results from using the broken-slope section.

For the following reasons, the actual factor of safety is probably greater than the indicated value of 1.45:

1. The weak clay most likely does not extend for the entire width of the dam.
2. The resisting forces on the ends of the potential failure wedge were neglected, i.e., a two-dimensional analysis was used for a three-dimensional problem.
3. A tension crack for half the depth of the dam is most unlikely.

4. The shear strength of the weak clay employed in the analysis, i.e., 500 lb/sq ft, is conservative. Actually, significant consolidation occurred during construction, so a minimum strength near 800 or 900 lb/sq ft existed immediately after dam construction.

Soil engineers normally like to have a factor of safety in excess of 1.5 but often design on 1.25. The Aduana Dam is, therefore, considered safe since it meets the requirements of good soil engineering design.

VIII. Settlement of Dam

A. *General*

Information on the amount and rate of consolidation of the weak foundation clay by the weight of the Aduana Dam was needed for two reasons. First, as shown by Figure 19, the strength of the weak clay is dependent on the consolidation of the clay. Second, an estimate of the amount and rate of settlement of dam was needed to select properly the dimensions of the dam and to design any piping and any other appurtenances which go through or on the dam.

Undisturbed samples of the weak clay were subjected to one-dimensional confined compression tests in the laboratory. The results of these tests were then employed to make an estimated settlement-time curve. The calculations indicated an ultimate settlement in excess of two feet; to allow for this settlement, the dam was built one meter higher than the desired thirteen-meters.

B. *Laboratory Tests*

Two specimens from Soil D were subjected to standard consolidation tests; the test results are presented in Figure 29. The top curve, a plot of void ratio (void volume divided by solid volume) against log pressure, indicates the amount of consolidation. The lower curve shows the rate of consolidation, expressed as the coefficient of consolidation, as a function of pressure on a log scale.

The test results in Figure 29 not only give the soil properties needed to make a settlement analysis, but also indicate the maximum pressure to which the tested samples had even been consolidated in their geological history. This maximum past consolidation pressure is in the range of the bend on the void ratio-log pressure curve and can be approximated by the empirical graphical construction shown in Figure 29. This technique gives approximately 1400 lb/sq ft; as

RESULTS OF CONSOLIDATION TESTS ON FOUNDATION CLAY

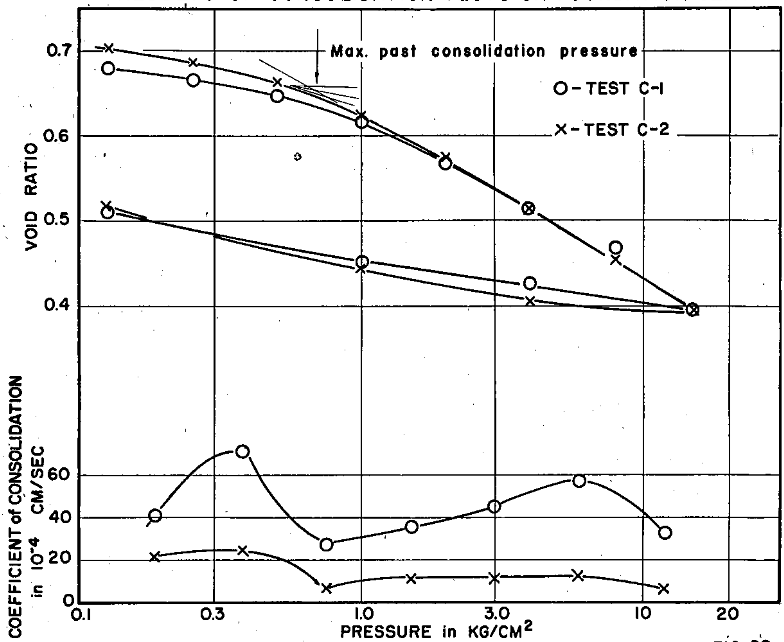

FIG. 29

shown by Figure 18, this value agrees with that suggested by the strength-pressure plot.

C. *Boundary Conditions for Settlement Analysis*

As is true in most soil problems, the most uncertain feature of the settlement analysis for the Aduana Dam was the subsurface conditions. The 19 drive-sample borings not only revealed considerable variation in the sub soil conditions but also failed to give any dependable quantitative information on the soil below a depth of 3 meters. The shallow weak clay was studied in holes made by Creole's auger and on samples taken by hand from pits. The auger borings showed the weak clay was underlaid by much stronger soils.

The sub soil conditions used in the settlement analyses are shown in Figure 30; they were approximated from all of the available information. The properties of Clay 1 were accurately known from the many laboratory tests on Soil D. A study of the drive-sample boring logs, and laboratory data on Soils A, B, C, and D, indicated the probable properties of Clay 2 as: Water content $\approx$ 20%;

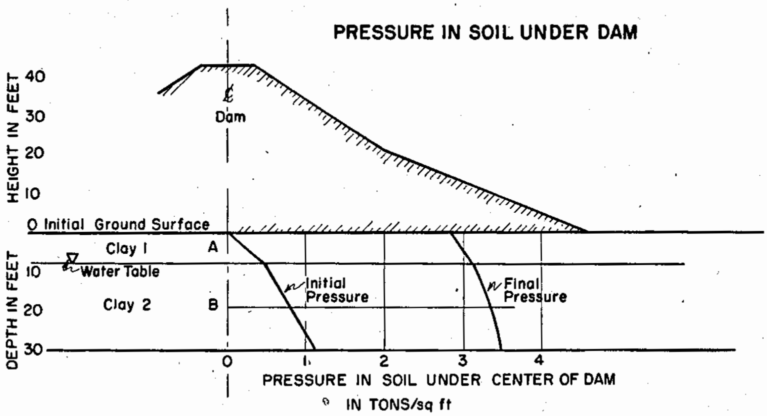

FIG. 30.

liquid limit ≈ 37%, and void ratio ≈ 0.54, Clay 2 was assumed similar to Clay 1 except that it had been precompressed to a higher pressure.

The simplified geological section in Figure 30 is thought to have approximately the same settlement characteristics as the actual section. The weak Clay 1 was intentionally thicker than actually exists to offset the neglect of all compressible soil below a depth of thirty feet.

D. *Settlement Analysis*

In the settlement analysis, the ultimate settlement was taken as the sum of the contributions of Clays 1 and 2. The rate of settlement was computed by considering the two clay strata as one thirty-foot stratum with drainage at both the top and bottom. The weak clay had top drainage through the sand filter.

Figure 30 shows the results of the pressure analysis. The initial pressure in the soil is equal to the weight of overlying soil. The pressure increment from the dam was computed from elastic theory.* Both the initial pressure and final pressure, equal to the initial plus the increment from the dam, are for equilibrium conditions, i.e., when all excess pore water pressure has been dissipated.

For the boundary and pressure conditions shown in Figure 30, the estimated settlement curve presented in Figure 31 was obtained.

*While elastic theories are unreliable for the estimation of shear stresses in soil, they give a good approximation of the induced vertical stresses.

Plotted on Figure 31 are several settlement-time values measured on the actual dam. The measured values give needed assurance that the boundary conditions employed in the analysis are approximately correct.

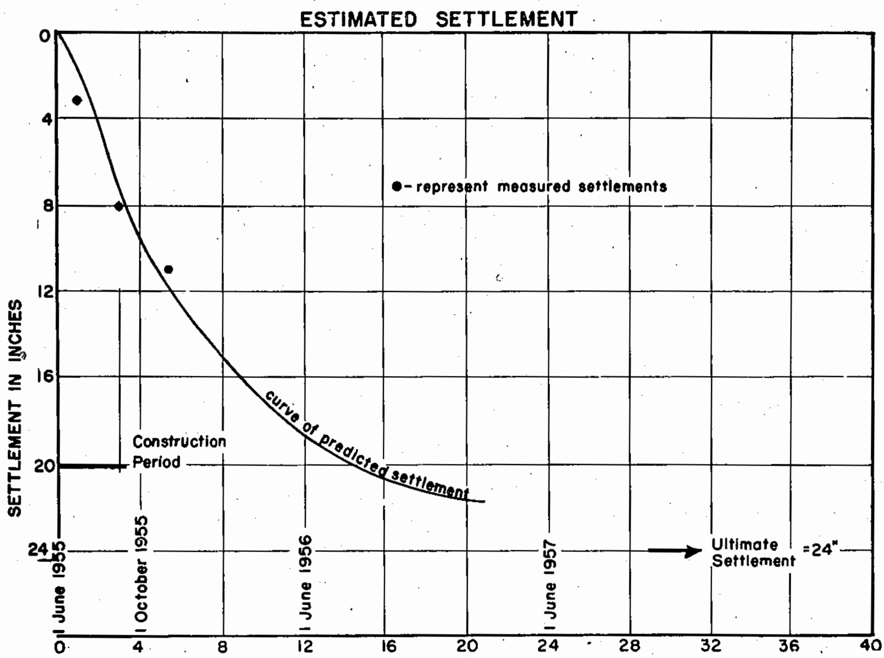

FIG. 31

IX. Performance of Dam and Reservoir

Creole engineers are making periodic measurements of:

1. Quantity of oil admitted and withdrawn.
2. Quality of oil in the Reservoir.
3. Rainfall.
4. Evaporation.
5. Settlement at a number of points.
6. Level of fluid in the Reservoir.

In Figure 31 are plotted the measured settlements available when this paper was written along with the curve of theoretically predicted settlement. The actual settlement is considerably closer to the predicted than would have been expected from the approximate nature of the analysis.

Figure 32 presents the available data on oil level in the reservoir and cumulative quantity of oil admitted to or withdrawn from the res-

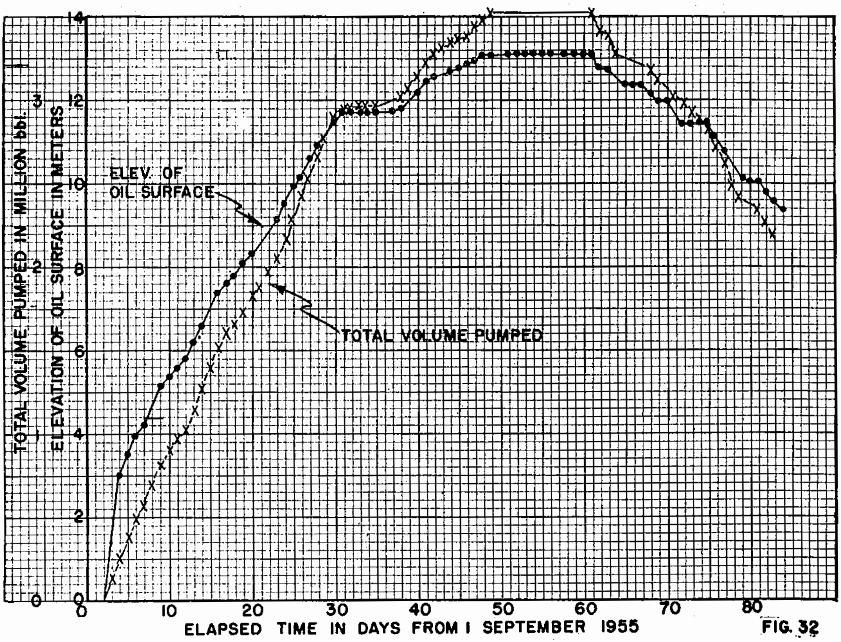

FIG. 32

ervoir. These data show the same volume (2,200,000 lbs) when the depth was 9.6 m during filling in September and when the depth was 9.6 m during emptying in November. In other words, no measurable leakage occurred during this period (actually, rainfall = leakage + evaporation). Creole reported 2425 barrels of oil of the 3,503,999 stored as "not recovered". Most of this 2425 was mixed with the top 2 or 3 inches of soil as described below.

After the Reservoir was emptied in January of 1956, thorough inspections of the dam and floor blanket were made by the writer and by Mr. George Breffeilh, Chief Engineer of Creole's Amuay Refinery, and Mr. Joseph Fuller, a Creole engineer. The average depth of oil penetration on the floor was a couple of inches, as shown in Figure 33, a photograph of a small test hole dug by Mr. Breffeilh. At several spots the oil had penetrated to 5 or 6 inches; and at one place a maximum depth of penetration of 18 inches was measured. The oil penetration into the upstream face of the dam and into the

FIG. 33.—TEST HOLE SHOWING EXTENT OF OIL PENETRATION INTO RESERVOIR FLOOR.

natural slopes was somewhat less than the bottom, averaging a couple of inches. Isolated soft spots with 8 to 11 inches of penetration were observed.

The soil underlying the oil-penetrated soil showed no evidence or moisture loss and was as dense or denser than when placed. No evidence of oil penetration could be detected in this underlying soil. The true leakage from the reservoir is, therefore, concluded to be essentially zero.

The main cause of oil penetration was the presence of cracks in the soil, varying from hair-line size to several millimeters in thickness. Even where no drying cracks existed, the oil penetrated dry soil, as the laboratory tests had indicated would happen. The soil drying was due to evaporation of soil moisture; the high and steady winds plus the high temperatures (caused by the sun and warm oil, admitted to the reservoir at 122 ± °F) accelerated evaporation.

The unrecoverable oil was extremely low—2425 barrels out of 3,503,999 or less than 0.07% of that stored. Evaporation loss, computed from changes in properties of the stored oil, was about 1% of the stored volume. The unrecoverable oil consisted of that which penetrated the soil plus a 1 to 4 inch thick layer of hydrocarbon sludge lying on part of the bottom. Periodic laboratory analyses of the oil withdrawn showed it remarkably free of ash and sediment and, in fact, essentially the same as oil withdrawn from steel tankage.

Several of the periodic reports by Creole indicated that water was withdrawn from the bottom of the reservoir along with oil. This water which fell as rain on the surface of the oil, because of its higher density, settled to the bottom and formed a layer between the oil and the soil blanket. This is further evidence of no oil leakage through the bottom of the reservoir, but more important, that rain does not form an emulsion with the oil but instead settles out as a separate phase as had been predicted by ESSO Research.

The very critical time schedule of operations by Creole precluded any slope protection to be effected prior to the first year's use of the dam and reservoir. Heavy rains ocurring during the fall of 1955 caused the erosion of the exposed faces of the dam and natural hill. The erosion was somewhat worse on the 1½ to 1 slopes than on the 2½ to 1 slopes as would be expected. Laboratory experiments have indicated that a layer of soil cement on the face of the dam will provide adequate slope protection. This method of protection is being given consideration for the new Incinerator Dam to be built during the summer of 1956. The actual slope protection used on the Incinerator and the Aduana Dams will depend on the field experimentation to be conducted during the summer of 1956.

It is still too early to make an exact cost comparison between steel and earth reservoir storage. The steel tankage is much more expensive, $2.00 versus $0.27 per barrel of storage, and takes considerably more time to design, fabricate and construct. Evaporation losses from steel tankage is smaller than from open reservoir storage; however, the small reservoir loss, makes this factor insignificant relative to the tremendous differences in initial cost. The better protection of stored oil to dust, rain, birds, etc., furnished by covered steel tanks is apparently not important. Differences in maintenance costs are yet to be evaluated; also, the operational features of large, concentrated storage volumes versus small, dispersed volumes are not yet evaluated.

The over-all excellent performance of the Aduana Reservoir plus the extraordinary savings in construction cost and time have led to the decision by Creole to construct more open earth oil reservoirs. A reservoir of more than double the capacity of the Aduana one is planned for the summer of 1956. The use of earth reservoirs to store fluids can be expected in a number of places in the world where the soil and operational conditions are favorable.

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