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THE STORAGE OF OIL IN AN EARTH RESERVOIR

BY T. WILLIAM LAMB,* *Member*

(Presented at a meeting of the Structural Section, B.S.C.E., held on April 11, 1956).

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 $= 1.055 \times 10^{-2}$ cm2

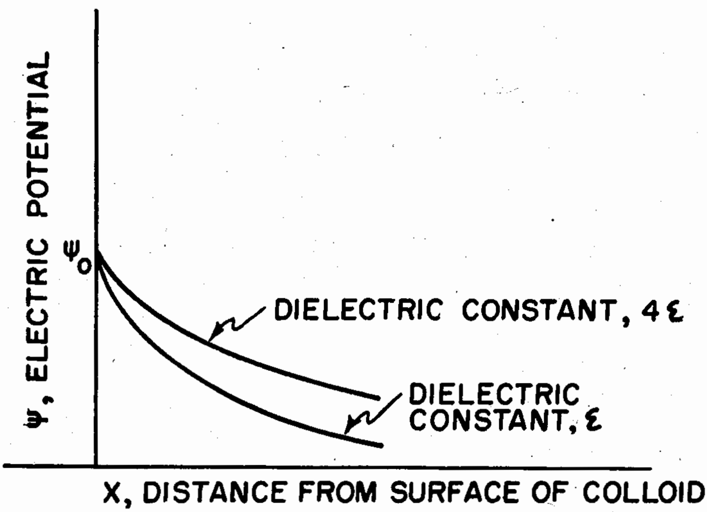
for water at 20°C, K in cm2 $=$ K in cm/sec $\times 1.028 \times 10^{-5}$

K in cm/sec $=$ K in Darcy's $\times 1.028 \times 10^{3}$

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

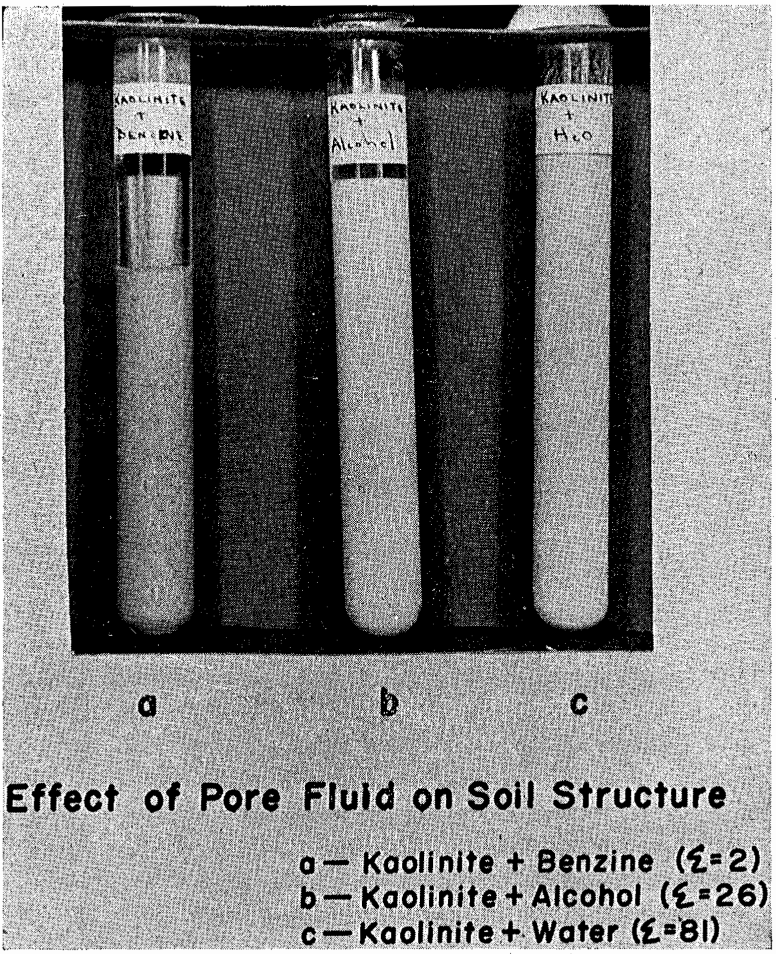


FIG. 2.

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,

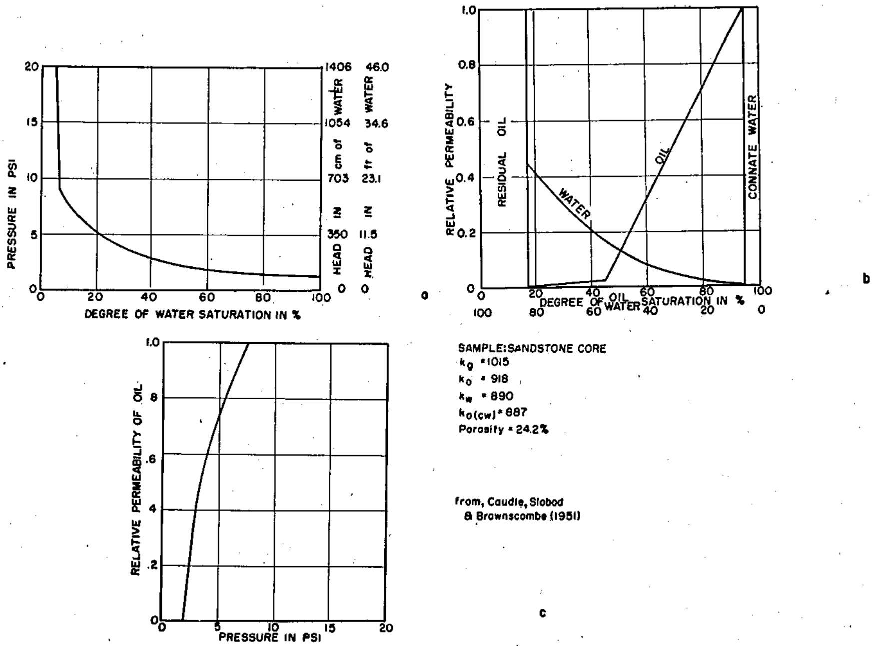


FIG. 3.

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

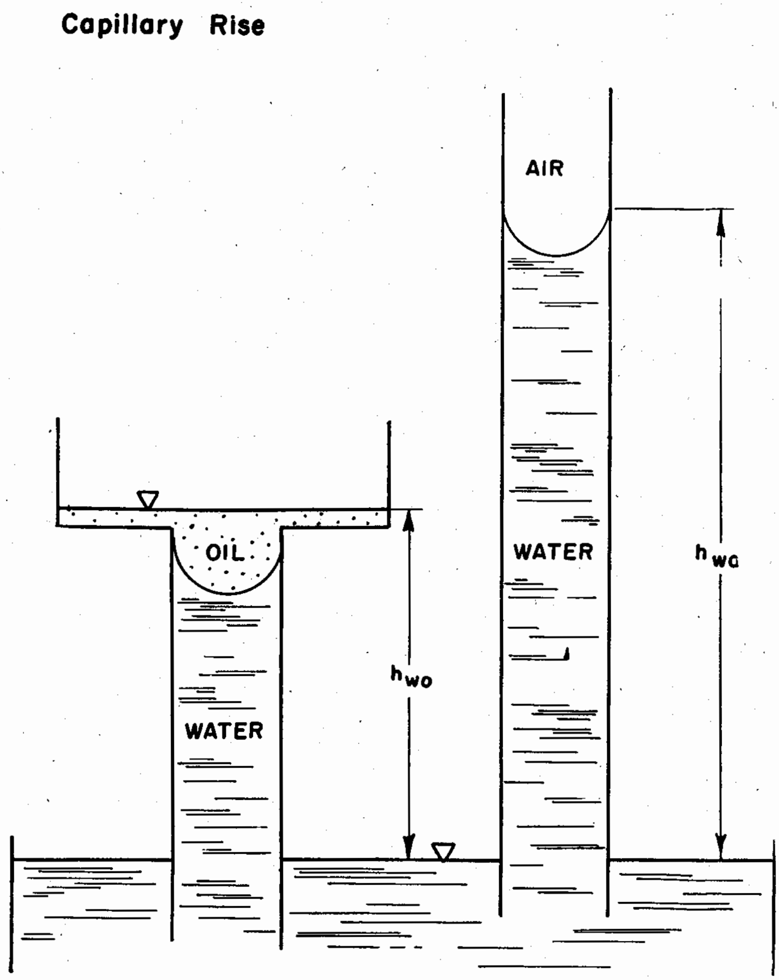


FIG. 4.

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-

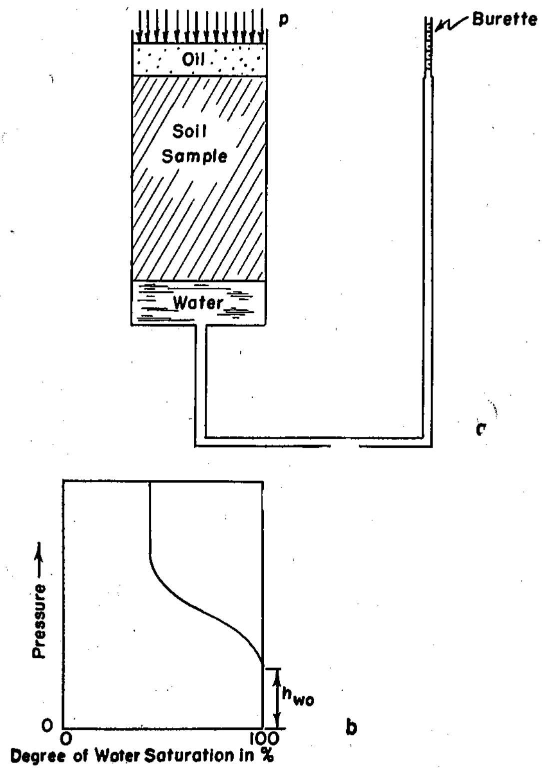


FIG. 5.

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-

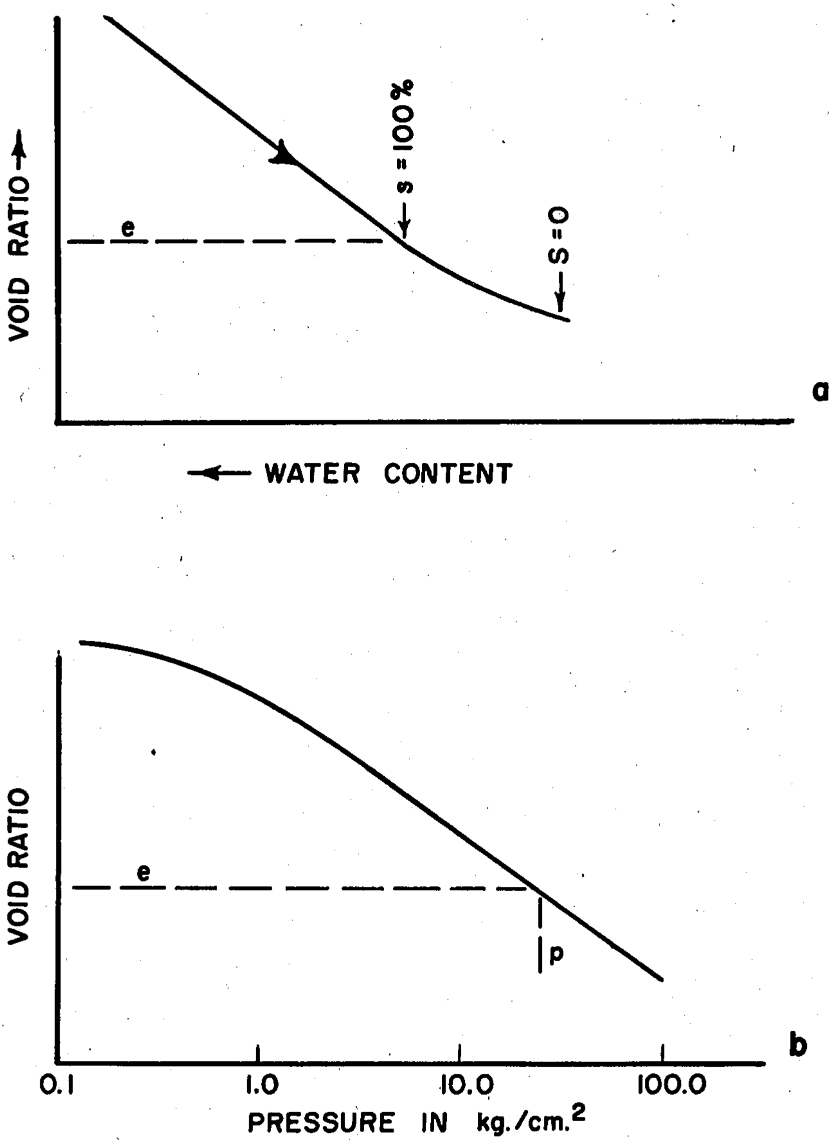

VOID RATIO
e
s = 100%
S = 0
a
WATER CONTENT
VOID RATIO
e
p
b
0.1
1.0
10.0
100.0
PRESSURE IN kg./cm.2

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

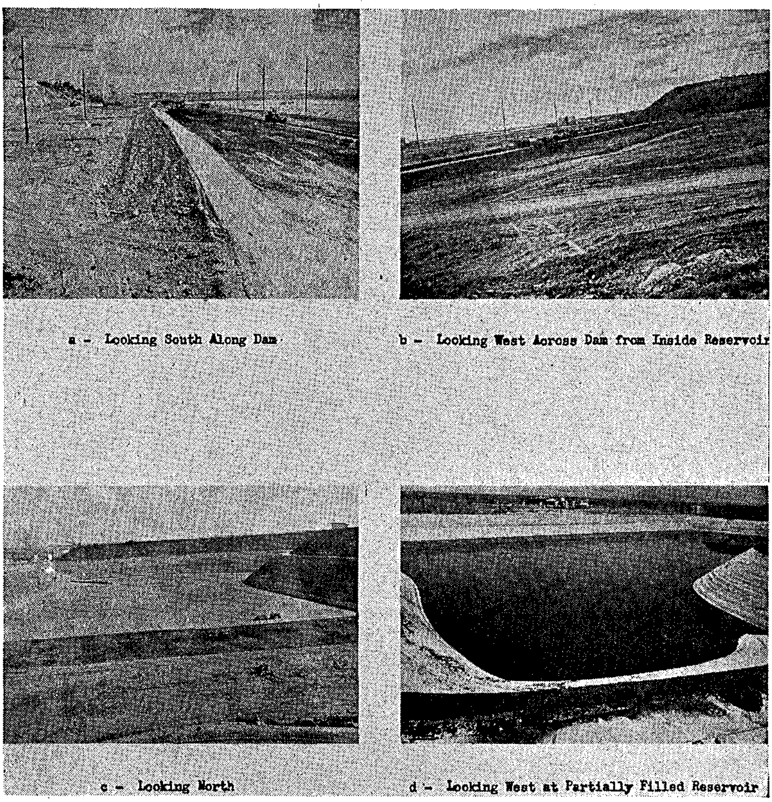


FIG. 7.—ADUANA DAM AND RESERVOIR.
(*Photographs by Creole*).

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

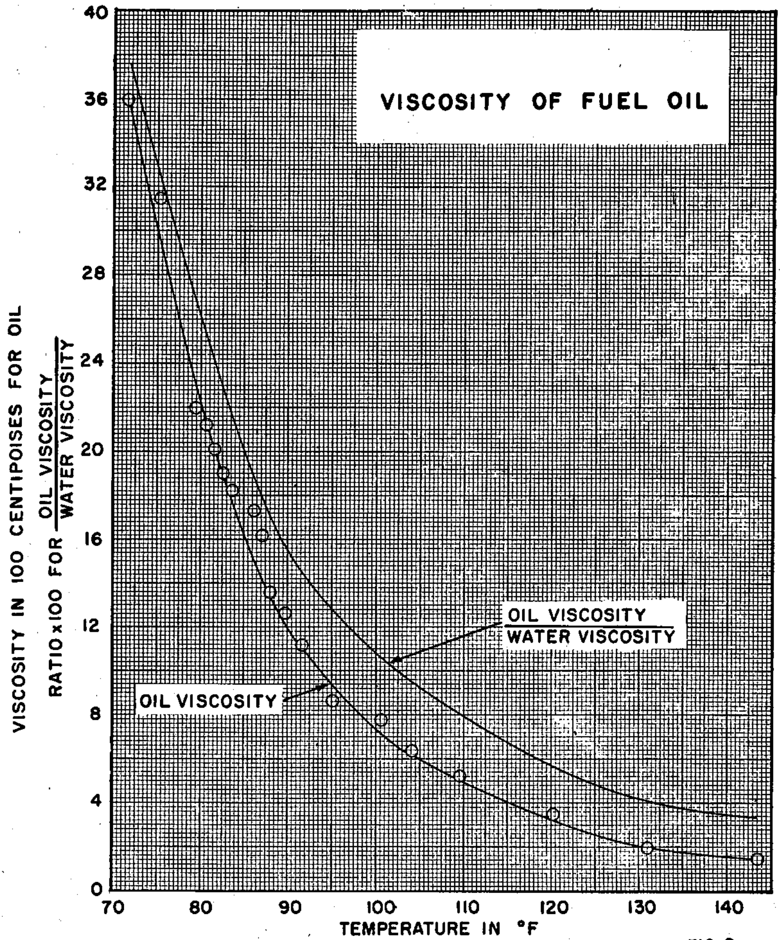


FIG. 8

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.

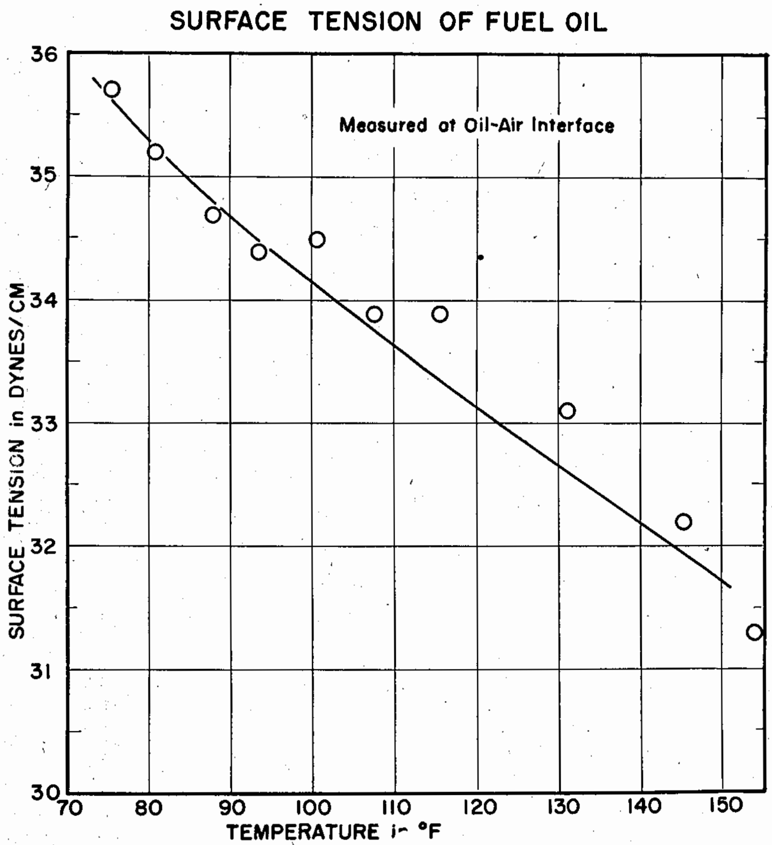


FIG. 9.

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

GRAIN SIZE DISTRIBUTION

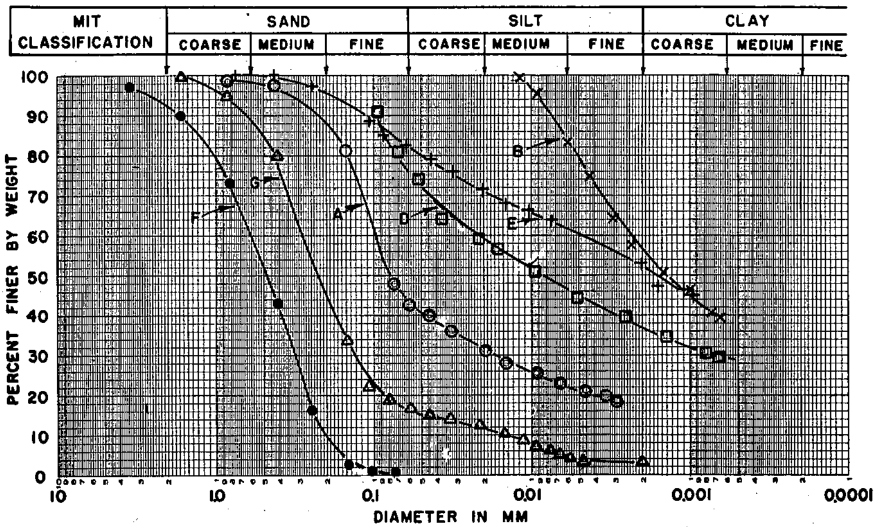


FIG. 10.

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 Designation	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.

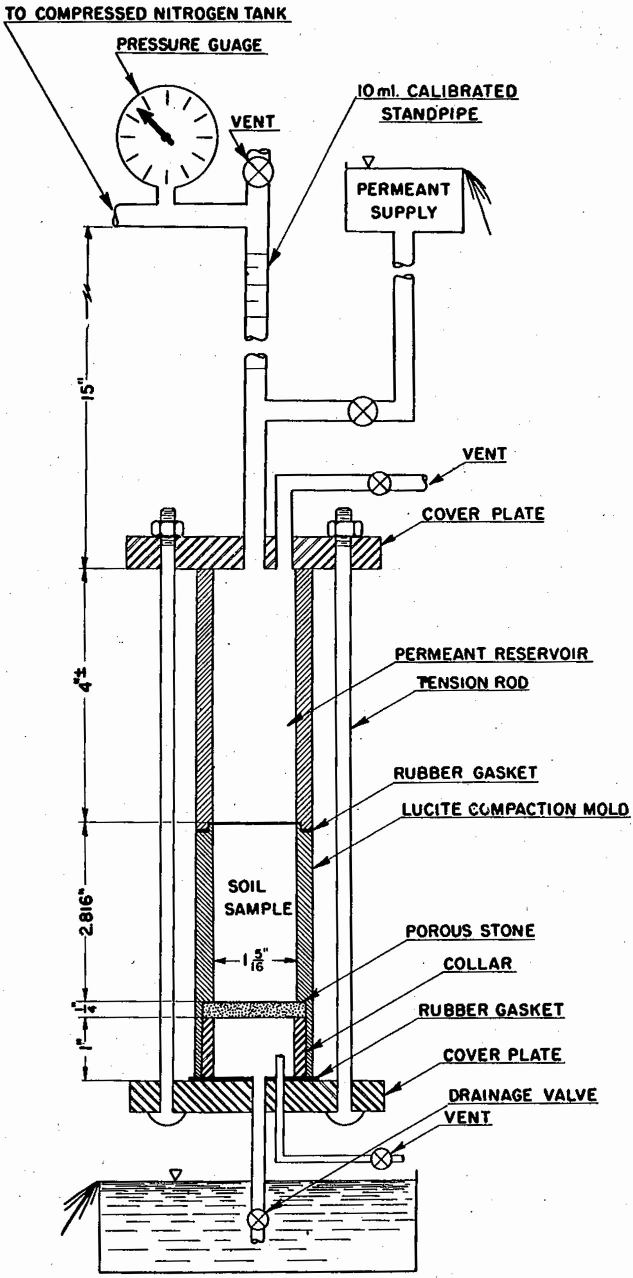
The theoretical considerations in Section II of this paper showed that the lower the water permeability of a compacted soil the lower the oil permeability of that soil and the higher the pressure required for the oil to force water out of the clay against the water-soil capillary forces. To facilitate 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 3×10^{-9}cm/sec was obtained; 1×10^{-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.

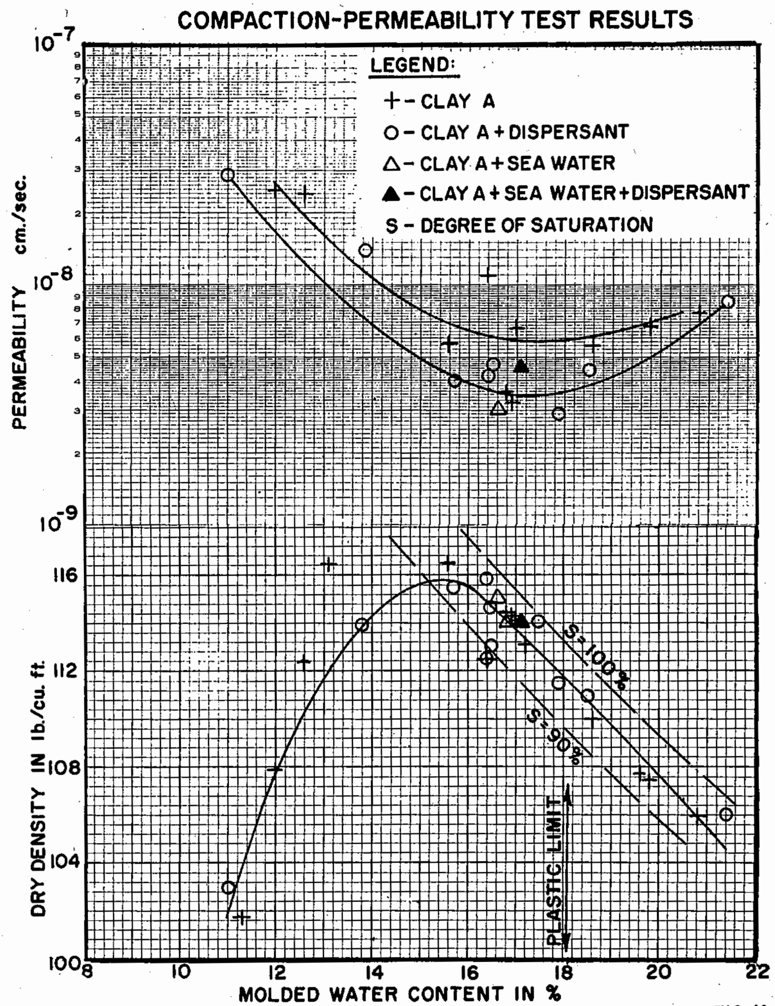


FIG. 12

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,

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

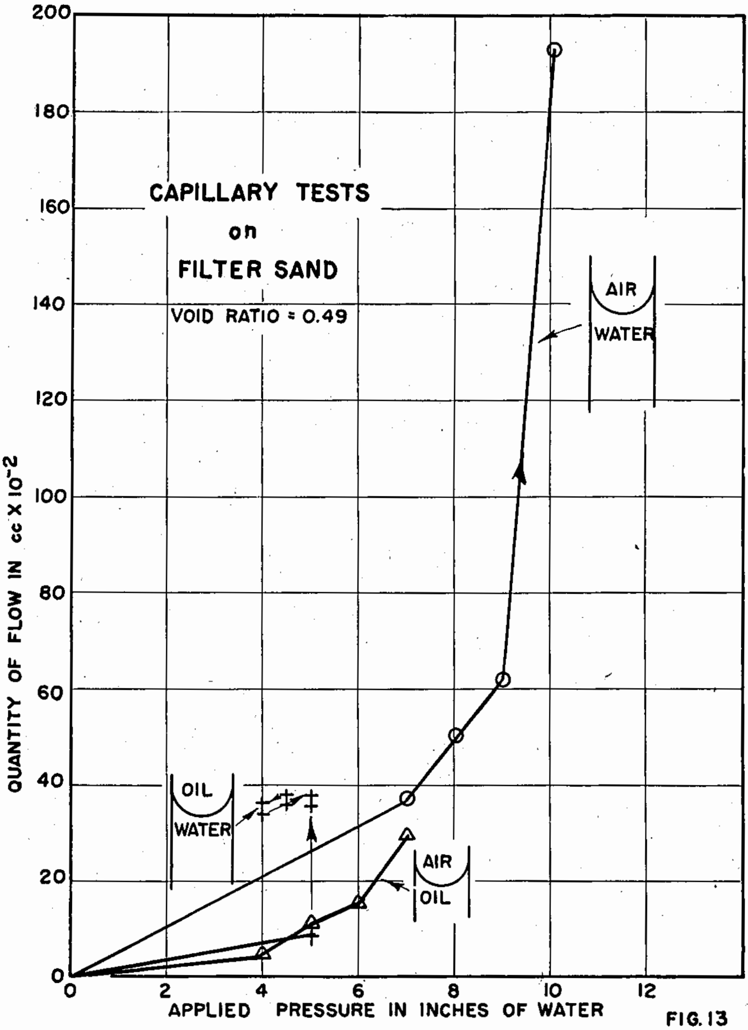


FIG. 13

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

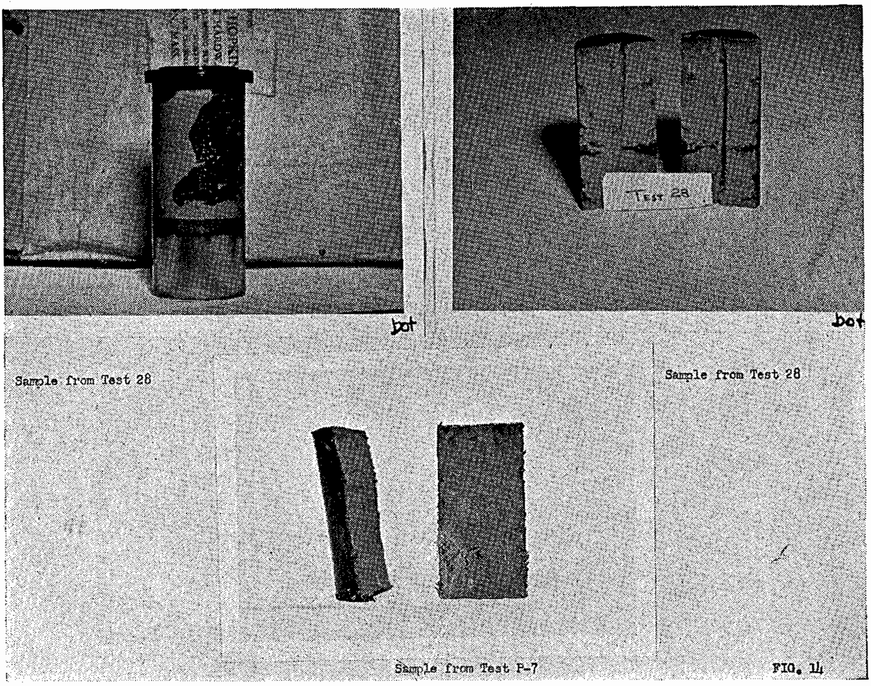


FIG. 14

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

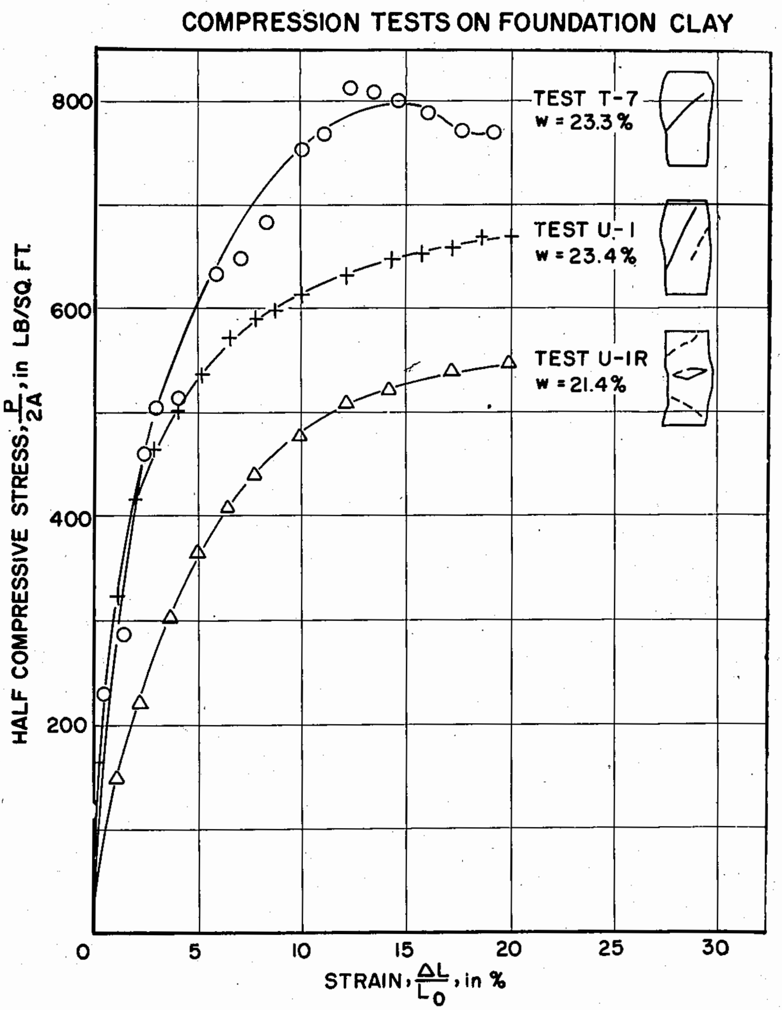


Fig. 15.

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 Com-pressive 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

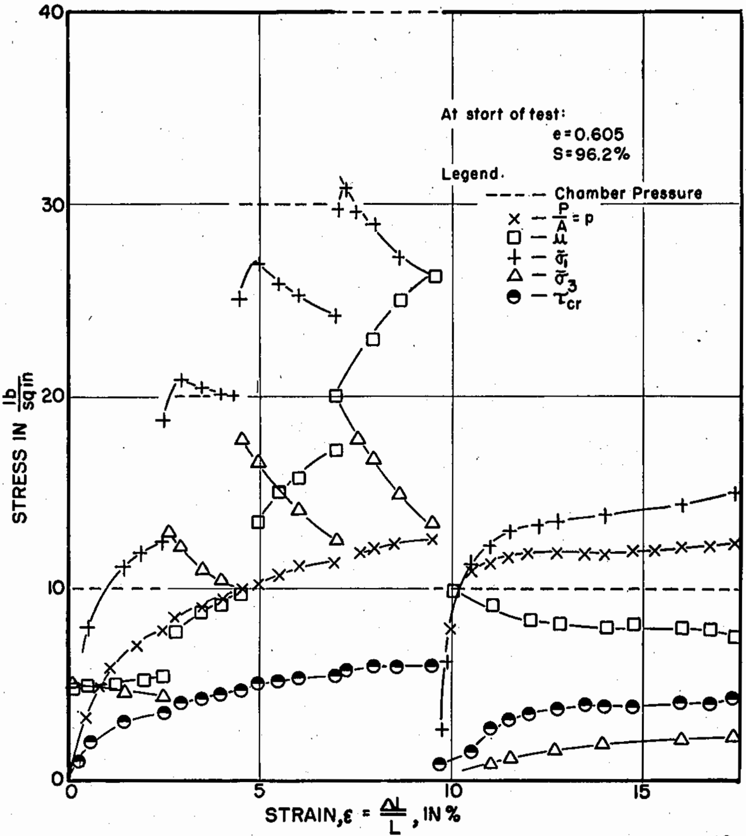


FIG. 16

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.

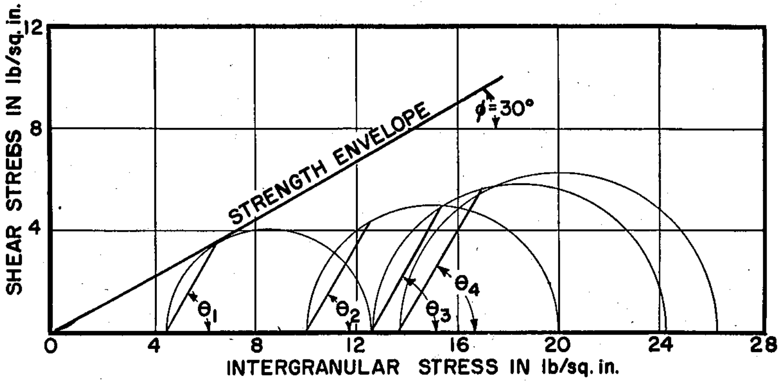


FIG. 17

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.

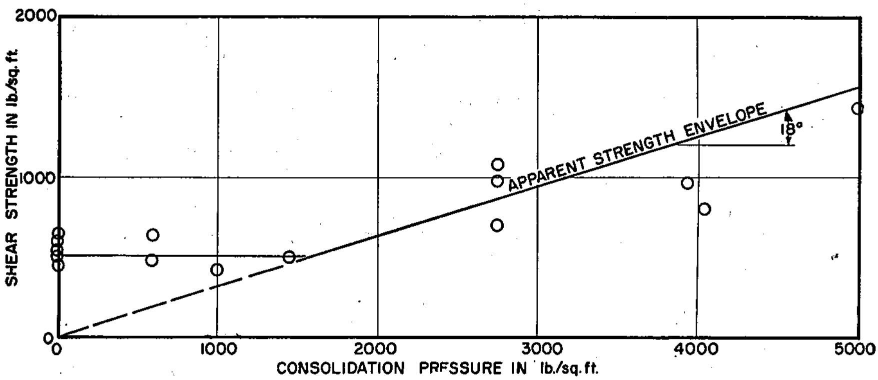


FIG. 18

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

STRENGTH OF FOUNDATION CLAY
AS A FUNCTION OF DAM HEIGHT

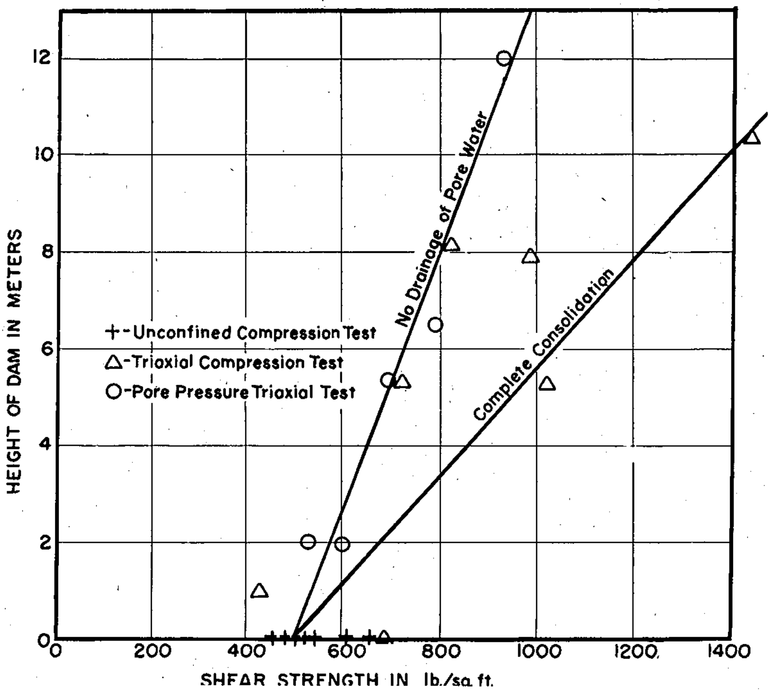


FIG. 19.

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.

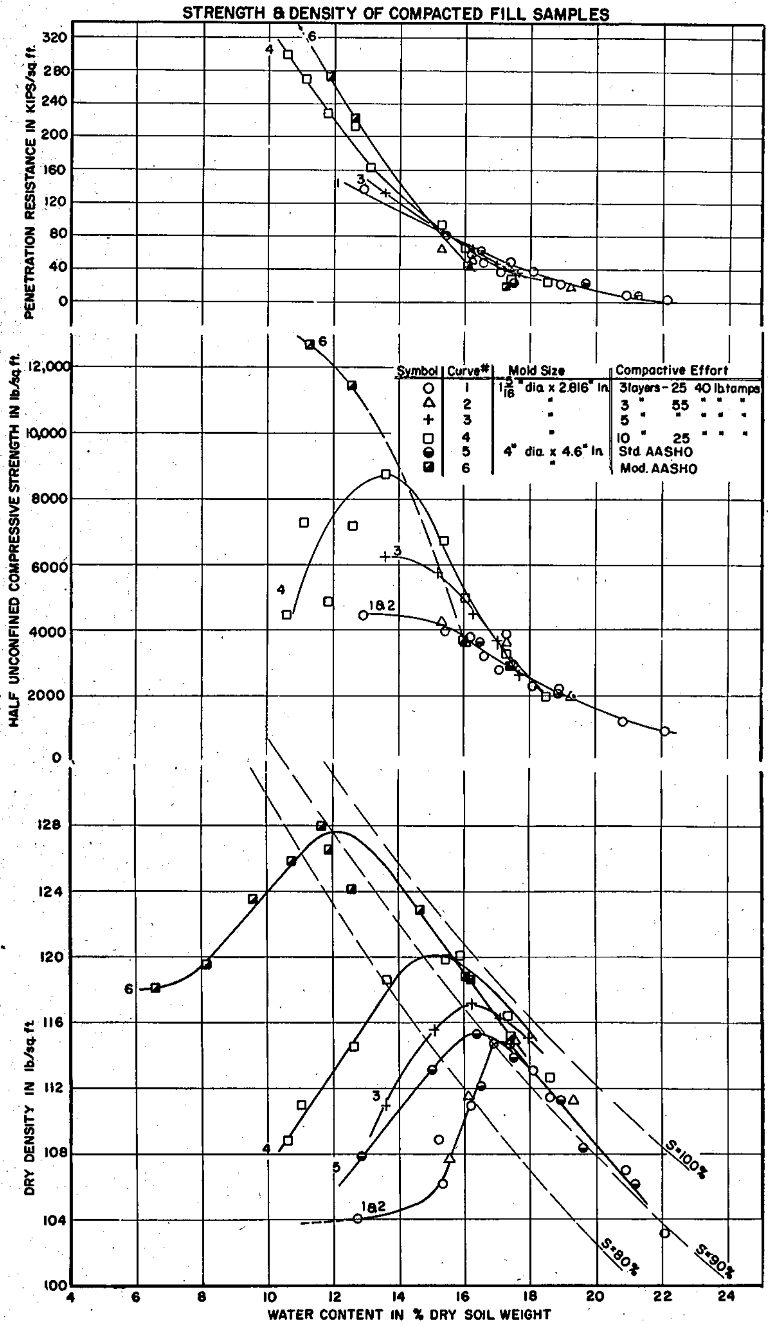
Figure 20 shows characteristically the influence of water content and compactive effort on density, but strength, the most important variable, can be better studied from other plots of the data; these plots are presented and studied in the following pages.

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
Std. AASHO
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.

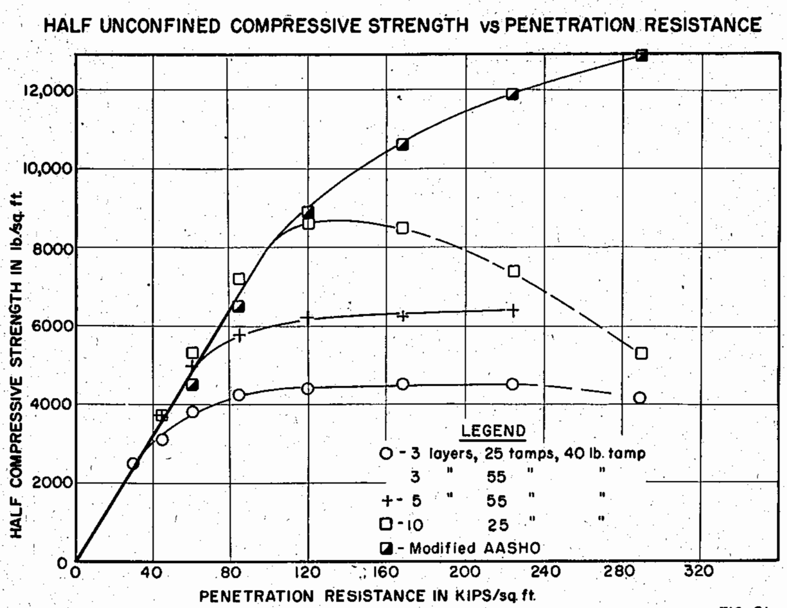

HALF UNCONFINED COMPRESSIVE STRENGTH vs PENETRATION RESISTANCE
HALF COMPRESSIVE STRENGTH IN lb/sq. ft.
12,000
10,000
8000
6000
4000
2000
0
LEGEND
O - 3 layers, 25 tamps, 40 lb. tamp
3 " 55 " "
+ - 5 " 55 " "
□ - 10 25 " "
◪ - Modified AASHO
0
40
80
120
160
200
240
280
320
PENETRATION RESISTANCE IN KIPS/sq. ft.

FIG. 21

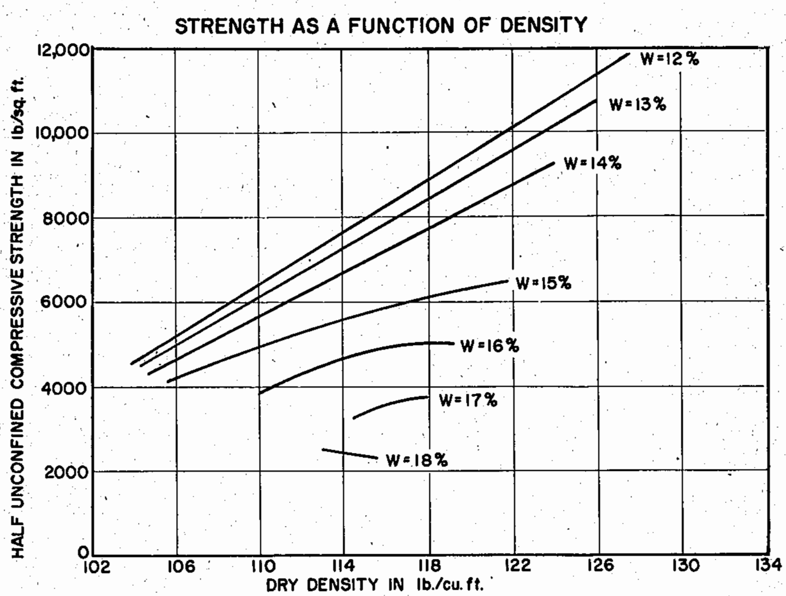

STRENGTH AS A FUNCTION OF DENSITY
HALF UNCONFINED COMPRESSIVE STRENGTH IN lb/sq. ft.
12,000
10,000
8000
6000
4000
2000
0
W=12%
W=13%
W=14%
W=15%
W=16%
W=17%
W=18%
102
106
110
114
118
122
126
130
134
DRY DENSITY IN lb./cu. ft.

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.

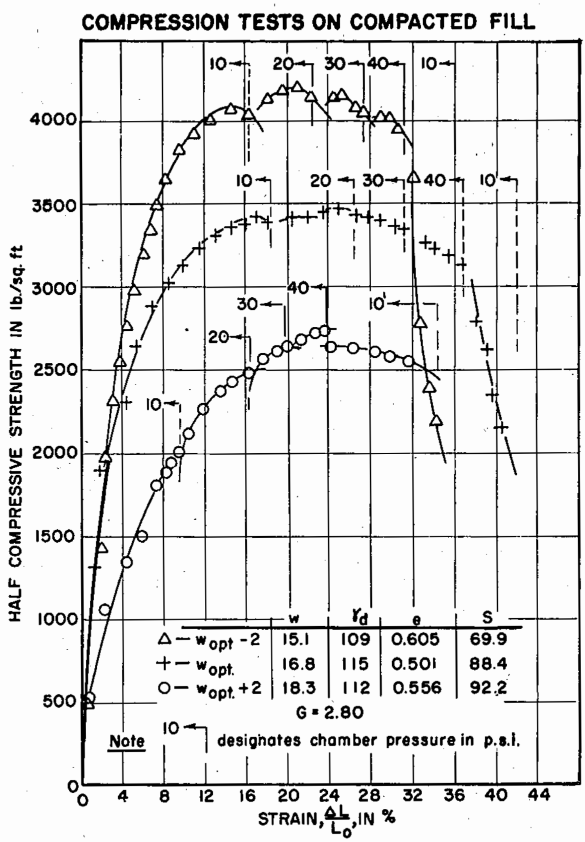


FIG. 23.

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.

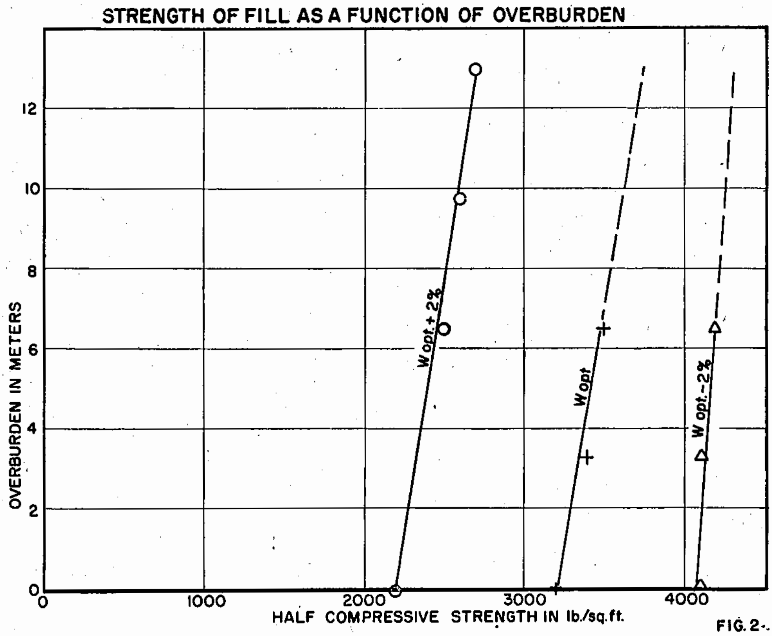


FIG. 2-.

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

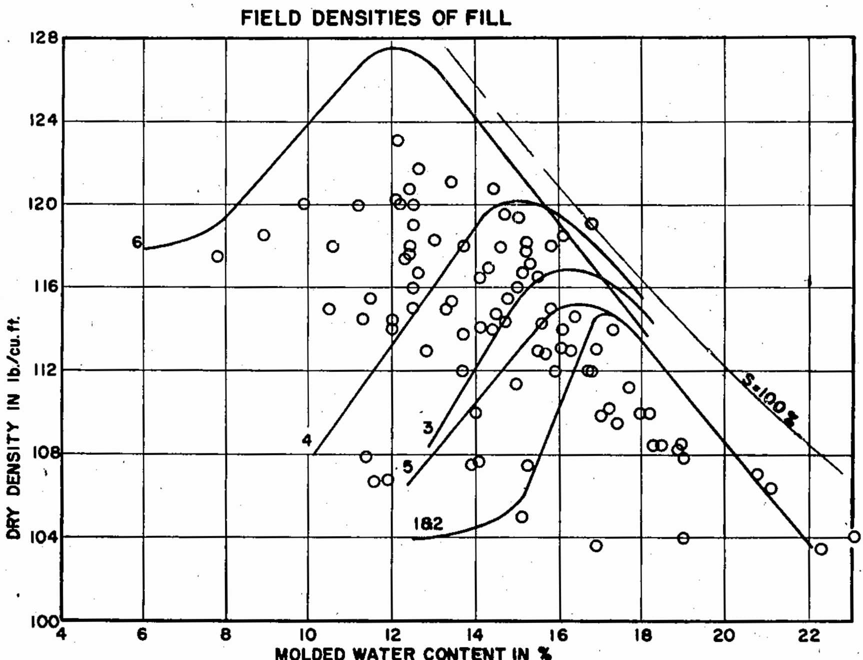


FIG. 25

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

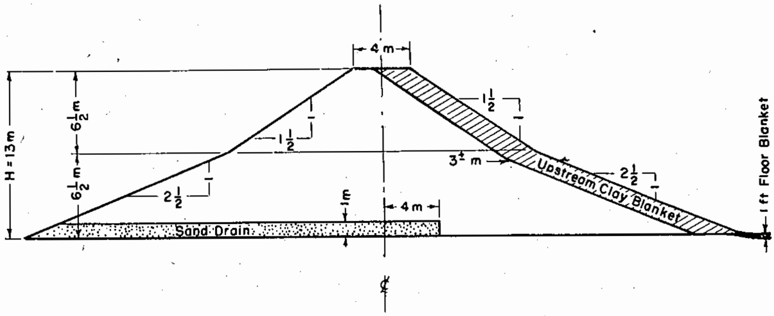


FIG. 26.—CROSS SECTION OF DAM.

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,

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

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-

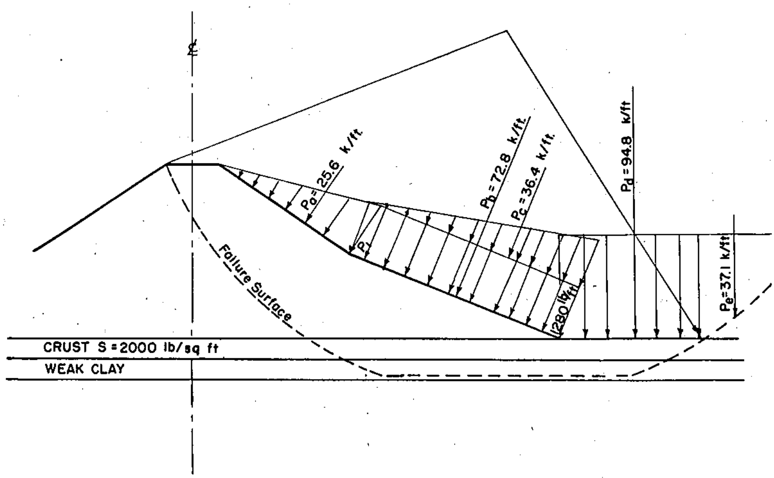


Fig. 27.—Forces on Dam from Full Reservoir.

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.,

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

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.

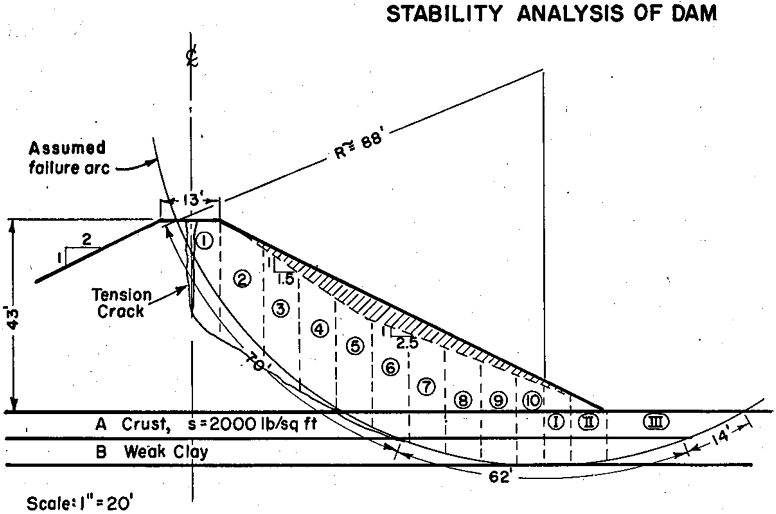


FIG. 28.

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

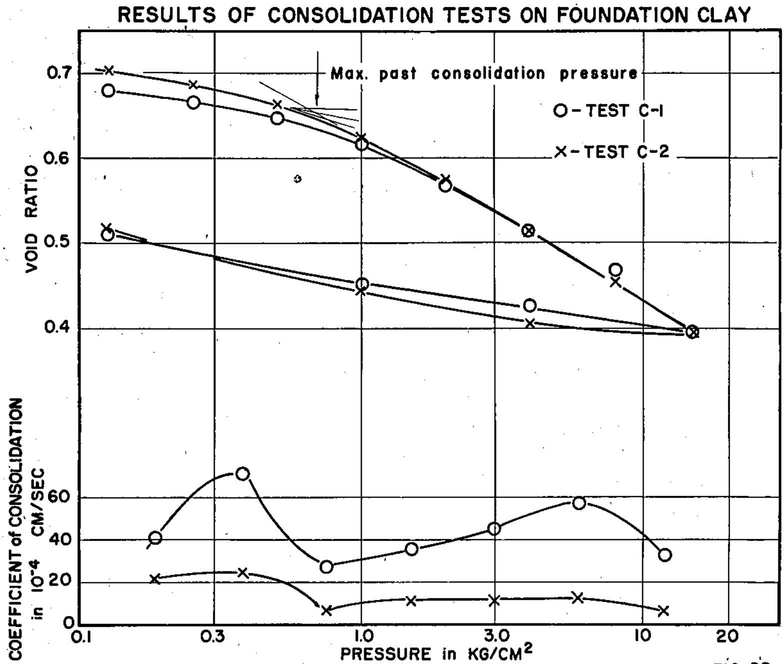


FIG. 29

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%;

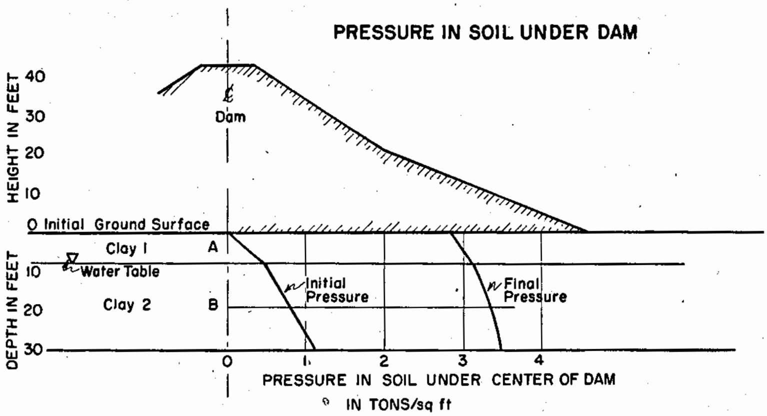


FIG. 30.

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.

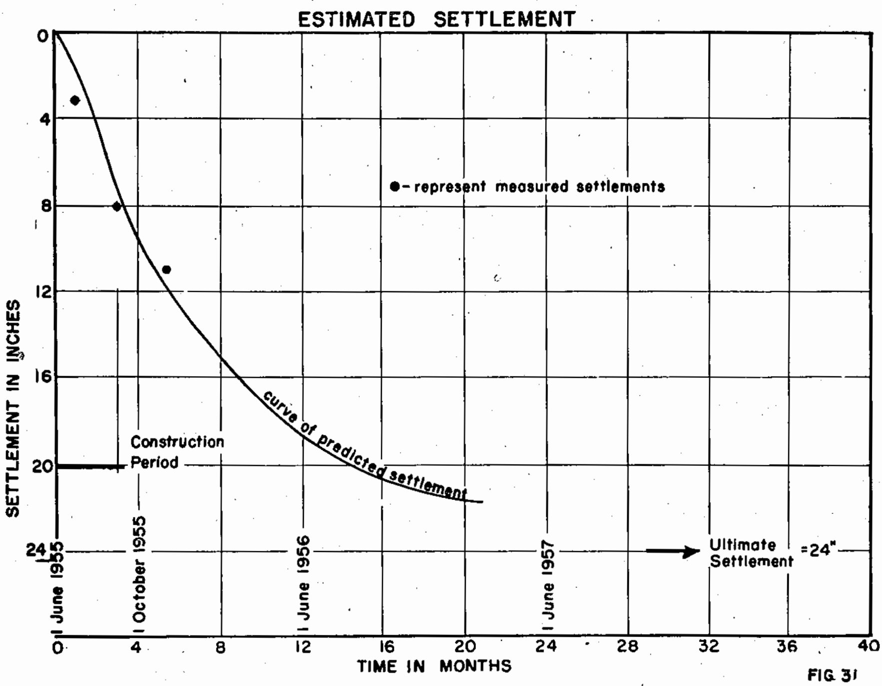


FIG. 31

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-

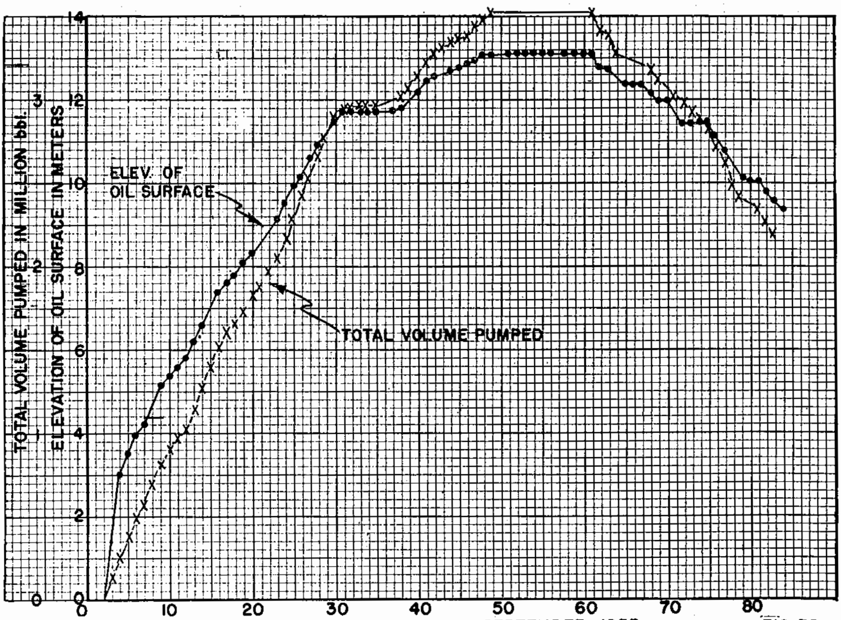


FIG. 32

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.

X. Acknowledgements

The idea of building an earth reservoir for the storage of oil at Amuay came from Creole. The field control and supervision of construction were by Creole engineers.

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Mr. Harry Shea and Mr. Edward Lobacz of the U.S. Corps of Engineers helped in the stability analyses.

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TRANSPORTATION FOR GREATER METROPOLITAN AREAS

By Colonel S. H. Bingham,* AUS (Ret.)

(Presented at a joint meeting of the Transportation Section, B.S.C.E., and the Northeastern Section, A.S.C.E., held on February 20, 1956).

It is a great pleasure and an honor to address this distinguished group on a subject which is important to all of us. I am no prophet, but after 40 years in the transportation field I have some ideas on how the future should look, and perhaps how it will.

The basic transportation problem facing all our cities today is that of opening a way for people and the essential goods of industry and commerce to move freely through streets choked with private automobiles.

There always has been a traffic and transportation problem in large cities and one means of mass transportation after another was developed to solve it. First we developed mass carriers on the surface—horse cars, trolley cars, buses, trolley coaches—then we had to go off the streets, underground and up in the air.

Today we are living through a movement away from the mass transit systems and increasingly greater reliance upon the private automobile. This trend, if unchecked, threatens on the one hand to deprive our cities of essential revenues if interference with the flow of people and goods proceeds to the point where business is forced to decentralize.

The automobile is undoubtedly the characteristic expression of American civilization today. A very large part of our economy is directly based on the manufacture and servicing of automobiles and providing them with fuel and highways. The automobile has enabled millions of our people to travel with comfort and flexibility never before available to people of moderate means. It is a major element of our high standard of living and, in addition to the strictly utilitarian values of the automobile, it has become a symbol of status. Sociologists have noted that families in need have cut down on food

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and other essentials to keep an automobile. Now I am not selling automobiles—perhaps I would be better off if I were. But no discussion of transportation would be complete without recognition of the increasingly important role of the automobile.

Some writers of city problems have stated that nothing can be done, that our cities are doomed, and the sooner we disperse them and decentralize the better. Theoretically, decentralization may be a fine idea. People with more room in which to live and play, factories on large accessible sites, close to their workers—it all sounds fine.

But, practically, it poses many problems. The older central sections of cities from which the outward movement takes place are usually the most valuable areas, and the source of the greatest tax revenue. Their decline creates a financial problem, and unless they are rebuilt, they create slum problems. The peripheral and suburban areas where the growth is taking place require the expenditure of large sums to provide facilities for the new population—streets, parks, schools, hospitals, sewers, water and the myriad other services that people need. And the transportation problem will persist and may become even more difficult. With our mobile population and work force few individuals live close to their jobs. Transportation to the job is either by automobile which requires highways and parking space, or by a mass transit system which finds it increasingly difficult to serve a more sparsely populated area at a low fare.

Decentralization, even if it were possible and desirable, which it is not, is not the answer, at least not within the visible future. Additional highways alone are not the solution to our traffic problem. The transportation problem in our cities today requires a more comprehensive approach which recognizes the inter-dependence of existing transit systems. With the increasing and unrestricted flow of automobiles, the greatest need for parking space is in the most congested part of our cities. It is manifestly impossible to provide downtown all the parking space that might be wanted by everyone who could drive to work.

Since it is impossible in our cities today to provide sufficient highway and parking space for everyone who would like to drive downtown, and since it is essential to get people to and from their jobs, and highly desirable for them to get downtown for other purposes, we must find ways and means of getting them there. This brings us to our mass transit systems, the only alternative.

Despite unparalleled population growth of metropolitan centers, and unprecedented high levels of employment, traffic on mass transit systems has steadily declined since about 1947, and there appears to be no slackening in the trend. This decline has been brought about in part by the almost universal adoption of the five-day week, by longer vacations, more liberal sick leave provisions in industry, television keeping people at home, and similar factors. But a major factor has been the shift of travel to automobiles.

Increasing costs common to all industries, accompanying a declining passenger load, have put the transportation industry in a difficult position.

Fare increases have helped—even though they have generally been too late. But increasing the fares resulted in additional loss of passengers over and above the secular trend. It also seems to me that the point of diminishing returns from increasing fares is being approached and perhaps has been reached on some properties.

To complicate the problem, the loss of transit riding has not been evenly distributed. The loss has been greatest in the non-rush hours. It has, therefore, been impossible to pare operating expenses to keep pace with the decline in traffic. In some respects then, mass transit has come to be a stand-by service—used to full capacity for about four hours—two hours in the morning and two in the evening—five days per week, and during periods of inclement weather when people are reluctant to, or cannot use, their cars.

Despite the decline in transit traffic, there still will always be a large population of riders for whom mass transit must be provided. These are those too young or too old to drive cars—groups becoming increasingly important in our population. There are those who do not have cars, another group becoming more important in the older sections of cities, as the more prosperous residents move to newer areas, or to the suburbs. There are large numbers of transit riders from families where someone else is using the car. And last, there are those wiser people who have been discouraged from driving a car downtown by traffic and the difficulty of finding parking space. For all of these people, and for the business dependent on them, mass transportation is absolutely essential.

Mass transit must, therefore, be encouraged and aided by all the forces of government and society to serve the riders, and to relieve the traffic problem. It is the only alternative to a complex breakdown of traffic flow in our cities today. Every avenue to maximize the use of transit systems must be explored. Such proposals as zone fares to encourage short haul riding, and lower fares in non-rush hours must be studied and tested, if study indicates a possibility of success and satisfactory methods of administration.

One obvious way to maximize use of transit, and solve the traffic problems, would be to bar unessential private cars from the congested areas of the city. This, however, would be politically impossible and practically undesirable. But the same result can be achieved indirectly to a substantial extent. The enforcement of existing parking regulations would go a long way in this direction. Insistence that new buildings make provision for off-street loading and unloading areas and construction of additional parking garages would help. The provision of large low cost parking fields outside the central area and adjacent to rapid transit stations and other transit facilities would be of the greatest help. And finally, improvement of transit equipment is necessary. We can never give the transit rider the same comfort and door-to-door service offered by a private car under the best conditions. But with parking difficulties, door-to-door service is rarely available today; the transit system can offer greater speed in many cases, and lower cost, most of the time.

In the mass transportation industry, we must also encourage technological advances and experimentation to develop new and better facilities to improve the service we offer and to reduce cost of operation. Which brings me—after this somewhat lengthy and perhaps melancholy discourse on transit in general—to two specific innovations—which will be of interest to you.

The Grand Central-Times Square shuttle in New York City needs rehabilitation and modernization and is very expensive to operate. In studying this problem I sought means to develop a more efficient method of handling this shuttle traffic. The shuttle is one of the more heavily used sections of the transit system, carrying over 60,000 people per day in each direction. It is a closed system so that a special transit method developed for the shuttle could be applied without affecting the other parts of the system. It was my desire to devise a way of carrying the passengers within the space occupied by the two center tracks of the four-track route, so that the outer tracks could be converted to walk-ways and air raid shelters.

Some kind of conveyor system seemed to offer a possible solution. Its continuous operation features offered high capacity in a small space, and the low operating manpower requirements would result in a low operating cost. We know that many passenger conveyor ideas had been suggested and some moving sidewalks and similar schemes actually operated, usually at expositions. But there was nothing proposed, or in existence, that seemed to offer any practical application for the shuttle.

I suggested to the Goodyear Tire and Rubber Company and the Stephens-Adamson Manufacturing Company, who had solved many industrial conveyor problems, that this was a fertile field for study.

The major problems were to develop a passenger conveyor system that could handle over 12,000 passengers per hour in each direction with present subway standards of safety, would give a speedy, comfortable and convenient ride, would be less expensive to operate than the present shuttle, and would not cost more than the conventional rehabilitation previously studied for the shuttle. It was desirable to fit this system into about half the space now occupied by the shuttle.

The conveyor system that was designed satisfies all of these conditions. It will carry 16,000 passengers per hour in each direction at least as safely, and more comfortably, than the subway. Passengers will make the trip in two minutes, with no waiting for trains. The present shuttle trip is two minutes of running time, plus the waiting time between trains. It was estimated that operating and maintenance costs would be 40% of present shuttle costs, and that installed in existing tunnels, the cost of the conveyor equipment would be less than 60% of conventional subway equipment. The proposed conveyor would occupy only the two center tracks of the shuttle. The remaining two trackways could be converted to well-lighted air raid shelters and walk-ways and adjoining basements would become potential shops to increase business and tax revenue to the city.

The proposed system differs from previous passenger conveyor proposals in that the passengers would ride in cars carried on belts and on banks of rubber tired wheels. Conveyor belts are used at low speed for passengers and cars at loading and unloading areas and at high speed to transport the cars from station to station. Banks of wheels, equipped with pneumatic tires, accelerate and decelerate the cars between conveyor belts moving at different speeds, and carry the cars around all turns. Passengers at stations walk onto the moving loading belts which have a speed of about one and one-half miles an hour. Parallel to and moving at the same rate of speed is the car loading belt. Riding on this belt are seven foot square, ten seat passenger cars. Nineteen cars pass the loading area every minute.

Passengers step from the loading belt into a car which is empty as it arrives at the loading area and is moving at the same speed as the loading belt. Doors open automatically when the cars reach the loading area and then slowly close prior to the time the cars clear the loading area. The passenger loading belt is nine feet wide and sixty feet long. The car loading belt is of the same length, but only five feet wide, so that cars overhang the belt both front and back, and fit snugly against the passenger loading belt. The cars are closely spaced, bumper to bumper, as they pass the loading area.

Once clear of the loading area, the passenger cars are rolled for about 50 feet over a bank of pneumatic tired accelerator wheels which quickly steps up their speed to 15 miles an hour. The acceleration is at the comfortable rate of two and three-quarter miles per hour per second. Spacing between the cars is increased as they are accelerated and they then ride on to the main line conveyor belt, running at 15 miles an hour. This endless high speed belt transports the cars between stations.

At the other station, the passenger cars are slowed down at the same rate in a 50-foot run over a bank of decelerator wheels and are delivered bumper to bumper on the car unloading belt moving at approximately one and one-half miles an hour. The car doors open and passengers step out onto a passenger unloading belt moving at one and one-half miles an hour, the same speed as the cars, and walk or ride to the exit at the end of the belt. The passenger and car unloading belts are the same dimensions as the loading belts. The empty car is then carried around a loop at the end of the shuttle system on a bank of wheels and deposited on the car loading belt ready to carry another load of passengers. The continuous flow of cars eliminates waiting and platform congestion.

I first broached this idea to those companies in 1948. It took until 1951 to develop a plan on paper that seemed workable and overcame the hurdles of safe passenger loading and unloading on moving belts, and on ways of accelerating and decelerating the cars. In April of 1953 a working model that proved the feasibility of the design was shown in New York City.

During the many months of exhaustive engineering to perfect the conveyor belt subway plan, full-scale testing equipment to solve human element problems, was built. A rubber conveyor belt 60 feet long and nine feet wide and a mock-up of five full-size cars were set up for human engineering. The cars operated adjacent to the belt duplicating actual loading and unloading conditions on the New York City subway belt system. An accelerating and decelerating system was built to determine the maximum rate at which a passenger car could be speeded up and slowed down on a conveyor belt system. These test installations proved the workability of the subway belt system. Hundreds of tests were made with all kinds of people, old and young, nimble and lame, carrying packages and leading children with not one mishap and it has been determined that a belt conveyor is the most efficient and least costly means of moving large numbers of people over short distances.

There is no reason why such a conveyor subway system cannot be extended to longer systems with way stations. All that is required for a way station is a bank of decelerating rollers, low speed unloading and loading belts, and accelerating rollers. Similar and simpler passenger conveyor systems are being developed for large parking fields, airports, railroad terminals, stadiums and other locations where great numbers of people have to be moved quickly and cheaply.

The second improvement to which I refer, which holds out great promise for many cities with transporation problems in which conventional subway construction is not feasible or cannot be financed is the Houston monorail, an entirely new concept of intra-city rapid transit. Before this line had been brought to my attention I had looked with disfavor on monorail operations as I felt no technological improvements had been made during the past 50 years in such proposed developments.

The Houston developers have put some new thinking into monorail planning. This system can operate overhead with stations at ground level, and underground for inter-city or central city operations. It will have the speed and capacity lacking in such proposed developments of the past and will be relatively inexpensive to construct and maintain. It will operate with a minimum of noise and will provide a smooth, comfortable ride combined with safety. The simplified design is pleasing to the eye in contrast to the ugly structures of the past. A pilot line was opened to the public for inspection in Houston last week, consisting of a full-size operating section one thousand feet long. The test confirmed fully the favorable predictions made about this new kind of monorail, and I am more enthusiastic than ever about its possibilities.

I have endeavored to offer some general principles for a solution, and have described some of the newest developments in the transportation industry. While we do not yet have all the answers, we must re-examine our approach to traffic and transportation problems and consider all ideas for a solution to any particular phase, no matter how revolutionary they may seem.

Here in Boston you are fortunate to have a Governor who has vision and is eager to do something about improving the situation and that he is supported by Mayor John B. Hynes of Boston and other officials and public spirited citizens anxious to make contributions toward a common goal.

As I stated in my 1950 report to the then Governor, the quickest and most economical way to encourage the travelling public to mass transportation systems is to make a study of physical connections to be made between mainline commuter services and the Metropolitan Transit Authority. There is an opportunity to provide fast, modern, safe, efficient and comfortable service at an attractive fare. It may be possible for mainline railroads to sell rights of way to the turnpike authority and relieve themselves of unprofitable commuter services. A more attractive service, such as the newly designed monorail could be installed over the turnpike right of way without interfering with surface traffic. Thus, additional turnpike facilities with the newest ideas in rapid transit, together with the use of electronics, centralized train control and the low-center of gravity train, (some of the innovations which I have enthusiastically supported) will go a long way toward relieving transit and transportation problems within a short period of time.

Unless cities can solve their traffic and transportation problems efficiently and economically, it will become expensive to do business in them and the forces for decentralization will become overwhelming. We must be ready and willing to break decidedly with the past if we are to improve upon our present modes of urban transit.

Surface traffic in the heart of a city should be kept to a minimum. Its primary function should be to serve as a feeder to rapid transit, rail and express bus lines at key stations. These stations ideally should be on the fringe of business areas and include ample parking facilities for private cars. The motorist should park his car at such a perimeter facility and then proceed to his destination via an efficient local mass transportation system.

In closing let me say that I believe the present study by Governor Herter's Committee on Regional Transportation is a step in the right direction. The downtown traffic congestion is a problem which concerns the greater metropolitan area and requires a coordinated study of all media of transportation. The important thing to point out, however, is the need to plan along generous lines and to provide adequate facilities not only for the demands of the near future but the probable travel demands of 25 to 30 years from now.

Great progress has been made in this age through engineering opportunities for those in the engineering profession are as good if not better than ever before. The engineer with a fresh view of the problem, using the broadest possible approach to its solution has the best chance of coming up with the right answer.

SOME NEW CONCEPTIONS AND OLD MISCONCEPTIONS OF PHOTOGRAMMETRY

BY CHARLES L. MILLER*

(Presented at a meeting of the Surveying and Mapping Section, B.S.C.E., held on April 4, 1956).

Introduction

During the past ten years, we have heard and read many papers and articles about what photogrammetry can do and how it can be applied to civil engineering as a method of mapping. The emphasis has been primarily on the value of photogrammetry with little effort expended on explaining the subject to the engineering profession. The purpose of this paper is to present some simple basic concepts and to correct certain misconceptions.

The Misconceptions

Engineers have a number of common misconceptions of photogrammetry. Some of these include: photogrammetry is

(1) Relatively new
(2) Only applicable to map making
(3) Not very precise
(4) Only applicable to "preliminary" engineering
(5) Rather mysterious
(6) A minor professional field
(7) Conducted only by technicians

The reason for these misconceptions is principally due to a reluctance on the part of the engineer to take the time and effort to obtain a basic understanding of the subject. As with most technical subjects, although the details are many and the ramifications complex, the basic fundamentals are few and simple. The full potential of photogrammetry has hardly been realized and is being retarded by a lack of common understanding and appreciation for the subject.

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A Basic Definition

Photogrammetry is defined by the American Society of Photogrammetry as "the science and art of making measurements with photographs." It is noted that this definition does not limit us to any one type of photography such as aerial photography nor to any one field of application such as map making.

Photogrammetry is essentially a method of making space measurements. Therefore it is not necessarily a separate field of endeavor divorced from engineering and surveying, but rather a tool to be utilized by all who require space measurements.

The engineer readily accepts soil mechanics as a tool to assist him in the design of his structures. He should also accept and study photogrammetry as a tool to assist him in the planning and design of his projects. In both cases—soil mechanics and photogrammetry—the expert has his place. But also in both cases the potential value of the fields will only be realized when each engineer has at least a basic understanding of and appreciation for the subjects.

How New

Although photogrammetry has experienced its greatest advance in application and acceptance in the United States during the last ten years, its development dates back over a century. Some of the principles of the science of measurement with pictures date back to the early 1700's before the discovery of photography.

Laussedat, the Father of Photogrammetry, first used the photographic camera for space measurement in 1851. The forerunner of our modern stereoplotters appeared about 50 years ago. Photogrammetric surveying by private firms in the United States is about 35 years old. Photogrammetry is therefore not something new but a method of measurement backed by over 100 years of active scientific development. It is new only in the sense that it is still in the process of coming to the attention of some of our engineers.

A Basic Concept

If we consider photogrammetry as a means of creating a spatial model of the object photographed and as a means of measuring this model, one can easily comprehend how it can be applied to a tool of space measurement. This simple concept of the method removes much of the mystery—we merely create a model and measure the model instead of the actual object. It is therefore applicable to anything that can be photographed.

Where Used

We can say that photogrammetry as a method of space measurement can be used to advantage anywhere it would be difficult and uneconomical to directly measure the actual object. For example, photogrammetry might be applicable if the object is:

(1) *Very large*—such as a portion of the surface of the earth.
(2) *Very small*—such as sand grain or roughness of a metal surface.
(3) *Moving rapidly*—such as a structure under dynamic loading or ballistics missile.
(4) *Changing rapidly*—such as a hydraulic phenomenon.
(5) *Inaccessible*—such as a moving dust particle in a furnace or atomic particle in a cloud chamber.
(6) *Complicated*—such as a deformed structure or the human face.

In the light of this concept—if it is difficult and uneconomical to directly measure the object, create a model of the object and measure the model—the unique uses of photogrammetry periodically reported are not so amazing but are natural applications.

Photography and Stereovision

A photograph is a perspective projection of the object photographed. It is a graphical record of a set of directions. In a perspective all of the rays from the object converge to one point. A perspective projection is the trace of the intersection of these converging rays with a plane located between the object and the point of convergence. The basic difference between photographic surveying and conventional surveying is that in one case we measure angles with a transit, and in the other with a camera. The photogrammetric camera may be thought of as an angle recording instrument. The human eye sees essentially a perspective projection. We are able to perceive depth because each eye sees a different picture. The eyes essentially triangulate distances—the distance between the eyes being the baseline. Points appear different distances away because they form different angles opposite the baseline. This angle is called the angle of parallax and is a measure of distance.

If we replace the eyes by cameras and record photographs of what the eyes would see, and then place the photographs in their proper position in front of the eyes, we would perceive the identical result as directly viewing the object photographed. Hence we can duplicate natural stereovision by means of photographs. Since the angles of parallax exist in both cases, we can perceive and measure depth by means of photographs. Essentially we require two photographs of the same object taken from different camera positions.

Creating the Model

When the photograph was formed, rays came from the object to the photograph. If we return to our camera position with a projector (which reverses the direction of the rays) the rays will return to their point of origin on the object. Similarly, if we have a second projector projecting the photograph from a second camera position, the rays from a given image point appearing in both photographs will intersect in space at their point of common origin. Under these conditions, we will project a three dimensional model coinciding in space with the original object.

If we hold the two projectors in their relative orientation, but move them closer together, the rays from common image points will continue to intersect in space but will form a smaller size model. The ratio of the distance between the projectors to the distance between the camera stations is a measure of the scale of the model formed by the spatial intersection of the rays.

The spatial model formed by the projectors cannot be perceived by the unaided eyes. As mentioned before, each eye must be presented with a separate and different picture. Therefore, one eye is only allowed to see the picture from the first projector and the other eye is only allowed to see the picture from the second projector. When this is done, the mind will fuse the two sets of images into the spatial model which appears as real to the mind as viewing a physical object.

The Fourth Dimension

In addition to the three linear dimensions of x, y, and z, the spatial photogrammetric model has an important fourth dimension—time. Our model is a true replica of the object photographed at a specific instance of time. This fourth dimension is the most important element in many types of problems. For example, if the object is moving or changing rapidly here is a method of freezing its exact shape, size and space location at any selected time or intervals of time for later measurement and study in the laboratory.

The object does not necessarily have to be moving or changing rapidly. The movement of glaciers and the settlement of large areas of terrain over a period of time (such as in the Los Angeles area) have successfully been measured by photogrammetry.

The Need for Control

We previously stated that a photograph is only a means of measuring or recording angles. In triangulation, we know that we require the length of at least one side of a triangle (a baseline) in a network of triangles before we can compute the triangles. Angles alone do not define size. The same is true in photogrammetry. We have to measure at least one distance in our system in order to determine size or scale of the model.

A second basic requirement in any surveying system is horizontal orientation. What are the directions of our lines with respect to our space framework reference axes? Here again, as in ground triangulation, we need the horizontal orientation of at least one line in our model. These first two requirements, scale and direction, can be satisfied by having two points in the model of known horizontal position. The distance between the two points establishes scale and the bearing of the line between the two points establishes direction.

A third consideration is the establishment of a datum plane. Since it takes three points to determine a plane, we need the differences in elevation between three points in the model to define the datum plane. If we are compiling a topographic map and would like our elevations referenced to our standard datum, we would need the mean sea level elevation of one of the three points.

In summary, to locate our model within some established space framework, we would need at least two horizontal and three vertical control points located within the model and referenced to the desired coordinate system.

We have spoken so far about an individual model formed by two photos, say A and B. However, photos B and C, C and D, etc., may form additional models which together form a continuous model of a large area. The individual models can be compared to individual triangles in a triangulation system. If we know everything about one triangle, we can compute the location of the rest. Similarly, if we have oriented, scaled, and referenced one of the individual models, we can determine the space location and all the points therein of the other models. Just as with ground triangulation, we can only continue this process of aerial triangulation until our error accumulation becomes too large. The important point is that we do not necessarily need "outside control" in every individual model.

Measuring the Model

The ultimate goal of most of our conventional surveying operations is the x, y, z coordinates of selected points, the graphical record of the xy position of selected lines, and the representation of relief by contour lines. In our photogrammetric method, once we have created and oriented our model, we achieve these goals directly without any intermediate measurements, computations, adjustments, or plotting. How this is done may be understood if you visualize that within the spatial framework of the model, we insert a reference mark in our instrumentation system such that it can be freely moved in model space, and such that its model space position is always known. (How this is accomplished may easily be demonstrated mathematically or graphically but we are dealing here with concepts and not technical theory.) With this movable and calibrated reference mark, we have the perfect surveying tool. This may be likened to the surveyor having a little black box with three dials that always indicate the x, y, z's of the box as it is carried from point to point. To trace the plan position of a line, we merely move the index along it. To run out a contour, we set the z value of our reference mark to equal the elevation of the desired contour. The mark is then moved until it touches the surface of the model. This is one point on the contour. Now if the mark is moved but constantly kept in contact with the model, we are tracing out the desired contour. A pencil point is coupled with the reference mark so that our point, plan line, or contour line is automatically plotted.

The Stereoplotter

There are three common approaches to solving engineering problems: (1) analytical or mathematical (2) graphical (3) mechanical. By mechanical we mean with a machine or instrument. Most photogrammetric problems may be solved by any one or a combination of these approaches. For carrying out the concepts previously presented and in actual practice, we find the third approach to be the most feasible.

The stereoplotter is the most basic and important instrument of applied photogrammetry. The stereoplotter is an instrument for creating and measuring the spatial model and recording the results. Practically all work in applied photogrammetry revolves around the use of this instrument. In addition, much of our theoretical and research in photogrammetry work is concerned with stereoplotters.

A stereoplotter consists essentially of (a) two or more projectors or other means of forming the spatial intersections, (b) facility for moving these projectors along and around three mutually perpendicular axes to reconstruct relative and absolute orientation, (c) means of separating the projections for the eyes, (d) a movable and calibrated reference mark, and (e) a plotting system.

A stereoplotter represents one of the highest forms of precision optical and mechanical instrumentation built by man. Although they all perform basically the same function, there are a number of different types and makes of stereoplotters available to the engineer and scientist. The commonly used plotters in the U. S. include the Bausch and Lomb Multiplex, Kelsh Plotter, Wild Autographs (Swiss), and Zeiss Stereoplanigraphs (German). Last year Bausch and Lomb introduced a new stereoplotter trade named the Balplex. Other important stereoplotters include those of Pivillier (French), Santoni (Italian), and Nistri (Italian). These instruments range in cost from approximately $4000 to approximately $75,000.

How Precise

All measurement systems and their instrument components contain certain sources of error. We therefore design our measurements in such a way that we can (1) reduce the errors to a tolerable level (2) correct for the errors by instrument design or measurement technique, or (3) make secondary measurements to enable us to compute the errors and correct the primary measurement. Of course we never completely eliminate errors but reduce the total error until we obtain the desired precision.

In this respect, photogrammetry is no different from any other measurement system in that it contains many sources of error. However, the photogrammetric engineer can reduce, correct, or compute these errors and so design any measurement project to obtain the desired precision.

In the convenience of the laboratory, we know that we can measure to 1/1000 inch with several common instruments with relative ease. In the drafting room we measure and plot points to the nearest 1/100 inch with a good engineers' scale. Therefore, measuring and plotting small increments of space is not unusual in the laboratory. Previously we presented a concept of photogrammetry as the creation of a spatial model of the object photographed and the measurement of the model. The precision of the method then involves how precisely we can create the model and how precisely we can measure it. The precision with which we can create and measure the model depends on many factors such as the camera, flight height, control, the stereoplotter used, human element, the nature of the object, and the design of the over-all system. Errors originate from these and many other sources. What is important is the value of all of these individual errors at the scale of the model. For example, in a modern well designed photogrammetric system, the total error at the scale of the model might be of the order of 6/1000 inch or 0.0005 feet. We therefore present a concept that the precision of our photogrammetric measurements is a function of the size of the model. For the conditions of our example, we could set up a table as follows:

If the Allowable Error Is	We Need a Model to a Scale of
0.0001′	1/0.2
0.0005′	1/1
0.001′	1/2
0.01′	1/20
0.1′	1/200
1′	1/2,000
2′	1/4,000
5′	1/10,000
10′	1/20,000
20′	1/40,000

In essence, the higher the precision, the larger the model for a given photogrammetric system. If measurements to 1/10,000 foot are required, we need a model five times the size of the actual object. If measurements to 10 feet are required, we can use a model 20,000 times smaller than the object photographed.

The error in running a contour line is about twice that of a discreet point. Since the allowable error in a contour is one-half the contour interval, the model sizes given in the table above must be multiplied by four if the numbers in the first column are contour intervals. For example, two foot contours would require a model scale of approximately 1:1,000.

It is obvious that there is no inherent limit to the precision of photogrammetry. In fact, with microphotogrammetry, measurements to the order of microns at the scale of the object are entirely feasible. When people speak of limits for the precision of aerial photogrammetric mapping, they really mean there are minimum practical operating limits on the altitude of an airplane.

Photogrammetric engineers work in terms of the measurement precision being a function primarily of the flight height or object distance. However, it can be shown that the concept of precision being a function of model size is compatible with other approaches and is presented primarily because it is easy to visualize.

Photo Analysis

In this paper, we have dealt entirely with basic concepts of the measurement aspect of photogrammetry. In addition to recording angles, the photograph records an infinite amount of qualitative information. Therefore, photo analysis and photogrammetry are closely allied but somewhat separate fields of endeavor since the areas of knowledge involved are quite different. Mathematics and physics are basic to the photogrammetric engineer whereas geology and agronomy are more important to the air photo analyst.

The nature and extent of the information and data which the highly trained professional photo analyst can obtain from aerial photos borders on the fantastic. Soils, drainage, geology, land classification, vegetation, human activity are only a few of the many areas of data that can be obtained.

It is not within the scope of this paper to discuss the qualitative aspect of photography but it is important to note the distinction between the work of the photogrammetric engineer and the photo analyst.

The Photogrammetric Profession

The final two misconceptions to be discussed are the size of the field and the type of people involved. For some time, we have lacked accurate factual statistics on the size of the photogrammetric activity in the United States. This is the subject of a current study at M.I.T. We feel this study will be quite revealing and establish that photogrammetry is a major field of professional activity.

It is true that photogrammetry utilizes a large number of technicians just as most professions do. We might compare a photogrammetric project with building a bridge. Just because the actual labor of building a bridge is handled by a large number of "technicians" and construction workers does not mean that no engineers are involved. Similarly, the fact that technicians furnish the labor in a photogrammetric project does not mean that engineers are not involved in planning, designing, and constructing the project. The professional engineer and scientist is as essential in photogrammetry as any other professional field. The tremendous advance and technical development of photogrammetric methods and instruments in recent years is sufficient evidence that a large number of high level engineers are hard at work in photogrammetry. Research activities alone occupy the services of a very large number of engineers and scientists.

The civil engineer educated and trained in photogrammetry has before him one of the greatest challenges to explore and develop new frontiers available to the professional man.

SOME EXPERIENCES OF A SANITARY ENGINEER AS AN EXPERT WITNESS

BY E. SHERMAN CHASE,* *Member*

(Presented at a joint meeting of the Sanitary Section, B.S.C.E., and the Northeastern Section, A.S.C.E., on May 16, 1956).

As a speaker on the subject of expert testimony, I shall have tc confess at once that I am an amateur. What I am going to say is not based on any textbook about how a witness should behave, how he should react to direct and indirect questions, and so forth. I shall base my talk primarily upon my own experience and the conclusions which I have reached myself as to how expert testimony should be given.

There are, ordinarily speaking, two types of testimony—factual testimony and expert testimony. The lay witness can only testify as to facts. He is not allowed or expected to express an opinion. An expert witness can give opinions based on his experience, his studies, or upon his research. He also is quite apt to be a factual witness as well as an expert opinion witness.

In giving testimony as an expert witness or as a lay witness, you are first subjected to direct examination by the attorney, who hopes or expects you to give testimony favorable to the client he represents. After you give your direct testimony (the technique of giving direct testimony is a little different from giving testimony on cross-examination), you are turned over to the lawyer on the other side for the cross-examination. One of the most important things which a witness should remember is that his manner of presenting his testimony may carry as much weight as his actual testimony, and sometimes even more weight. That is particularly true in the case of testimony being given before a jury.

The lawyer, also, in framing his questions and in his examination, modifies his questions and his manner of asking those questions, according to whether he is having a case tried before a jury or before a master or a judge. A master or a judge is not inclined to be

*Partner, Metcalf & Eddy.

misled, let us say, by dramatics. A jury may be swayed by dramatics and by oratory. Judges and masters are not.

It is, I would say, always better to be giving testimony before a judge or an auditor rather than before a jury, because the expert is usually testifying on technical matters which are difficult for the average layman to comprehend. Judges and auditors are lawyers. They have been accustomed to understanding complex problems of all sorts, from engineering to medical. It is my experience that the average lawyer or the average judge can grasp the essentials of technical matters with relative accuracy. Not always, but by and large, they make a much more satisfactory audience than jurors, for the presentation of testimony of a technical nature.

On the other hand, there are times, as I have seen happen, when if you are very young, quite boyish-looking, and the attorney on cross-examination gets you mixed up, the jury is so aroused by the rough tactics of the cross-examiner (especially if there are women on the jury) that the jurors forget the merits of the testimony and are inclined to side with the young witness.

I remember one case in Ohio quite some years ago (a pollution case), when the witness was a young chemist from our office. He was shortly out of Worcester Polytechnic Institute. He was a nice-looking young man, and in cross-examination he was confused. He blushed easily. He looked so honest and was trying so hard to be correct in his testimony that I could see the several women on the jury looking at him with the greatest of compassion. I knew right away that the side for which he was testifying had acquired friends on that jury. The attorney had made the mistake of badgering our young witness a bit; and, although he was confused, the confusion was offset by his obvious honesty and his good looks. To the jurors, it seemed as though he were their son and they felt for him as for a son.

The expert witness, as I said a little earlier, is permitted to express opinions based not only on his study of the particular problems involved in the case but also on any study that he ever made which was germane to the particular problem involved.

Sometimes that sort of testimony gets sidetracked. I remember a New Hampshire case. Arthur Weston was testifying in connection with the protection of the sanitary qualities of a water supply pond where the matter of bacterial analyses was being considered in court.

When Mr. Weston started to testify as to the interpretation of the results of the bacterial examinations, the attorney for the opposite side objected. I have always felt that the attorney for the side Mr. Weston was testifying for was a little bit lax, because the other side's attorney, objecting to Mr. Weston's interpretation of bacterial analyses, said to Mr. Weston, "Did you ever make any examinations yourself?"

"No, I never did, but I have interpreted hundreds of analyses."

"Well, since the witness has not made examinations himself, I believe Mr. Weston should not be permitted to give testimony because he has never made any analyses."

The judge allowed the exclusion of Mr. Weston's testimony. That is not in accordance with the usual rulings in similar situations, because Mr. Weston could have said that as a result of his study and experience he could and did interpret bacterial examinations, even though he had never made analyses. When the lawyer for the opposition came to me, and objected to my testimony, saying, "You never made any bacterial analyses yourself, have you?" I replied, "I have made hundreds." That shut him up.

There are certain precepts, certain advice that may be of value to some of you younger men who may be called upon to testify in court on technical matters.

Of course, one of the very first things that an expert must do—and his attorney must see that he does it—is to show that he can qualify, right up to the hilt. If you are modest you will hesitate to blow your own horn; you will hesitate to tell all you have done and the honors you have received. But don't let modesty deter you from giving all your qualifications.

The lawyers of utility companies, when they have an expert witness to put on the stand, qualify him right up to the hilt. This is necessary, not just for the court in which the case is being tried, but also at times for the record. You have to remember that some of the questions and answers that are brought out in court are for the record and not for the judge or jury.

Whether you are a lay witness or an expert witness, state the facts. If you are not quite sure of the facts, refresh your memory on the case by reference to notes, if you have them. It is very easy to get misled by your own memory. Memory is a very tricky thing.

I remember a case in Dayton, Ohio, where a large sewer of the city of Dayton was discharging at the time of the trial, through an outlet about 5 ft. above the normal water level of the Miami River. (This was a pollution nuisance case brought by residents below the city.) The sewer discharged into an arm, or bayou, of the river in which sludge deposits had taken place and had caused a generally unsightly condition. The witnesses for the plaintiff, described the conditions exactly as they were at the day of the trial. In the state of Ohio there is a 6-year statute of limitations. The attorney for the plaintiff asked these witnesses to describe conditions as of that morning. Also, "How was it last year?"

"Just the same." Each witness testified that for 6 years back the conditions were "just the same."

When the witnesses got to the seventh year, they did not remember; but for 6 years they were sure.

As a matter of fact until about 3 years prior to this date of the trial, the sewer discharged under water. This shallow bayou was then the main thread of the stream, and the bridge which they described as being there had not been built. Now they remembered—or they had assumed—that that condition had existed for 6 years, but it had existed only 3 years. This is one of the hazards you may run into if you try to remember the conditions which existed some time ago.

One of the real difficulties of an expert in giving testimony on technical matters is the presentation of his material in nontechnical, homely language. I remember another case in Ohio. One of the witnesses for the defense was a young chemist. The case revolved in part around the discharge of acid-iron wastes into the Portage River. This young chemist had been talking about ferrous sulfate. When it came to cross-examination, the attorney said to him, "Well, now, Mr. ————, you have been talking about ferrous sulfate. Can you spell it for us?" "Oh," said the chemist, "$FeSO_4$."

Another moral: Instead of learning to express themselves in ordinary familiar language technical men show a tendency to use technical language rather than English. Technical witnesses must remember the very important need for simple illustrations and nontechnical language.

I recall one case where I was talking about organic matter. Organic matter means something to the chemist, or to anyone who is studying chemistry but little to the layman. In speaking about the decomposition of organic matter, I described the sources of pollution which contribute organic matter, and how such organic matter is decomposed. "Even with a bouquet of the most beautiful flowers which you may have in your house in a vase of water, if the water is not changed, the stems of those flowers will undergo decomposition, and the water will smell to high heaven."

That is the type of illustration that the ordinary layman understands. Nobody has ever failed to empty the water from a vase of flowers after they have been kept very long without a change of water! This example illustrates a natural source of organic pollution, and also illustrates the decomposition of organic matter.

A trick question that lawyers always love to ask, (along the line of "Have you ever stopped beating your wife?") is this one, "Have you talked with anyone about this case?" The man testifying for the first time thinks it is wrong to discuss the case with anybody. This is not so. Unfortunately, a nervous witness will say that he has not, when you know he has. Then he will have to admit that he has talked it over with somebody, and so is discredited in the eyes of the court. Say, "I have talked it over with the lawyer." Admit it.

I remember a former federal judge in Ohio, whom I knew as a practicing attorney before he was a judge. He said that lawsuits are not won in the courtroom but outside, in the way in which preparation has been carried on. This applies not only to the attorney but also to the expert. If you are going to give expert testimony, it is, of course, most important that you prepare your material just as thoroughly as possible. Be prepared as best you can, with respect to the matters which you are going to present, by anticipating the case that the other side is likely to put on. Be prepared for the questions which may be asked on cross-examination. Understand the weaknesses of your own case, and the strength of the other fellow's case; and, by all means, brief your attorney or the attorney representing the side retaining you. Prepare him, too, as thoroughly as possible.

It is my experience that, as a rule (there are always exceptions), the younger attorneys are quicker to catch on to technical matters than are the older attorneys. I have seen high-ranking attorneys with a long-established reputation find it quite difficult to grasp the technical aspects of some cases I have been associated with; but younger men catch on with a good deal of rapidity.

In direct examination, you have somewhat more latitude than on cross-examination, and in direct examination it is not unwise sometimes to give a little lecture on the subject about which you are testifying. The lawyer who is examining you directly may ask a question which gives you an opportunity to expand your answer to the benefit of the court as a matter of education.

On the other hand, on cross-examination it is probably wise to answer "Yea, yea," or "Nay, nay," because the cross-examination questions may be loaded to catch you in a trap. Again, they may not: they may be straightforward. But, if you volunteer information on cross-examination, you may find yourself subject to embarrassing moments.

I recall once being asked something like this, on a water supply case: "When you came to Hudson Falls, didn't Dr. X tell you that there was never a case of typhoid traceable to the water supply?"

"Yes, I suppose he did, that's what they always tell me."

The attorney for the opposition then raked me over the coals. "Do you mean that Dr. X would willfully mislead you?"

I felt like a plugged nickel. That is what happens oftentimes, when you try to be smart. I got it, and I deserved it.

Occasionally, however, on cross-examination the opposing attorney will lay himself open, and then, if you are alert, you can very well turn it to the advantage of the side which is retaining you, for whom you are testifying.

On one case of cross-examination I was asked, "Why did you arrive at that opinion?"—Which immediately gave me the opportunity to explain five reasons for my opinion. That is a very unwise question for an attorney to ask because, if he has a witness who knows what he is talking about, the witness has the right to answer, and the answers may not please the questioner.

Another very common cross-examination question is one like this: "Isn't it possible that a man could fall from the fifth floor of the 'Y' Building and live?" The answer should be: "It would be possible, but highly improbable." That usually will shut up the cross-examining attorney. I was asked that question once and replied in just those few words. The Assistant Attorney General said in a displeased tone, "I did not ask you to make a speech."

It is unquestionably true that there are two sides to every lawsuit, and as an expert you will probably find yourself forced to make admissions. The late Thomas W. Proctor, who was a well-known lawyer in Boston, said, "When it comes to making admissions, make them as though it didn't matter. Make them, but in a manner that indicates that they did not count in the scheme of things."

Another favorite trick of some attorneys is to say (after a recess), "This morning, Mr. Witness, you said that two and two make five." You don't know but what you did say so, but you had better say, "I didn't say that." The lawyer for the other side is trying to get you to make contradictory admissions.

If a question raised in court doesn't make sense—sometimes the questions of the attorney for the side you represent don't make sense—let your own attorney down as easily as possible, but when it comes to the cross-examiner, don't hesitate to say, "That doesn't make sense to me, Mr. XXX." One attorney has stated, "You experts generally have it all over us lawyers, talking on something we don't know too much about. You sometimes embarrass us by saying, 'Now, to me, I don't understand that question,' or 'That question doesn't make sense—doesn't sound intelligent to me.'" That throws them.

One of the essential factors in expert testimony, or in any testimony, is to be thoroughly convinced that the truth and the virtue lie on the side which you represent. If you don't believe that your tie-up has adequate grounds either for defense or for offense, you had better not take the job on.

Present your testimony in as convincing a way as is possible for you to do. Talk what you believe, and believe what you talk.

There was a case some years ago where the witness on the other side had a hesitant way of testifying. In talking with the court stenographer afterwards, I said to her, "What do think of his testimony?"

"I don't think he believed it himself."

As it happened, he had a hesitant, nonconvincing way of presenting his testimony.

As far as possible, present your evidence in as objective a manner as possible. Present it in much the way you would present a technical paper before an engineering group. You don't present a paper before a group like this without believing what you are talking about, and a convincing manner is one of the very important things in connection with giving any kind of testimony, for that matter.

Some of the funny cases I have been mixed up with are really something out of this world.

The funniest one I ever had was some years ago when Mague the garbage contractor for the city of Newton, Massachusetts had a transfer station on Lexington Street between Auburndale and Waltham. An abutting owner, named Keith claimed that this transfer station was operated in an unsanitary and objectionable manner and brought action to have Mague enjoined from operating the transfer station. Mr. Proctor was Mague's attorney. The court gave Mague a year in which to correct conditions. At the end of that time, after Mague had spent ten to fifteen thousand dollars to improve conditions, this chap Keith petitioned to have Mague adjudged in contempt of court for failing to correct conditions.

Then we got into the situation, and one of my jobs was to run out and sniff the ambient atmosphere and report back to the attorney. The work Mague had done and the way he was conducting the station had materially improved the situation. The garbage wagons still drove in and out. They still looked unsightly. The station didn't stink, and there were no odors over the property line.

One day I was making an inspection along the line between Keith's property and Mague's and got over onto Keith's property and his daughter ordered me off. During the next inspection as I went along the line, very carefully keeping on Mague's land, there was a chap haying on both the Keith and the Mague property. So, as I walked by him, I spoke and said, "Say, are you cutting hay for both Mague and Keith?"

"No, I am cutting it for Keith."

"Did you know you were over on Mague's property?"

He said, "No. I will go down and see the old man, and make it all right."

The next week when I made my inspection the chap was raking up the hay, loading it on the wagon, but was on the Keith property close to the line. I said, "Well, did you get straightened out with Mr. Mague?"

"No, Mague said 'Your mistake, let her lay.' I'll get even with the old x%"#$%&X%$#%."

When I came back to the lawyer's office I told him Keith had a new witness. Sure enough, when the case was heard, this chap appeared as one of the witnesses. He testified that, when he was making hay on the Keith land, the odors from the transfer station were so nauseating that he had to get down from the load of hay and lie down in the shade of Keith's house. The following then ensued:

Cross-Examiner: "Now, Mr. X, you don't like Mr. Mague."
Mr. X: "Oh, I ain't got nothin' against the man."
C-E: "Didn't you have trouble with him?"
Mr. X: "Oh, yes, I remember that; it was nothin'."
C-E: "You were pretty mad."
Mr. X: "Oh, no!"
C-E: "You didn't call him hard names?"
Mr. X: "Oh, no!"
C-E: "You didn't call him a x%#"#$&*¢X%&$%?"
Mr. X: "Oh, no, I seldom use that language."
When you are testifying, be careful of your language.

In connection with that case, the washings from the garbage wagons and from the transfer station itself were flushed down into a large leaching cesspool. I had looked into this cesspool and had seen the highest point to which the contents had risen. Acid digestion had taken place and the accumulation of garbage had become a more or less homogeneous mass of yellow-gray sludge. Up near the top of the cesspool there was a 10-in. drain leading off to the top of a bank which, in turn, sloped down to a marsh. The contents of that cesspool had never risen anywhere near this overflow evidently put in as a safety measure. But on the day the view was held when the auditor came out to see the premises, Keith led the auditor down to the point where this drain came out near the top of the bank, Keith was going to show the auditor the end of this overflow pipe, which Keith claimed had been in operation. I was suspicious of the way Keith acted when it was found that everything below the drain was clean and dry, so I went poking around the bushes below the end of the drain and down the slope. Out of sight hidden by the bushes was a half peck of fresh garbage.

You have heard of "salted" mines. This was a case of "salted" garbage at the end of the drain. What had happened (after the "salting") was that the day before the view there had been a heavy thunder shower, and this "salted" garbage at the end of the drain had been washed out of sight by the runoff from the land above the drain, much to the disappointment and puzzlement of Mr. Keith.

Another rather amusing experience was in connection with a well in Connecticut, which, it was claimed by the owner, had been polluted by drainage from the City dump. We had had test wells put on a semicircle around this well, claimed to have been polluted by the dump, and we had taken samples from these wells at periodic intervals, as well as from the well itself, to see if there was any evidence in the groundwater of the pollution that was attributed to the dump.

As an additional test, we put a well in the dump itself, which was a few hundred feet distant from the plaintiff's well and dumped into it a load of salt. We did it after dark, because we didn't want embarrassing questions, but did want to find out whether or not there would be an increase in the chloride content of the water. There was no change as far as chloride was concerned. That was not too conclusive, so in the course of the defense we did not use that negative evidence as we might have; it was not too good. The expert on the other side, when he was put under cross-examination, was asked about the tests he had made, and his test analyses, which incidentally checked with our own analyses. Then the attorney for the City inquired if he had made any more tests. He was hesitant, so the attorney pressed him. He admitted that he had dumped a load of salt in the dump, and that his results were also negative. The dump had had two doses of salt, instead of one and neither side had cared to put their results into evidence, although for different reasons.

One final word I would like to leave with you in connection with testifying. Particularly in answering cross-examinations, think first about what the question really means; second, don't be led into hasty, ill-considered answers. You may find yourself in the position of having to explain, which is always very bad. Be a little more afraid of the smooth, polite cross-examiner than of the browbeating type.

I remember Mr. Eddy, Senior, saying that the cleverest cross-examiner that he had ever been up against was Charles Choate, Jr., who asked his questions in such a pleasant manner that you wanted to answer them just as he wanted you to answer.

Another example of the value of, first, being prepared, and then of taking your time in answering, was an incident that occurred in the case of Connecticut vs. Massachusetts, on the diversion of the Swift and Ware Rivers. Mr. Eddy, Senior, was testifying in behalf of the Commonwealth. One afternoon he hadn't finished yet someone from our office observed that the attorney from Connecticut, and one of the experts were huddling over a small book. The observer also saw that this book was the report which Mr. Eddy had made about 1906 to Cincinnati, on sewerage for that city.

That evening, in the course of preparing for the next day's testimony, we were given the report in our office, to comb it with a fine-tooth comb and see what the Connecticut people had up their sleeves. We came across some paragraphs which, it seemed to us and to Mr. Eddy, could be thrown up at him the next day.

Sure enough, he was handed this report. "Now, did you make this statement?" and they read from the report.

"May I see that report?"

"Oh, yes."

He took it, looked it over, turned back one page and said, "Now, if you will turn back to the previous page, you will see that I said so-and-so." The statement in that report, when taken as a whole, had no contradictory effect on his testimony although the excerpt the attorney tried to get in the record might have done so.

This is a favorite trick with attorneys—to take something you have written which seems to contradict your testimony—so remember what you have written.

In another case in Ohio the attorney asked me questions on points which, so far as I could see, had nothing to do with my testimony or the case. I answered him anyway. Later I talked with one of the attorneys about those questions. He said, "We were reading them right out of American Sewerage Practice. You answered almost verbatim."

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

APRIL 23, 1956.—A Joint Meeting of the Boston Society of Civil Engineers and the Northeastern Section of the American Society of Civil Engineers was held this evening at the M.I.T. Faculty Club, 50 Memorial Drive, Cambridge, Mass. A dinner was served after which the meeting was called to order at 7:45 P.M.

President Edward C. Keane of the Northeastern Section, ASCE, was presiding and after introducing the head table guests called upon President John G. W. Thomas of the BSCE to conduct any business of that Society. President Thomas called upon the Secretary who announced the names of applicants for membership and also that the following had been elected to membership:—

Grade of Member—Henry H. Baker, Jr., Paul S. Breck, Raymond F. Fitzmaurice, Joseph E. Heney,* Bernard S. Johnson,* Charles L. Miller, Saul Namyet, Alex Ostapanko, Gino P. Perrotta, Richard M. Powers,* Clarence N. Truesdale.

Grade of Junior—Herbert C. Moore, Jr.

The Secretary also announced that E. Sherman Chase would be the speaker at the Joint Meeting of the BSCE and ASCE on May 16, 1956.

*Transfer from Grade of Junior.

President Keane then called upon Ralph W. Horne, Moderator, to conduct the Panel discussion on Mandatory Registration of Engineering and Land Surveyors in Massachusetts. Mr. Horne, before calling on the panelists, presented a chronological review of previous polls on compulsory registration conducted by BSCE and ESNE, of opposition to previous bills filed in the Legislature and of previous efforts on the part of appointed representatives from BSCE, ASCE, MSPE and Architects to get together on a single bill which was mutually satisfactory. The panelists were Oscar S. Bray (ASCE), Thomas R. Camp (BSCE), Edward H. Barry (ASME and Mass. Board of Registration), Charles T. Chave (MSPE, ASME, AIChE) and Frederick S. Bacon, Jr. (AIEE).

Following the panel presentation the meeting was open to discussion from the floor.

Ninety four members and guests attended the dinner, and one hundred twelve (including some students from M.I.T., Northeastern University and Tufts University) attended the meeting.

The meeting adjourned at 9:55 P.M.

ROBERT W. MOIR, *Secretary*

MAY 16, 1956.—A Joint Meeting of the Boston Society of Civil Engineers and the Northeastern Section of the American Society of Civil Engineers and Sanitary Section of BSCE was held this evening at the United Community Service Building, 14 Somerset Street, Boston, Mass. A dinner was served after which the meeting was called to order at 7:35 P.M., by President John G. W. Thomas.

President Thomas stated that the minutes of the previous meeting held on April 23rd would be published in a forthcoming issue of the Journal and that the reading of the minutes would be waived unless there was objection.

President Thomas announced the death of the following members:—

Otis D. Fellows, who was elected a member February 19, 1947 and who died April 13, 1956.

Howard A. Gray, who was elected a member June 10, 1914, and who died April 23, 1956.

Thomas A. Berrigan, who was elected a member May 17, 1948 and who died May 11, 1956.

The Secretary announced the names of applicants for Membership in the BSCE, and that the following had been elected to membership on May 14, 1956:—

Grade of Member—Warren F. Baker, Carl N. Lundgren, John G. Moses, John P. Ottensen, Ralph W. Quigley, Jr., Anders B. Sandquist, Leo G. Sweeney, Apolinaras Treinys, Melvin Wolf.

Grade of Junior—Oliver H. Gilbert, Jr.

President Thomas called upon Mr. John F. Flaherty, Vice Chairman of the Sanitary Section, BSCE to conduct any necessary business for that Section at this time.

President Thomas introduced Edward C. Keane, President of the Northeastern Section, ASCE and asked him to conduct any necessary business for ASCE at this time.

President Thomas then introduced the speaker of the evening, Mr. E. Sherman Chase, Partner, Metcalf & Eddy, who gave a most interesting talk on "Some Experiences of a Sanitary Engineer as an Expert Witness".

A brief discussion followed the talk.

Fifty five members and guests attended the dinner and ninety three members and guests attended the meeting.

The meeting adjourned at 8:45 P.M.

ROBERT W. MOIR, *Secretary*

STRUCTURAL SECTION

APRIL 11, 1956.—A meeting of the Structural Section was held in the Society Rooms on this date.

Prior to the meeting, the executive committee met for the purpose of planning for the coming year.

The meeting was called to order at 7:00 P.M. by Chairman A. L. Delaney, who introduced the speaker, Dr. T. William Lambe of the Massachusetts Institute of Technology.

Dr. Lambe's subject was the design and construction of a dam and reservoir to store oil at the Amuay Venezuela Refinery of the Creole Petroleum Corp. After a brief description of the mechanics by which a damp soil retains oil, Dr. Lambe described the actual construction and operation of the 4,000,000 bbl. reservoir. This reservoir was constructed for slightly less than 20 per cent of the cost of equal tank storage facilities and actual operation has proven the over-all economy of this type of storage. Compared with the large volume and low cost of operation, the losses through evaporation and seepage are insignificant. The paper was illustrated by both black and white and color slides. At the conclusion of the talk, there was a short discussion period.

Thirty members and guests attended this meeting.

The meeting adjourned at 8:15 P.M.

RICHARD W. ALBRECHT, *Clerk*

MAY 9, 1956.—Meeting of the Structural Section was held in the Society Rooms on this date. Prior to the meeting, the executive committee met for the purpose of planning the program for the coming year.

The meeting was called to order at 7:00 P.M. by Chairman A. L. Delaney. After reading the minutes of the previous meeting, the Chairman introduced Mr. Ralph Riddle of the Bethlehem Steel Company.

Mr. Riddle spoke briefly about the films to be shown and mentioned certain outstanding features of construction of the Chesapeake Bay Bridge. The Bethlehem Steel Company provided two films. The first one covered the general methods of steel making and the folling of the steel into plates and shapes. The second film, which was in color, covered the erection of the Chesapeake Bay Bridge from the start of foundation construction through to the completion of the bridge. The films proved to be both interesting and instructive. After the films had been shown, Mr. Riddle answered questions asked by members of the audience.

Twenty-six members and guests attended this meeting.

The meeting adjourned at 8:15 P.M.

RICHARD W. ALBRECHT, *Clerk*

TRANSPORTATION SECTION

JANUARY 25, 1956.—A joint meeting of the Boston Society of Civil Engineers, with the Surveying and Mapping Section, and the Transportation Section of the Society, was held on this date at the United Community Service Building, Somerset Street, Boston.

In the absence of officers of the Transportation Section, President Edwin B. Cobb acted as presiding officer for the Section. The list of nominees for officers of the Transportation Section for the coming year was presented, and President Cobb called for an election of the nominees by a show of hands. The following were elected: Chairman, John Clarkeson; Vice-Chairman, Paul A. Dunkerley; Clerk, Leo F. DeMarsh; Executive Committee, Marcello J. Guarino, Robert A. Snowber and Joseph W. Lavin.

After the election, President Cobb introduced the speaker of the evening, Prof. John T. Howard, Department of City and Regional Planning, M.I.T., who gave an interesting talk on "Planning Transportation for Tomorrow's Cities". A discussion period followed.

Sixty-eight members and guests attended the meeting.

LEO F. DEMARSH, *Clerk*

FEBRUARY 20, 1956.—A joint meeting of the Northeastern Section of the American Society of Civil Engineers and the Transportation Section of the Boston Society of Civil Engineers was held this date at the M.I.T. Faculty Club.

Edward C. Keane, President of the N. E. Section of the A.S.C.E. conducted a short business session and then introduced the speaker of the evening, Col. S. H. Bingham, Retired Executive Director and General Manager of the New York City Transit Authority.

Col. Bingham, who has been Consulting Engineer to many large cities in the United States and foreign countries on mass transportation problems, gave a very interesting talk on "Transportation for Greater Metropolitan Areas". In his talk he mentioned that decentralization, although proposed by many people, is not the answer to the transportation problem and also, additional highways alone do not provide the solution. Col. Bingham said that the transportation problem in our cities today requires a more comprehensive approach which recognizes the interdependence of existing transit systems. He pointed out that due to the increasing use of the automobile, traffic on mass transit systems has steadily declined since about 1947. Col.

Bingham said that unless mass transit is encouraged and aided by all the forces of the government and society to serve the public, there will be a complex breakdown of traffic flow in our large cities.

Col. Bingham described two innovations in the transportation field: the first, the Grand Central-Times Square shuttle and the second, a monorail installation at Houston, Texas.

A discussion period followed the talk. About 150 members and guests attended.

LEO F. DEMARSH, *Clerk*

HYDRAULIC SECTION

MAY 2, 1956.—A joint meeting of the Hydraulic and Sanitary Sections, B.S.C.E. was held in the lecture room of the Hydrodynamics Laboratory at M.I.T. The meeting was convened at 7:00 P.M. by J. C. Lawler, chairman of the Hydraulics Section.

By general consent it was agreed to dispense with the reading of the minutes of the February meeting of the Hydraulics Section.

Chairman Lawler introduced Mr. Darrell A. Root of the Sanitary Section who announced a meeting of the Sanitary Section with the main society on May 16, at which the speaker will be Mr. Sherman Chase. The topic will be, "Some Experiences of a Sanitary Engineer as an Expert Witness".

There being no further business Chairman Lawler introduced the speakers, Professors Arthur T. Ippen and Donald R. F. Harleman of M.I.T. who presented a program under the title, "Density Current Problems in Hydraulic and Sanitary Engineering".

Professor Ippen spoke first, discussing four related problems of density currents and jet mixing. These were:

1. Steady underflow and equilibrium conditions.
2. Diffusion by jets of two liquids.
3. Critical conditions for underflow under sluice gate.
4. Fresh water-salt water conditions in estuaries.

Professor Harleman followed Professor Ippen with a discussion of a laboratory study relating to the problem of controlling thermal density currents at the condenser cooling water intake for the TVA Kingston Steam Power Plant.

There were many questions during the question period, revealing much interest in the subjects of the program. Following the question period, the meeting was adjourned to the laboratory for demonstrations of density current flows and density current mixing processes. The laboratory facilities were generally available for inspection. On completion of the demonstrations the meeting was informally adjourned at 8:45 P.M. Total attendance was 58.

JAMES W. DAILY, *Clerk*

SANITARY SECTION

JUNE 12, 1956.—A special joint meeting of the Sanitary Section and Hydraulics Section was held at the Society Rooms.

The meeting was called to order at 7:00 P.M. by Chairman Darrell A. Root who called upon Chairman Joseph C. Lawler to conduct any business of the Hydraulics Section.

Mr. Root then introduced the speaker of the evening, Mr. Langdon Pearse, Sanitary Engineer of the Metropolitan Sanitary District of Chicago.

Mr. Pearse outlined the events leading up to the founding of the Sanitary District, then showed 75 kodacrome slides and gave a commentary on the Sewerage Treatment Works in Chicago.

This talk was followed by a very interesting discussion period.

Seventy-five were present at the meeting. Twenty-five attended Dinner at Pattens Restaurant.

The meeting was adjourned at 8:45 P.M.

JOHN F. FLAHERTY, *Clerk, Pro-tem*

CONSTRUCTION SECTION

JUNE 6, 1956.—An organizational meeting of the Construction Section of the BSCE was held at the Society Rooms, 715 Tremont Temple, Boston, Mass.

John G. W. Thomas, President, presided. President Thomas opened the meeting at 7:05 P.M., and called for nominations for Officers of the Section.

The following slate of Officers for the Section was presented:—

Chairman	Albert A. Adelman
Vice Chairman	Steven R. Berke
Clerk	Robert J. Hansen
Executive Committee	
	William A. Fisher
	William R. Hooper
	Anthony S. J. Tomasello

It was moved, seconded and voted, that the nominations be closed, following which it was moved, seconded and voted that the Secretary of the BSCE be instructed to cast one ballot for the slate as nominated.

President Thomas then called upon Mr. Warren N. Riker, New England District Manager of the Raymond Concrete Pile Company who introduced Mr. Gordon A. Fletcher, Vice President of the Raymond Concrete Pile Company who spoke on "Construction of the Texas Tower". The talk was followed by a movie showing construction phases.

The meeting, which was attended by 47 members and guests was adjourned at 8:25 P.M.

ROBERT W. MOIR, *Clerk, Pro-tem*

ADDITIONS

Members

Henry H. Baker, Jr., Main Street, Ashburnham, Mass.

Everett D. Burns, 66 Washington Ave., Needham, Mass.

Paul B. Breck, 901 Beacon St., Boston, Mass.

Keistutis P. Devenis, 18 Westgate, Cambridge, Mass.

Raymond F. Fitzmaurice, 58 Boutwell St., Wilmington, Mass.

George L. Fulton, 120 Colonmel Ave., Burlington, Mass.

William H. Hamilton, Y.M.C.A., Huntington Ave, Boston, Mass

Joseph Heney, 22 Norfolk Road, Randolph, Mass.

Bernard S. Johnson, 1073 Beacon St., Brookline, Mass.

Carl N. Lundgren, 54 Hooper St., Quincy, Mass.

Charles L. Miller, 1-263, Mass. Institute of Technology, Cambridge, Mass.

John G. Moses, 5 Bond St., Boston 18, Mass.

Robert B. Nunley, Summer St., Marshfield, Mass.

Alex Ostapenko, 122 Warren St., Apt. 11, Brighton 35, Mass.

John P. Ottesen, Chas. T. Main, Inc., Nallihan Baraji, Turkey.

Gino P. Perrotta, 29 Bennington St., Quincy, Mass.

Richard M. Power, 229 Jackson St., Lawrence, Mass.

Ralph W. Quigley, Metcalf & Eddy, 1300 Statler Bldg., Boston, Mass.

Maurice A. Reidy, Jr., 75 Walker St., Newtonville 60, Mass.

Anders B. Sandquist, 55 Cedar Point Dr., Pocasset, Mass.

Richard J. Supple, 22 Monument Ave., Charlestown 29, Mass.

Frank W. Tomasello, Jr., 51 Hickory Rd., Norwood, Mass.

Apolinas Treinys, 30 Pleasant St., Dorchester, Mass.

Clarence N. Truesdale, 11 Cristofaro St., Wakefield, Mass.

Melvin Wolf, 31 Revere Street, Holbrook, Mass.

Juniors

Oliver Gilbert, Jr., 1-079 Mass. Inst. Technology, Cambridge, Mass.
Charles C. Ladd, 201 Westgate West, Cambridge, Mass.
Carlton H. Moore, Jr., 145 Beaumont Ave., Newtonville, Mass.

DEATHS

Thomas A. Berrigan, May 11, 1956
Otis D. Fellows, April 13, 1956
Howard A. Gray, April 23, 1956

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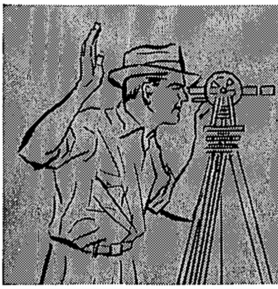

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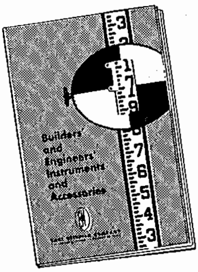

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