Proceedings of the

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ON

SOIL MECHANICS AND FOUNDATION ENGINEERING

FEBRUARY 6 AND 7, 1942

PART II

The University of Texas
COLLEGE OF ENGINEERING AND
BUREAU OF ENGINEERING RESEARCH
Austin, Texas

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PART II

USE OF SOIL TESTS IN THE DESIGN AND CONSTRUCTION OF EL PALMITO DAM

by

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Introduction

The El Palmito Dam is located on the Nazas River, in the State of Durango, in Northern Mexico. Its watershed covers an area of about 7000 square miles on the east slopes of the western Sierras at an altitude between 5000 and 10000 feet. From the dam site the Nazas river flows east for a distance fo 130 miles through mountainous terrain to reach the flat fertile valley of Torreon. Here irrigation has been practiced for the last 50 years using the flood waters, and has been developed to such an extent, that at present the construction of this dam was imperative in order to stabilize the finnancial status of this region.

The dam at present under construction will have a reservoir capacity of 2,400,000 acre-ft, which is about 2.3 times the average yearly run-off. Records show, however, that occasionally the reservoir can be filled in a single year. A power plant with 56,000 hp. installed capacity will be built at the outlet of the tunnels to furnish power to the irrigation district where more than 600 deep well pumps are at present in operation. There is in addition a large market for power for mining and other industries.

Dam Design

The site is located in a formation of volcanic rhyolitetuffs. In the river bed this formation has a depth of about 100 feet, and is underlaid by successive layers of sedimentary volcanic ashes and rhyolites. The two banks are made up of alternate layers of soft decomposed tuffs, rhyolites and rhyolite-tuffs, which give a somewhat less stable formation.

Taking into consideration this geological formation, a masonry structure was discarded, having in mind that this dam would have to be at least 300 feet high in order to store sufficient water. A rock-fill dam was planned with an impervious clay apron upstream, similar to San Gabriel dam in Southern California, which at the time was under construction. This design was soon abandoned after a more careful investigation discovered a rather broken condition of the formation on both banks, with very soft decomposed tuffs in places. This condition indicated the advisability of widening the clay core instead of going into elaborate and more expensive bank protection. This resulted in changing the original rock-fill dam to an earth dam. The clay core to have an upstream slope of 2:1 and downstream of 0.5:1.

At the time construction was started, a more detailed survey was made of all the materials available, and taking into consideration the relative cost of placing clay and rock in the dam, again a change in the design was made, increasing the width of the clay core. This core as being constructed will have a 2:1 upstream slope, and 1:1 downstream slope, as shown in Fig. 1. These changes in design were possible as the construction proceeded, and without negociating with contractors as the dam is being built by direct administration. The berm 164 feet wide at the upstream slope is intended to provide against sliding of the rock fill when the clay core of the upstream cofferdam becomes saturated.

The foregoing illustrates the reasons necessitating the revised designs which were developed as more complete information was obtained pertaining to the geological characteristics of the site and the amount of and characteristics of the available materials. Data obtained from the laboratory soil investigations contributed materially in determining the final design which we consider to be the most practical and economical.

Simultaneously with this change, additional test hole drilling on both river bed and banks disclosed a very broken condition of the rock formation; water was invariably lost in the holes on both banks and showed up in appreciable amounts in those in the river bed. The rock is moderately hard but light in weight and porous. In places the rock forms sound blocks but is generally very fractured, with many wide crevasses loosely filled. With these indications the number of cutoffs was increased to five as shown in Fig. 1. Later as these cutoffs were drilled for grouting their necessity was well substantiated by the large amount of grout consumed. Along the main cutoff wall the grout holes were required as close as one foot centers, a number of these holes took over 1000 sacks of cement each in order to grout them to refusal.

All grouting was done in stages; this method being known generally as progressive grouting. Compressed air "Union Iron Works" grout mixer-injectors were used, maximum pressure 100 pounds per sq.in., neat cement grout. The concrete cutoff-walls extend 15 feet into rock, and are drilled for grouting 80 feet deep at cutoffs Nos. 1 and 5; 100 feet at Nos. 2 and 4; and 130 feet at No. 3. For vertical hole drilling in the river bed the most successful machine was found to be the well-drill with 4 and 6 inch bits. Besides being the most economical to operate on these rocks, the large size hole also was more effective in taking the grout. On the banks, for inclined drilling, the calyx, diamond drill, and pneumatic machines were tried, obtaining the best results with the pneumatic percussion machine equipped with independent air rotation.

To date there has been drilled approximately 250,000 lineal feet of deep holes for grouting, with an average consumption of about half a half a sack of cement per lineal foot.

The spillway will be located in a saddle one mile from the dam, constructed with a concrete fixed crest, for a capacity of 280,000 sec-ft.

Borrow Pits

Within a two mile radius upstream from the dam all the material required for the impervios core can be obtained. On both sides of the river there are flat terraces of alluvial origin, having an average soil depth of about 12 feet, which makes an ideal location for the borrow pits. The material from these terraces varies from silt and fine sande loams close to the river to more clayey and heavier loams on higher ground. Previous to beginning construction a general soil survey was made to determine the amount of material available and its classification. Sample pits 3 x 5 feet square were dug 330 feet apart in checkerboard pattern, and where necessary, intermediate additional pits were dug. Representative samples were obtained from each pit and classified according to their mechanical analyses. The Proctor procedure was followed to study compaction vs. moisture content. The regular Proctor cylinder was used dropping a 5.5 lb. rammer 20 times from a height of 18 inches.

Also many permeameter tests at constant head were made to cover the different varieties of soils. This enabled us to select from all the materials available the best from the standpoint of weight, impermeability and distance from the dam. In general all the materials available in this region are rather light in weight. The following tables will illustrate:

Optimum compaction and permeability:

Classification	Abs. density.	Dry density- lbs. per cu.ft.	Permeability "K" ft. per year (Proctor Permeameters)
Sandy Loam	2.50	104	0.10
Silty Loam	2.50	103	0.04
Sandy-clay loam	2.55	116	0.06
Clay loam	2.50	108	0.02

Average mechanical analyses:

Classification	Sand %	Silt %	Clay %
Sandy loam	68	22	10
Silty loam	35	55	10
Sandy-clay loam	60	20	20
Clay Loam	42	30	28

The above analyses are shown in graphical form in Fig. 2.

Enough material of the sandy-clay and clay loams are available to complete the impervious core so that the more sandy and silty loams, which are not so satisfactory, can be discarded.

At the borrow pits all these soils are underlaid by gravel and sand bank deposits. This condition has been found very advantageoious in obtaining a gravel-earth mixture for the fill, giving a better excavation face for the shovels to work and reducing the hauling distance to the dam. It will be shown later in this article how the fill has been improved with the addition of gravel, and the smooth roller equipment perfected for this purpose.

Clay on Bedrock

The overburden in the river bottom was sand and gravel to a maximum depth of about 30 feet. This was removed for the complete clay core zone, and the bedrock cleared and surface cleaned with a compressed air jet previous to placing any clay. Fig. 3 shows cleared bedrock.

To seal all superficial cracks, inclined 15 feet holes were drilled at intervals of 15 feet for surface grouting. There was however a zone upstream, within the clay core, where the rock was so badly broken and so much water filtering through, that more holes for surface grouting were required. Notwithstanding, in places the rock was so completely cracked and decomposed tuffs so soft, that although the main filtrations had been sealed, there still remained a weeping condition which made difficult the placing of the clay. Undoubtedly at these places greater filtrations are to be expected when the reservoir is filled. To insure a good contact and an impervious clay fill over these wet and soft places, it was resorted to a stabilized soil as used in highway work, mixing clay with portland cement. Tests were made in the laboratory using from 4 to 10 percent cement in order to select the proper mix. Compression and permeability results are shown in the following table:

Cement %	7-day Comp. Test. lb. per sq. in.	28-day Comp. Test lb. per sq. in.	Permeability "K" ft. per year
0	9	-	0.21
4	121	146	0.02
6	117	132	-
8	145	152	-
10	153	169	-

For compression tests the stabilized soils were compacted in regular concrete cylinder forms 6 x 12 inches. To experiment further the lumps of the broken cylinder with 4 percent cement were immersed in water and held together indefinitely without crumbling. This shows how much a soil can be improved by the addition of a small percentage of cement, where used for special conditions in earth dam construction.

An adherence test was also made to compare with straight clay. This test was made compacting the clay sample in a 6 x 12 inch cylinder form over a porous percast cement block, and tested after 7 days with a simple homemade apparatus. Results were as follows:

Cement %	Dry density lb. per cu. ft.	Adherence lb. per sq. in.
0	107	1.5
2	108	6.6
4	103	23.0

The 4 percent mix was selected as being the most economical within acceptable requirements. This process was considered very effective in covering porous zones, thus affording a filtering medium, which, when saturated would be less susceptible to piping.

To further check the permeability of this stabilized soil, a field test was made while building the cofferdam. As shown in Fig. 4, two 4 inch by 20 feet long perforated pipes were laid horizontally in trenches cut in the bedrock and filled around with gravel. Ordinary clay was compacted over one of the pipes, and a layer of clay with 4 percent cement 3 feet thick over the other. The clay fill was continued up and two months later, when the cofferdam was completed, the pipes were tested with water under 45-foot head from barrels placed at the crown of the cofferdam. After two weeks of continuous testing filtrations became regular, and showed the plain clay to be twice as permeable as the clay with 4 percent cement.

Hand Tamping

With such an uneven bedrock foundation as shown in Fig. 3, a large amount of hand tamping was necessary. Pneumatic tampers were used throughout the work, clay was placed in layers 2 inches thick, running over the tample four times. To check the work of tampers, frequent samples were tested for compaction. Very good work can be done with tampers as shown from the following results obtained as an average of many samples:

Gravel in sample %	Dry Density soil along lb. per cu.ft.	Dry density soil & gravel lb. per cu.ft.	Compaction %
0-10	109	112	100
10-20	107	112	107
20-30	112	118	101

All material larger than 1/4 inch is considered as gravel. Gravel is in percent of dry density, by weight.

Hand tamping is rather slow, and in many places where work had to be done very rapidly, a cheap concrete was used to even up the bedrock foundation preparatory to running over the sheepsfoot roller. This concrete was made with 80 percent fairly clean bank-run gravel, 20 percent clay, and 2-1/2 sacks of cement per cu. yd. The setting of this mixture was so quick that it would resist a 15-ton truck without sinking, six hours after being placed, thus permitting to follow up quickly with the sheepsfoot roller. This procedure while being a little more expensive, may prove more economical in many tight places or where labor is more expensive.

Moisture Application

The borrow pits are at a low elevation with reference to the river, and it was possible through the existance of an old irrigation ditch to irrigate the materials by gravity. This procedure not only being more economical but has proved to be effective for a good soil compaction. The average of many compaction samples of materials irrigated in the pits has shown a difference of about 5 percent higher in dry density over those sprinkled on the fill.

After the pits are innundated it usually requires from four to six weeks for the moisture to reach the optimum content required, Sometimes on hot or windy days a light sprinkling on the fill with a tank truck is necessary to compensate for moisture lost. Besides the increase in dry density, this system has proven very accomodating for construction work, as tank trucks and water hoses are eliminated. Clay sticking to the shovel dippers at the pits has been very much eliminated by mixing a certain percentage of gravel with the clay.

Gravel in Soils:

As previously explained, all the soils at the pits are underlaid by a deep bank of gravel. See figure 5. The proper exploitation of the pits required a bank excavation as deep as possible in order to obtain most of the materials within a short distance from the dam. The use of a greater depth of material gave a higher working face for the 2-1/2 cu. yd. diesel shovels, which was very advantageous as mixing materials was simplified and a larger percentage could be used.

Immediately after starting the fill a certain percentage of gravel was used in the soil in the downstream section of the dam. Gravel was not all uniform nor small in size and it caused much trouble with the sheepsfoot rollers, breaking the cleaners and wedging in between the feet. Since the impervious core is very wide, experiments were made in this lower section using a smooth roller weighing 15 tons. This roller was made from a shell of a used boiler 5 feet in diameter, filled with concrete, and drawn with a tractor. Results were very satisfying, showing dry densities obtained as good or slightly better for the soil (excluding gravel) than those obtained with the sheepsfoot roller. The gravel-soil combination of course gave higher weights. The smooth roller in these experiments could go into higher percentages of gravel without any difficulty. As to the smooth surface which the smooth rollers usually leave between compacted layers. it appears that larger pieces of gravel puncture this plane and knead the two layers together.

Permeameter tests were made in the laboratory for soils with gravel but they were not satisfactory, even with the use of a larger permeameter. Then practical tests were made in the field in order to obtain a comparison of permeability with fills with less gravel rolled with sheepsfoot roller. For the purpose of this tests the clay core was divided in three zones: Zone "A" upstream rolled with sheepsfoot roller in layers of 6 inches with 12 passes; Zone "B", middle section, rolled in layers of same thickness with 8 passes of smooth roller and two additional with sheepsfoot roller to scarify; Zone "C", downstream, rolled with 8 passes of smooth roller, without scarifying, and same thickness of layers. Permeability tests were made using an empty 55-gal. gasoline drum with the bottom cutoff and set in the fill as shown in Fig. 6. The top of the drum was left covered to prevent evaporation. The excavation all around the drum was well packed with cement mortar in order to limit the filtration strictly to the bottom. Results obtained are given in the following tables:

Permeability tests:

Tests:	Zone "A"		Zone "B"		Zone "C"			
	1	2	3	4	5	6	7	8
Clay %	2.4	2.4	3.3	3.4	3.9	3.0	2.4	3.3
Silt %	40.6	50.6	33.7	39.8	40.1	43.0	44.6	45.7
Sand %	57.0	47.0	63.0	57.0	56.1	54.0	53.0	51.0
Soil Dry Dens. lb. per cu. ft.	110	110	116	112	119	114	107	114
Gravel %	30.5	38.7	43.3	51.5	48.7	35.8	38.5	41.4
Filtration (")	1.9	2.4	0.5	6.1	3.8	1.4	5.2	1.9

(") Inches per day which water surface lowered.

Compaction results:

Sheepsfoot Roller - 12 passes				Smooth Roller - 8 passes			
Gravel	Dry Dens. - lb. per cu. ft.		Compact. %	Gravel %	Dry Density lb. per cu. ft.		Compact %
	Soil	Soil & Grav.		Soil	Soil	Soil & Grav.	
7.8	106	109	100.6	5.0	101	102	100.7
15.8	103	109	98.7	13.4	105	108	99.3
25.1	104	113	102.6	26.2	111	120	103.5
32.9	110	119	103.4	35.4	113	122	105.7
42.5	105	120	102.2	46.3	112	125	104.9
Avg.	105.6	114.0	101.5	Avg.	108.4	115.4	102.8

The above results can be summarized as follows:

(1) With the use of gravel in amounts up to about 40 percent, the combined soil and gravel dry density increases an average of 8 percent compared with soil alone, for both smooth and sheepsfoot rollers.

(2) An increase of soil compaction of about 2 percent is obtained with the use of a sheepsfoot roller and 4 percent for smooth roller, when amounts of gravel were increased from 10 to 40 percent. This compaction based on soil alone, gravel not included.

(3) With the use of gravel in quantities up to about 40 percent mixed with the soils available at El Palmito, there does not appear to be any increase in permeability.

The limit of gravel for good compaction work seems to be about 40 percent. It also appears that with the use of gravel the optimum moisture content is lowered. From the standpoint of construction the smooth roller has other advantages: It is easier to pull, requiring less power, and considering that fewer passes are required to do the work, the cost of compaction may be reduced as much as one third; also the rolled surface as left by the smooth roller facilitates the movement of the hauling equipment. This smooth surface is of great importance in the rainy season since water penetrates much less, and almost immediately after the rain has passed, work can be resumed.

It would seem natural that in compacting with a smooth roller the upper half of the layer would take a higher degree of compaction that the lower; nevertheless this does not appear to be the case as shown by the following results from sample tests:

	Soil Density Soil alone	Lb. per cu.ft. Soil & Gravel	Gravel %	Compaction %
Upper half	110	110	4.4	102.9
Lower half	110	111	4.2	102.1

It is not the intention to accept the results of the above tests in order to arrive at the conclusion that smooth rollers are preferable to sheepsfoot rollers. These experiments are based only on the materials available at this location and with the purpose of using higher amounts of gravel, and although based on a large number of experiments, these conclusions may not be final if applied to rolling equipment of other types and weights than those employed here. We were confronted with the proper employment of the materials as found in the borrow pits to be used in a wide clay core. From the standpoint of engineering and construction, the use of gravel was of the greatest importance to increase weight and friction, and to lower unit cost per cubic yard of fill, keeping other factors such as permeability within acceptable limits. For this purpose a special heavy smooth roller was designed with two drums 5 ft. 10 in. in diameter and 5 ft. long, with a 1-1/2 in. wearing plate, and filled with water, sand, and gravel. See figure 7. To make the rolling equipment as standard as possible the frame for this roller was made exactly like that of the sheepsfoot roolers that we are using, which are the Bureau of Reclamation type with two drums 5 ft. in diameter by 5 ft. long. See figure 8. This sheepsfoot roller with water and gravel weighs 19.5 tons, and the smooth roller with same ballast weighs 23 tons.

As the placing of the impervious core proceeded and considering information obtained from afore mentioned experiments, the clay core was approximately divided into equal parts for the purpose of distributing the materials from the borrow pits. In the upstream zone the sandy-clay loam is used with a gravel content not to exceed 20 percent and rolled with 8 passes of the smooth roller and 2 passes with the sheepsfoot roller to scarify between layers. This material occurs at the pits in faces up to 27 ft. in height. When this height of face in the borrow pit does not provide a material with 20 percent of gravel, the material as encountered is excavated and used in the upstream zone, but in this case compaction is done with the sheepsfoot roller. In the downstream zone the clay soil is used with a gravel content of approximately 40 percent and rolled with 8 passes of the smooth roller without scarifying between layers. This material occurs in the pits in shallow faces never over 12 ft. in height, thus it is always possible to add gravel.

Our experiments with smooth rollers were concerned chiefly in regard to increasing the amount of gravel in the fill, thereby raising the weight without appreciable loss in impermeability. These experiments, although not very extensive, have been mentioned here with the idea of contributing to a better knowledge of the use of smooth rollers in relation to present day practice of soil mechanics.

Soil Tests for Control

Samples from test pits in the fill are taken daily to check the moisture content and compaction obtained. These tests are made following the usual Proctor procedure to compare compaction obtained in the fill against optimum as obtained in the Proctor cylinder. For future reference a record is kept of the location of each sample in the fill and all its physical characteristics. Also rapid moisture and gravel percent determinations are daily made as desired by the field inspectors for their guidance. All the data obtained from these tests in the laboratory is daily passed to the inspector to keep them informed. Permeameter tests of the different materials as obtained from the borrow pits are also made at regular intervals to ascertain that material used conforms with the specifications.

Fill Settlement

As the fill is going up a telescopic pipe is being left to determine its settlement. This pipe is made out of successive lengths of pipe, one inside the other, 2 and 1-1/2 inches in diameter by 5 feet long. The Reclamation Service sounding apparatus is used, and elevations are taken at the lower end of each 1-1/2" pipe where the salient fingers of the apparatus catch as it is raised. Fig. 9 shows a graph made from the records up to date. It should be noted that the fill has not been carried very fast, and yet the graph shows increasing settlements as construction is proceeding. No settlement has been observed in the bedrock.

Rock-Fill

Rocks available in the vicinity of the dam are of volcanic origin, as explained above, mostly rhyolite-tuffs, with an average density of 2.0. This rock if quarried and dumped in the fill would have at least 35% voids, giving an apparent density of only 1.3. To increase this weight river gravel is added in the approximate proportion of 60 rock to 40 gravel. Materials are dumped over the slope in lifts of about 24 feet high and sluiced down with water under pressure from a hose. See Fig. 10. This makes the rock roll down and carry the gravel which fill the voids. Besides the mechanical action of filling the voids with gravel, increased settlement of the fill is obtained through the lubrication action of the water. Water is added at the ratio of about 2 cubic yards per one of fill.

Characteristics of Materials used in rock fill:

	Abs. Density	Water absorption
Sand	2.45	2%
Gravel	2.43	2%
Rhyolite-Tuffs	2.00	14%

Average sample from rock-fill:

Abs. Density	2.2
App. density	1.8
% Voids	23
% Sand	26
% Gravel	31
% Rhyolite-Tuffs	43

Gravel as used in this process, besides reducing the voids, gives also an increase in volume, which makes the rock-fill cheaper than if made of rock alone on account of the high cost of quarried rock. This fill is non-sliding and heavier, and being tighter it affords a good protection to the clay core against wave action and erosion from drainage.

Field samples to determine apparent density are obtained by digging pits of about 30 cu. ft. Volumes are determined by means of dry sand whose volumetric weight is known.

The upstream slope of the dam will be finished with a layer 10 feet thick of large uniform rock carefully placed so as to give an even surface. On the downstream slope rip-rap will be placed by hand.

Inspection Gallery Under Dam

This gallery is excavated through rock across the canyon at a depth of 30 feet below the bedrock surface and is carried up through the original rock on both banks. This is perhaps a unique feature for an earth dam, but in this case it was considered necessary taking into consideration the height of the dam and the broked condition of rock in the river bed and banks. It will permit at all times to do additional grouting if necessary and affords a free access for inspection. Several vertical drill holes will be left along the floor of the gallery where drainage and water pressure can be measured.

Since this gallery is not intended for drainage, its outlet is at an elevation 100 feet higher than its floor. At this elevation an entrance connection is made from the operating tunnels. Arrangements are made for pumping out through this same entrance any drainage which may accumulate, this procedure being necessary when inspection is desired. Only in very few places will it be necessary to line the gallery with concrete.

Construction Process

The regimen of the Nazas river is very erratic, and floods as large as 140,000 sec-ft. often occur. Furthermore, floods may be expected twice a year; during the rainy season from July to September and in winter due to run-off caused by melting snow in the mountains. These last arriving very suddenly when snow over the high and steep Sierras melts at the arrival of the warm winds from the Gulf. The canyon is rather narrow from the view point of an earth dam, and a channel for diversion was excavated along the left bank. This channel permited the diversion of flood waters while construction of the fill was in progress on the right side. This step is shown in Fig. 1 as 1st stage. At the same time excavation was advanacing on three 20-feet circular tunnels in the left bank, to be used for diversion when the channel would be closed, and later for operation purposes. The fill at the right side was built 150 feet high, and at present, with all three tunnels finished and lined, the diversion channel is being filled. This step is shown in Fig. 1 as 2nd stage. Third stage will follow immediately afterwards as shown in Fig. 1. In the closing of this channel there exists such a flood hazard, that a scheme was devised whereby, in the event a large flood might arrive during construction, the fill could be saved, and the danger of flooding the towns and cultivated lands below, averted; it might well be called an "emergency spillway"; and is built in the diversion channel by lining with concrete the downstream slope of the clay fill as it is being carried up. At 25 foot intervals this lining is run horizontally over the fill in the form of a berm and protected upstream with a cutoff wall, thus making a regular spillway crest. In case of a flood, the three tunnels will take care of the water up to a certain extent, beyond which, water will pass over the emergency spillway, cutting out some clay until its proper erosion bed is formed. For this fill a practical investigation was made to determine the kind of material which should be selected to withstand best the erosion in case of a flood. Test embankments were rolled with different kinds of materials, and water run through channels of similar cross section which were excavated in each class of material. The tests were run fro 45 minutes, and the best material to withstand erosion was selected. This was found to be the one containing the most clay.

Total volume of the impervious fill is 4,000,000 cu. yds. and that of the rock-fill 3,500,000 cu. yds.

Equipment

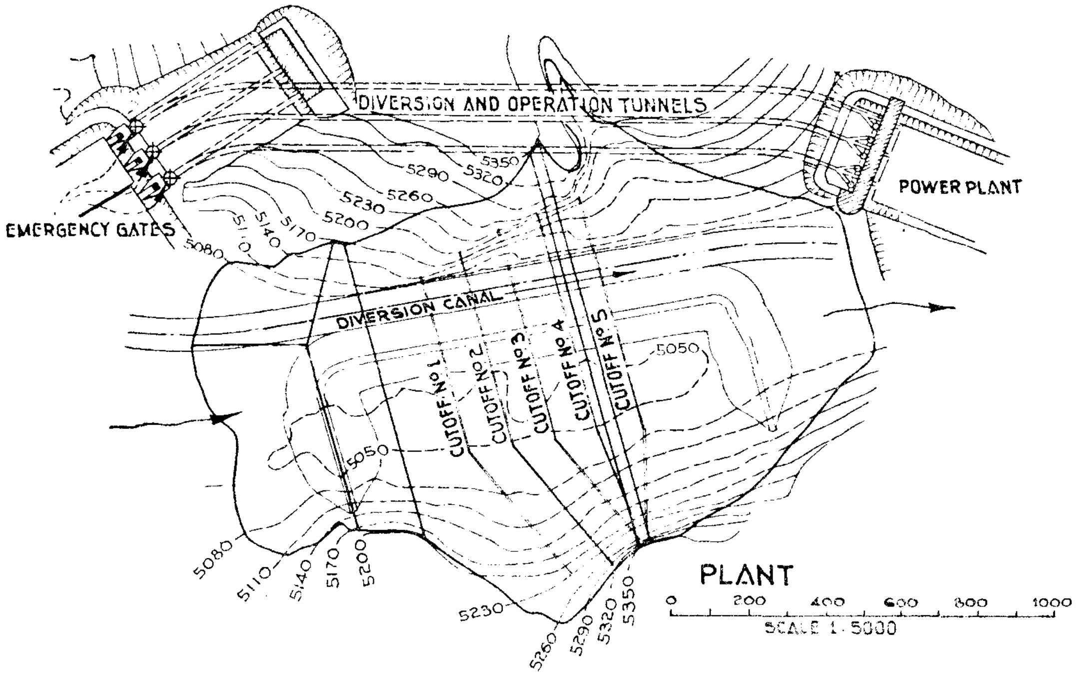
Five 2-1/2 yds. diesel shovels are used for excavation in the borrow pits and quarries. For other work four 1-3/4 and one 3/8 yd. diesel shovels are available. Some dragline and clam shell equipments are available to interchange with these shovels, except for the 3/8 yd. Trucks for hauling clay and rock are 4 wheel with duals on rear wheels, 8 cu. yd. scoop type dump bodies, diesel motor. For concrete and minor work, 3 and 5-ton capacity gasoline trucks are in use. Well-drills mounted on tracks with 6-in. bits are used in the rock quarries. Power and compressed air is obtained from a central power plant with three diesel-electric 365 kv-a generators and three diesel air compressors

1000 cu. ft. per min. each. There are also two electric-driven air compressors with a combined capacity of 1500 cu. ft. per min. Concrete in tunnels and major structures is handled with two Rex single 7-in. concrete pumps.

Conclusions:

The purpose and endeavor of this paper is to show the importance of a field laboratory. Prior to beginning construction work an investigation of the materials available had already been made in the Central Laboratory. When work was started a more thorough investigation of these materials was made and the experience obtained in the field laboratory enabled the men who in the future were to handle these materials, to get a first hand knowledge of them. It has been possible through the aid of these laboratory investigations to improve in the design of the dam, and throughout the construction period, to devise methods for the best use of the materials. Also, and not less important, has been the value of all soil tests to follow up the construction and as a check on completed work. The laboratory at this dam is a field laboratory provided with all necessary equipment for testing soils, rocks, aggregates, and concrete.

The construction of this dam was planned and is under the supervision of the National Irrigation Commission with central offices in Mexico City. The dam is part of a vast program of irrigation conducted by the Federal Government. Officers for this Commission are Engrs. Secretary of Agriculture Marte R. Gomez, president, Adolfo Orive Alba, executive director, and Juan Mas, secretary. At the Central Office Andrew Weiss is consulting engineer, Antonio Coria, technical director, Cesar Jimenez, chief of designs, and Manuel Bus,Tamante, chief of construction. At the field H. V. R. Thorne is general superintendent, and the writer, resident engineer.



PLANT

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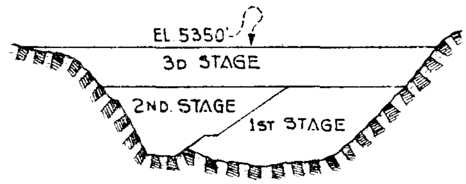

ELEVATION OF DAM

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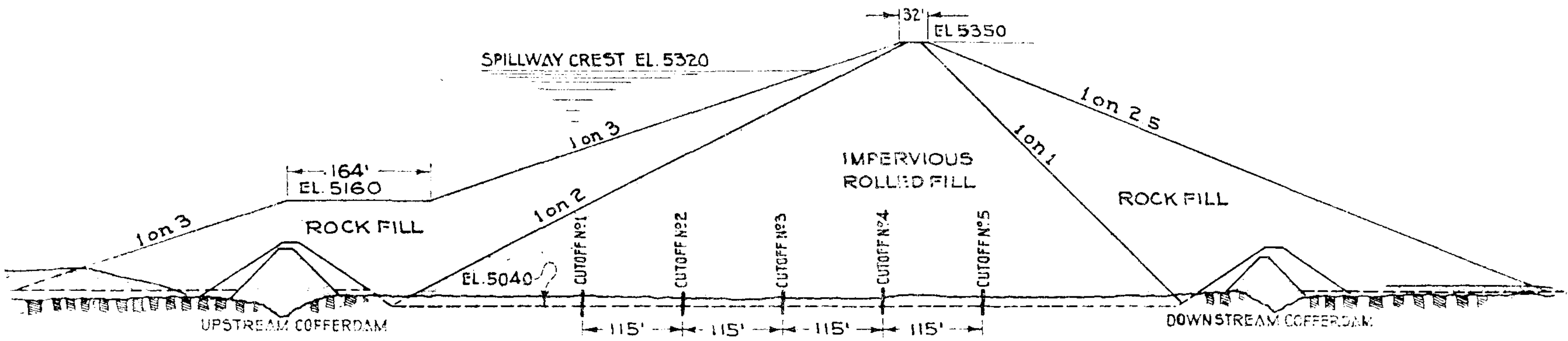

TYPICAL SECTION - EL PALMITO DAM

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Fig.-1

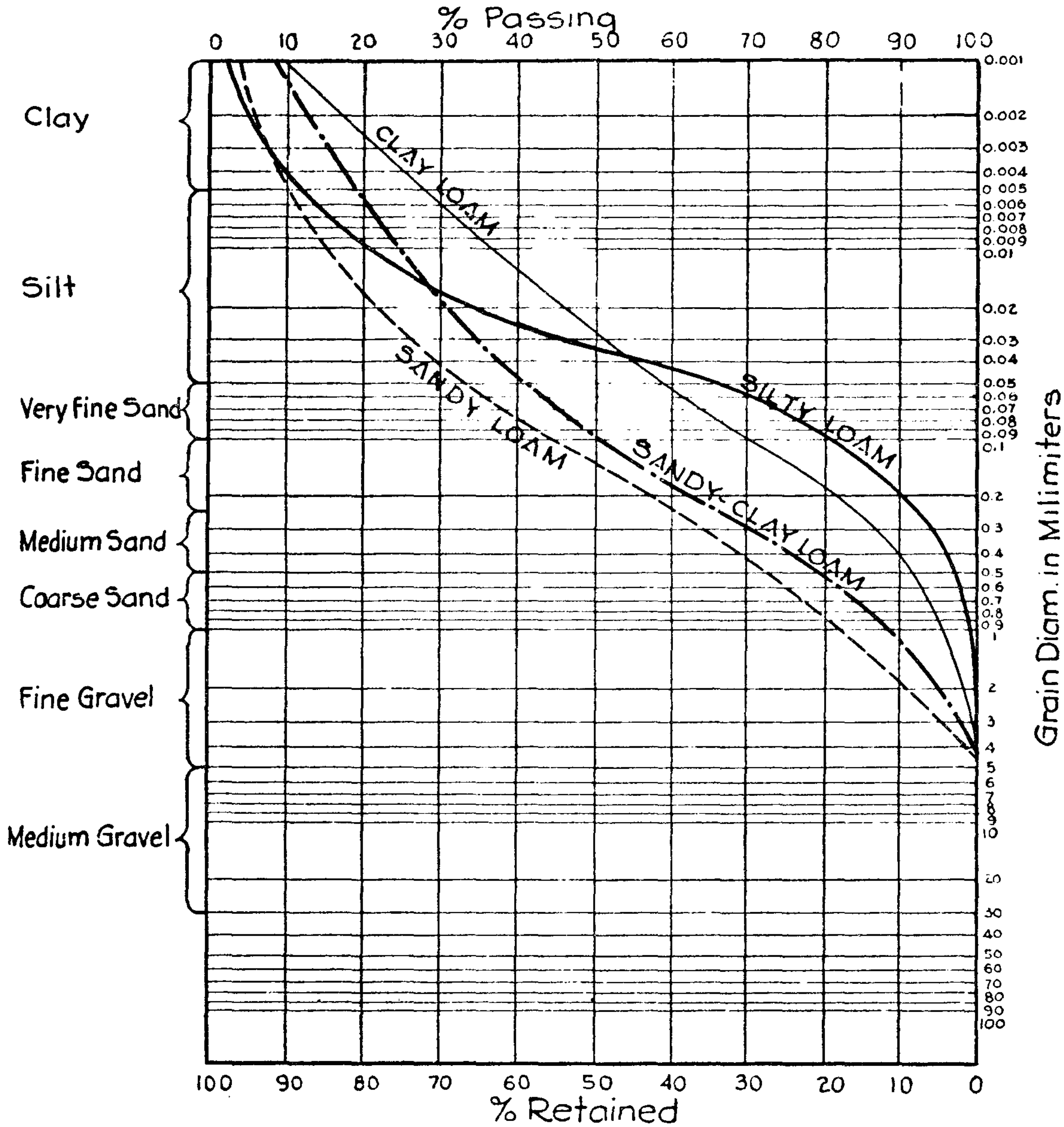
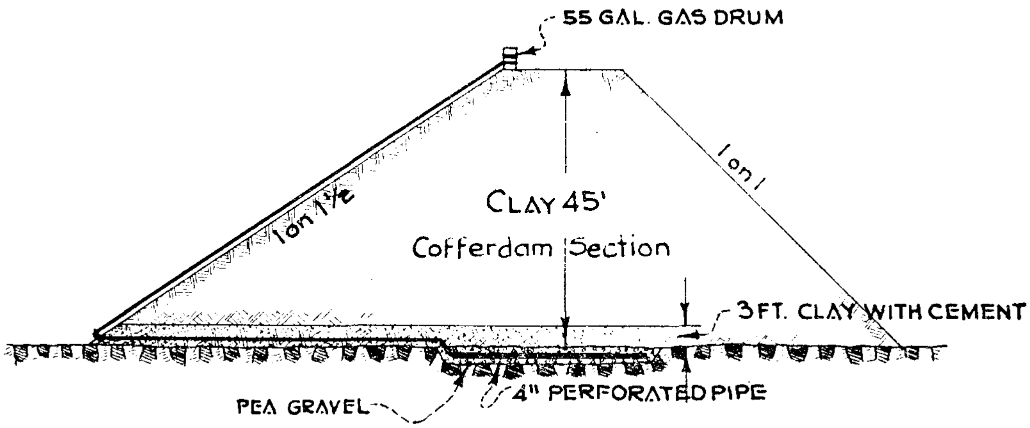


Fig. 2

MECHANICAL ANALYSIS
EL PALMITO SOILS



FILTRATION TEST IN
CLAY FILL WITH CEMENT

Fig. 4

Fig.3

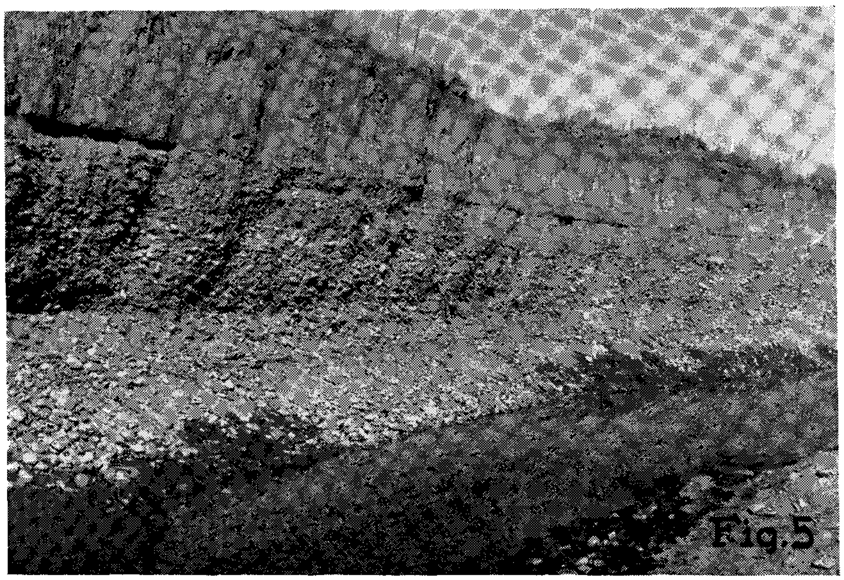
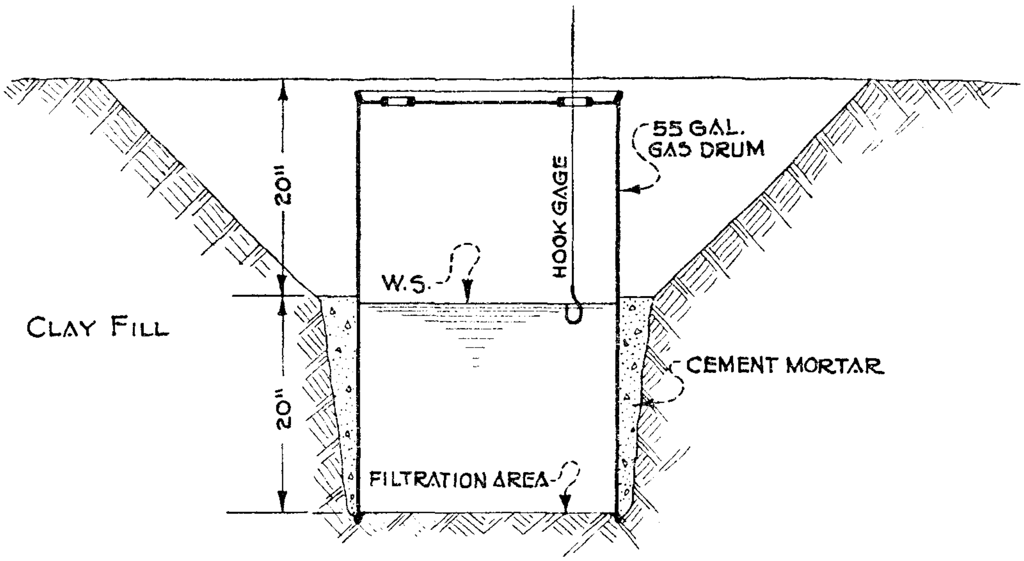


Fig.5



FILTRATION TEST IN FILL

Fig. 6

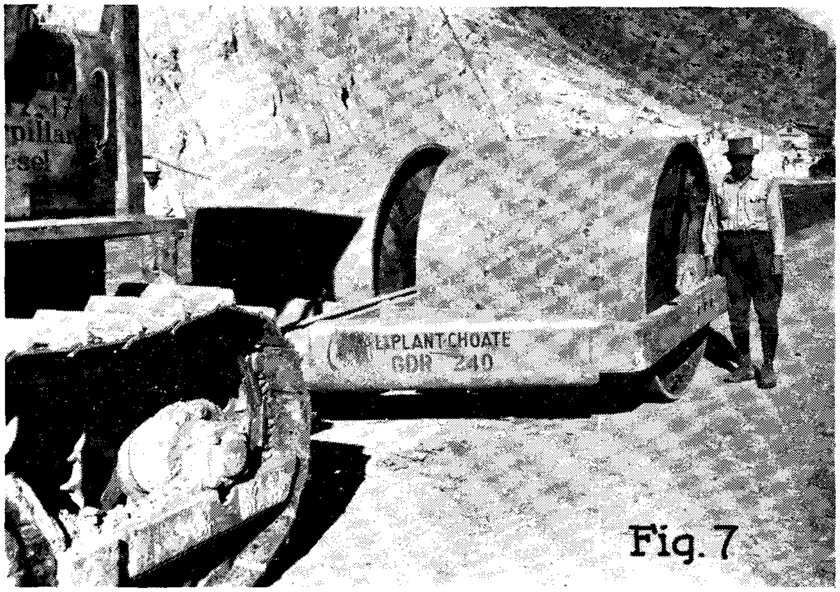


Fig. 7

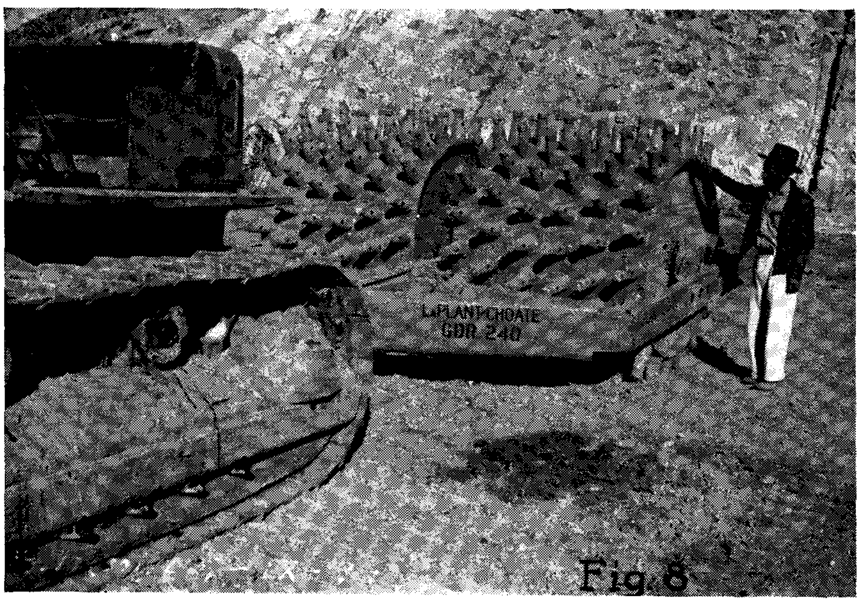
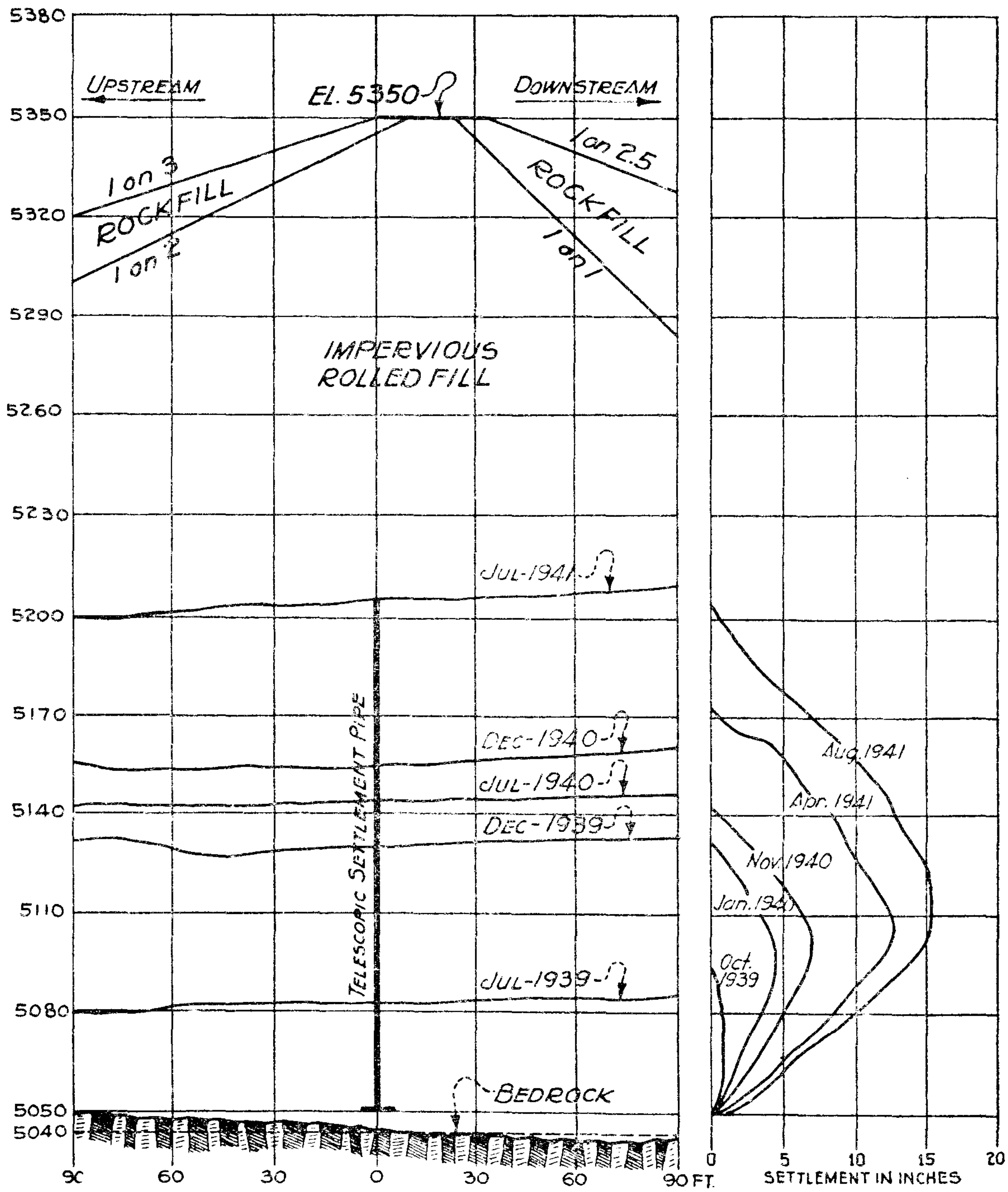


Fig. 8



CROSS SECTION OF DAM

Fig. 9

Fig. 11
Inside of one of three diversion tunnels 20 ft. in diam, lined with concrete.

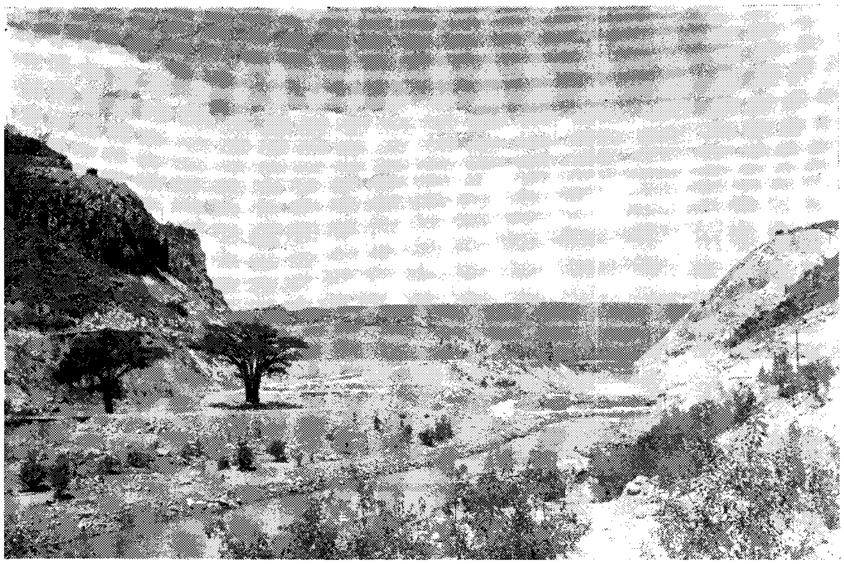


Fig. 12
View of the Palmito Canyon, looking upstream. At left clay fill and at right diversion channel.

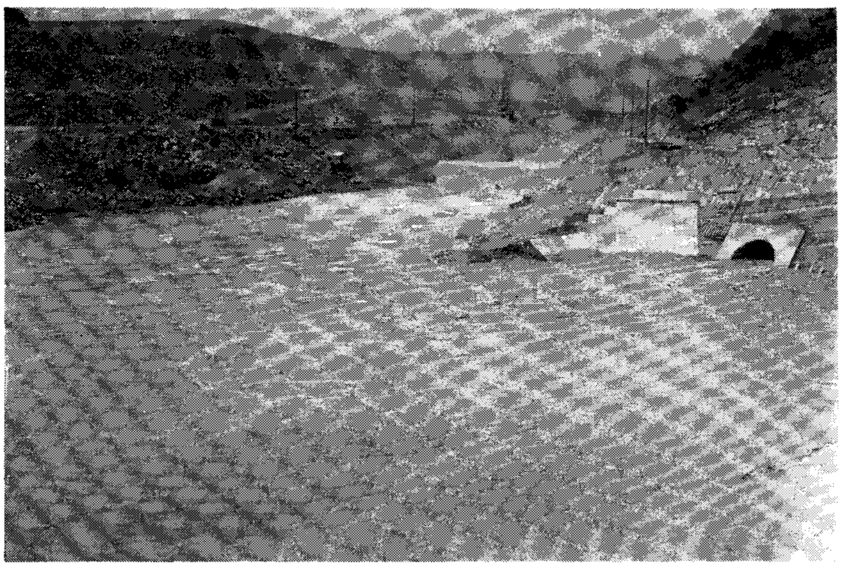


Fig. 13
View looking upstream. At left clay fill. Center diversion channel and tunnel at right.

PLANNING AND INTERPRETING SOIL TESTS

by

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Since the beginning of Soil Mechanics, engineers have endeavored to improve planning and interpretation of soil tests, so that all could have the benefit of singular and unified methods of arriving at logical solutions of soils problems.

Many engineers have become so accustomed to the use of certain practices involving the principles of Soil Mechanics, that they accept what has been done as realism. But if they were to view numerous soils theories with a hypercritical eye, they would realize that what they had been accepting as the real think, is perhaps only an imitation.

A truly scientific theory is one which is vindicated by practice. Laws of science are rules or recipes for the conduct of practical affairs. There are two ways in which they fulfill their use. One is to provide us with a large mass of data in condensed form. The other is to explore singular features of seemingly distinct occurrences. The first step generally leads to the second, but may not be absolutely necessary. Research engineers, generally follow one of two methods of investigation. One operates on a trial and error basis, collecting relevant information piece by piece. The other follows a hunch, but abandons it at once, when it comes into contact with observed facts. But there is no one method. The unity of the principle resides in the nature of the result, and the unity of the theory with practice.

In order to fully evaluate the principles of planning a soil testing program, it is necessary to make use of accumulated data and to take cognizance of what reactionary aspects of the materials are already known. With this in mind, one can plan a veries of tests that will include; (1) the effect of disturbance, (2) geological considerations, (3) type of test best suited to the materials being investigated, (4) condition boundaries of the test, (5) proper technique of testing.

EFFECT OF DISTURBANCE: Engineers have long recognized that specialized laboratory tests performed on disturbed or partially disturbed soils are not in themselves of great value. Fair interpretation of results is difficult, consequently, the data are usually relegated to the "files", where they well belong. Fortunately for the soils engineer, only two major tests are necessary to determine the effective structural characteristics of a soil. Once the consolidation and strength characteristics of a soil are completely known, all other tests are incidental and become mere pawns of the former. While the effect of disturbance is pronounced on the determination of the consolidation characteristics of a soil, it is not a criterion that, except for economical engineering, will vitally affect a consolidation study, which because of the number of theoretical assumptions, drainage and boundary conditions, is in itself only a fair estimate of anticipated settlement. This is one of the reasons why computed settlements are usually greater than actual settlements, discounting the not too clear aspects of secondary consolidation. On the other hand, the effect of disturbance on the strength of soils is profound. Soils under vertical stress undergo a gradual rearrangement of internal structure as the stress is slowly increased, thereby resulting in volume change. Assuming a somewhat flexible, confining restraint such as would be afforded a loaded element in a soil mass, one notes that as the vertical stress is increased, we have the anomaly of the same forces affecting consolidation that are at the same time tending to produce a shear failure. Hence, a semi-confined, stressed soil is only in a transitory stage, that as pressure is increased, rapidly progresses into fully developed shear and when no more strain can be taken, suddenly ruptures or undergoes metamorphosis and fails in plastic flow. Disturbance in sampling or testing merely places a soil well along in that vague but nevertheless transcendental stage in which less stress is required to produce failure. Tests that are used to measure the strength of soils will therefore regardless of tyep or testing technique show a corresponding decrease is strength if a material is disturbed during or after sampling.

While selection of the test most applicable to the determination of the shearing strength depends greatly upon the material type, the determination of the effect of disturbance is allied to the test type regardless of material class.

Evaluation of the effect of disturbance by means of the direct shear test is questionable because volume changes are hard to measure and the data obtained are difficult to apply. The simple compression test offers more possibilities because one can plot a definite stress-strain curve. The characteristic shape and slope of the stress-strain curve is often used as a criterion for evaluation of possible disturbance, but it is quite probable that this holds true for only the sensibly plastic materials. Unfortunately, the ordinary range of soils runs the gamut of plastic to brittle materials, hence it may be that internal particle arrangement, pseudo structures set up by cyclic desiccation, relation of moisture content to capillary and/or chemical forces have as much to do with the shape of the stress-strain curve as possible disturbance. The simple compression test is probably a better all-round test than the direct shear test but since the application of a primary axial stress sets up secondary stresses over which we have no control, it appears that the triaxial compression test would have more significance.

The triaxial compression test due to its flexibility of operation provides us with a tool that can be of aid in determining the effect of disturbance on the obtained strength values. If we assume that a perfectly undisturbed sample retains more elasticity than a disturben specimen; i.e., the structure set up by nature has not been sensibly altered, one can with a certain degree of success determine the effect of disturbance by cyclic loading, provided the strain has not exceeded 25 to 50 per cent of the ultimate. The resulting hysteresis loop will close on the general trend of the stress-strain curve if the material is truly undisturbed and has not been prestressed. Other triaxial compression test series performed during the design of the Denison Dam have shown that the apparent prestress can be approximately determined by the proper combination of quick and slow tests if the samples are undisturbed and the geologic history is well known.

GEOLOGIC CONSIDERATION: Probably no one consideration is of more importance in planning a test program, than a complete reconstruction of the geologic history of the soil before it is sampled and tested. In fact a straight forward dependence on the so-called pre-consolidation load as determined by the consolidation test without a thorough knowledge of past geologic history is as bad as not knowing whether or not a given material is disturbed.

Certain clays such as the volcanic clays from the bed of Lake Texcoco, Mexico, D.F. for which geologic history is well known and which are compacted to abnormally high void ratios, make perfect examples of the determination of the consolidation characteristics because of the "classic, almost photographic enlargement" of the consolidation test features. Other clays like the glacial clays from the Great Lakes Region in which the geologic history is more obscure, are difficult to rate with respect to determination of the condition of the soil. Recently re-worked alluvial, plastic clays also present problems in identifying sample disturbance. Other soils that present even more difficulty in the determination of condition, are marly and shaley clays. These materials, while somewhat brittle in their initial state of consolidation, become truly plastic when the structure is broken down and the material remolded. Soils of this type have been laid down as a sedimentary deposit and consolidation effected by the weight of material that the forces of nature may have long since removed. It may be probable that periods of recompression have occured, caused by the deposition of additional material, with re-expansion upon its removal, so that the number of compression and expansion cycles to which the material has been subjected is problematical. One may never know whether we are now living in a period of compression or expansion. Even with an accurate knowledge of that fact, the determination of the pre-stress by means of the shapes of the void ratio-pressure curve or intersection of Mohrs rupture curves for quick and slow triaxial compression tests is practically impossible. Additional complications arise when primary expansion occurs.

CONSOLIDATION & EXPANSION TESTS: The consolidation test is a means of measuring relations between vertical stress and volume change. Generally it is assumed that negative volume changes are to be measured, but on certain material types, positive volume change will result even under a considerable load. The general types of materials susceptible to extreme volume changes range from the truly plastic clays to the somewhat brittle marly clays. On the basis of tests performed by the writer no truly plastic clays[1] have ever shown primary expansion characteristics when tested in their natural state.

However, numerous poorly consolidated shaley to marly clays have definitely shown expansion qualities. These clays, invariably when air-dry show visible laminae. They are barely perceptible as expansion commences, but become progressively wider as time elapses. Their appearance and extent coincides with the beginning and extent of expansion.

The fracture planes are always in a generally parallel direction and occur as frequently as 25 to 50 to the inch. The fact that the planes follow a common direction, indicate that the separation of the platty phases is due to mechanical or chemical causes. It appears that there are inherent weaknesses in poorly consolidated marly to shaley clays. Examination of these fracture planes often indicates that micaceous silts and iron oxides have settled into extremely thin layers and this stratification coupled with an inherent but basal lamination in the parent material have produced a material structurally weak and especially reactive to expansion. That expansion is caused by progressive separation along well defined planes of discontinuity is apparent. Just what causes the initial separation is not so certain. The causes may be purely chemical, mechanical or a combination of the two.

Assuming that capillary forces are developed, it is probably certain that individual clay grains will be under stress and will tend to change volume. The difference in dimension for any one grain will be small, but the stress is present and it would be relieved by failure along planes of weakness,

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[1] Plastic clays obtained from Eastport, Maine; Chicago. Ill.; Mexico, D.F.; Columbus, Ohio; Woodward, Oklahoma.

exhibited by the lamellar structure. It is apparent then, that the usual testing procedures employed on plastic materials cannot be employed in testing materials that have definite primary expansion characteristics.

Fig. 1 shows the results of a standard consolidation test performed on a volcanic clay. Note the rapid change of void ratio with increase in vertical stress and how clear cut the determination of the pre-consolidation load. Now refer to Fig. 2 which illustrates some of the possibilities in planning a test procedure for use on expansive materials, It is at once apparent that the only variations in testing technique can be a change in the rate and magnitude of vertical stress application. Curve (a) shows a typical test curve of void ratio plotted against pressure on a logarithmic scale. Point "A" corresponds roughly to the void ratio in situ and point "B" represents a new void ratio obtained after permitting the test specimen to expand under zero pressure (water being available to the specimen at all times.). Load increments are then applied in geometrical progression. It is obvious that the character of the material has been considerably altered, thereby changing the apparent consolidation characteristics. Curve (b) represents the pressure-void ratio curve of a specimen with the same initial void ratio, point "A" loaded incrementally. However, note that the initial load increments were insufficient to prevent expansion and the void ratio increased until the vertical stress and expansive pressure became stabilized after which consolidation continued. Again the test procedure has introduced complexities in the structure of the material which makes interpretation difficult. Curve (c) shows the pressure-void ratio curve of a specimen in which the initial void ratio, as expansion tended to develop, was maintained until vertical stress and expansive pressures were equalized after which the consolidation characteristics were determined from data secured by the application of additional load. The apparent structure of the soil has probably not been greatly altered since individual particles have been restrained from reorientation. However it is possible, that internal rearrangement may have occurred, Nevertheless, this type of testing procedure has more to qualify its justification than any of the others considered. Curve (d) shows a plot of a typical test in which no free water is available to the material being tested. No expansion has occurred, since no cause has been introduced. However, since we are interested in causes as well as results, this type of test has no particular significance, except in cases where it is known that there will be no variations in natural moisture content.

SHEARING TESTS: Determination of the shearing strength of soils is and probably always will be subject to question. The strength of soils is usually measured by means of one or more of the following tests, (1) Punching shear, (2) Direct shear, (3) Simple compression, (4) Triaxial compression. Unfortunately the worth of the results obtained, depends not only upon the choice of test and technique but also upon the particular material type, its history (present and past) and how it is to be used.

There are two broad groupings into which all soil tests can be placed; (1) tests to determine structural qualities of undisturbed materials, and (2) tests on soils that are to be reworked mechanically and used as a construction material. While there is a certain over-lapping, the same type of test and choice of technique can not be used for both cases. The problem is further complicated by the fact that sands, clays, marly or shaley clays or combinations of each, require separate and distinct types of test. The discussions presented herein, are limited to undisturbed materials and no reference is made to planning and interpretation of tests on remolded soils.

TESTS ON SANDS: Sands are generally more insensitive to variations in choice of type of test and testing technique than most soils. The strength of sands is a function of the density and particle shape, hence regardless of type of test one can approximately determine that relation. Undisturbed sands will have slightly higher strengths than sands remolded in the laboratory because nature has set up a structure that cannot be duplicated in the laboratory. Materials of this type are somewhat difficult to handle because of the lack of cohesion, therefore the choice of test is limited to the direct shear or triaxial compression test which provide fixed and flexible confinement, respectively. Experiments have shown that comparable results can be obtained with either of the above testing devices. The actual selection of the method depends greatly on the size and type of structure being built. For structures that require complete and comprehensive stress analyses, the triaxial test is preferred because the principal stresses and drainage conditions can be controlled. The effect of pore water pressures can therefore be studied. However, for small and unimportant structures where soil tests are used only as a guide to common practice, the choice of test or testing technique is also unimportant. Substantially the same results will obtain, whether the test is performed, saturated or dry, fast or slow, if the drainage conditions are similar in all cases.

TESTS ON CLAYS: Clays on the other hand are extremely sensitive to choice of test type, test conditions, technique of test operation and time of testing. The history of the determination of the shearing strength of clays can be traced through progressive years of (1) direct shear tests, (2) squeeze tests, (3) torsion tests, (4) simple compression tests (5) triaxial compression tests. Early investigators assumed that the strength of clays could be expressed in the terms of two simple constants, i.e., the cohesion, and the angle of internal friction. Later investigations have shown that conception to be fallacious. We now know that those "constants" are not constants, and that depending on type of test and testing technique, we can get most any answer we please. The proper choice of test depends to a great extent upon the type and magnitude of the problem. Thus for tests to determine the strength of a highway subgrade, whether it be composed of natural or stabilized soil, engineers might prefer the punching shear test or some modification thereof, while others might prefer the simple or triaxial compression test. If one is dealing with a soils problem in which consolidation tests have shown that volume changes under the imposed stress conditions will be small one might successfully employ the simple compression test. On the other hand if one is confronted with a soil that undergoes considerable volume change under stress , it is all important to employ a type of test that will enable one to determine the condition strength, and at the same time, study the stress-strain characteristics under several variable conditions.

The triaxial compression test is the only method devised so far in which control of major and minor principal stresses, strains, and drainage conditions is available. One can also control the time and the method of applying the stresses. On Fig. 3 are shown the results of a series of tests on a plastic clay with time of test being the major variable, the stress ratios being standardized at the start of all tests. It can be seen, that as the total time of test increases, that there is a gradual transition of the apparent angle of internal friction to the true angle of internal friction.

This is due to a gradual transfer of the partial stresses carried by the liquid phase to the solid phase, or in other words from total stress to effective stress. Thus we have an increase of strength with increase in time, other things being equal, which might well correspond to a gradual mobilization of strength as accomplished by nature during the ages. Note that though, theoretically the virgin shear curve should start at zero shearing and normal stress, this is rarely the case. One explanation is - - - that at the time of sampling (assuming an absolutely undisturbed material), we are well along on the scale of time, have therefore an initial strength, commonly termed "cohesion", which may be approximately equal to the "effective" pre-consolidation.[2] It is obvious therefore that we can determine the total range of the strength of clays by means of the quick and slow triaxial test for certain conditions of moisture. Another variation of the quick test that will usually show slightly higher strengths is the consolidated quick test. The relative positions of the curves may be used to illustrate the effective shearing strengths of the material. If the curves are well-grouped, we have an ideal material, i.e., a material that has an initial strength approximately equal to the ultimate strength.

TESTS ON SHALEY CLAYS: Due to the lamellar and generally horizontal structure of many shaley clays and the inherent weakness of the laminae, a type of shear test in which failure can be obtained along planes of discontinuity is undoubtedly the best type of test to employ on material of this type. Since most shaley clays have been alid down in generally horizontal strata it appears that a test in which the shearing forces can be applied in the direction of the

[2] Unpublished test data obtained during the design of Denison Dam (1939) by the late J. P. Hartman, D. R. Ketchum and the author.

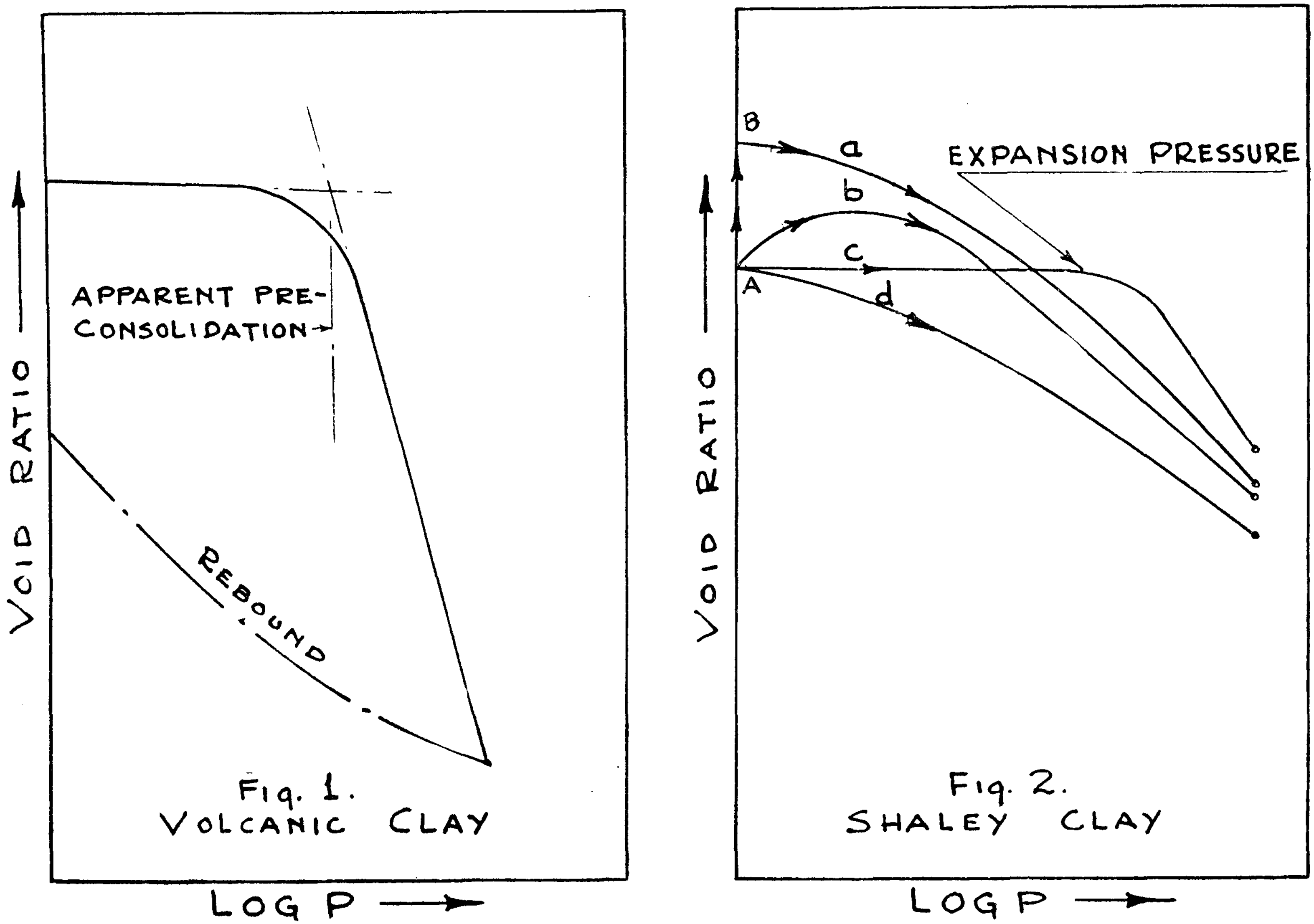
laminae, with a normal pressure acting perpendicular to the stratification is desirable. It is also necessary to determine what effect the direction of the principal particle orientation will have on the shearing strength. A testing method in which the plane of failure can be fixed, is provided by the direct shear test, which can be successfully employed on materials of this type.

The simple compression and triaxial compression test in which specimens are loaded axially without and with lateral support, respectively, usually develops stress combinations that result in a shear failure occurring at some well defined angle, with the major axis. It follows then, that when the plane or planes of failure intersect the laminae, a higher strength value results, than is obtained when failure occurs parallel to the laminae. The influence of stratification of the strength of shaley clays can be strikingly shown if test specimens be prepared for test with the principal planes of stratification at zero, 45 and 90 degrees to the major axis of the specimen and failed in direct shear or simple compression.

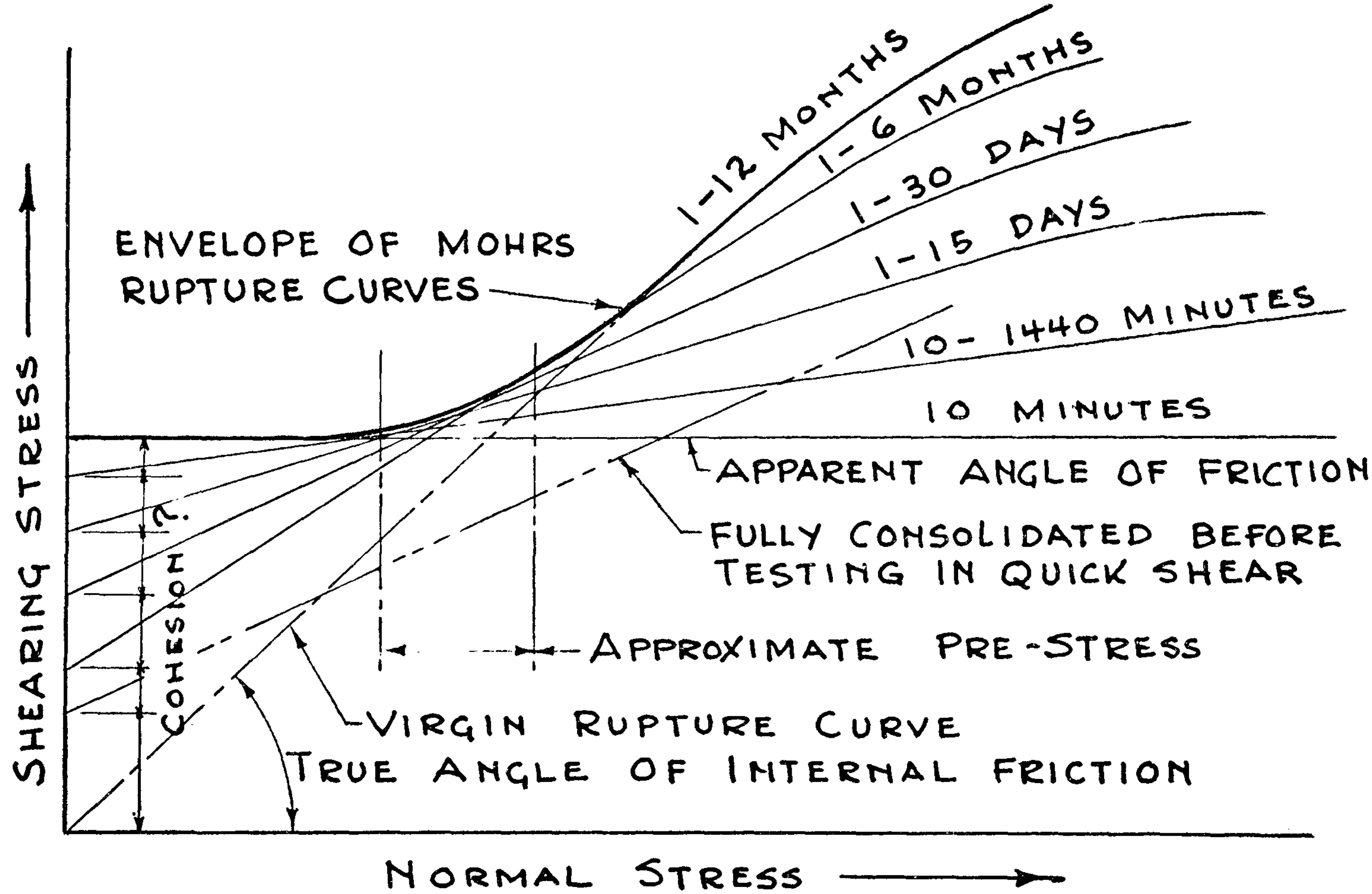
ANALYSIS AND INTERPRETATION OF DATA: The problem of analyzing soil test data is difficult, even when the tests have been planned to fit into a converging and concise program; is doubly difficult if no planning is used. The fact that great divergence of test results usually occurs, even in fairly uniform materials, adds confusion to the picture. Averaging of scattered values cannot be directly accomplished by arithmetical means. A number of conditions that vitally affect the condensing of a mass of data must be considered: such as, (1) disturbance during sampling and preparation for test, (2) geologic history, (3) type of test, (4) testing technique,(5) condition boundaries of test. Each of these phases should be carefully examined and if a predominant recurring feature is noted, it may be taken as a basal fact that may indicate a particular trend. Usually a mass of supporting data are obtained during a series of tests. These data are summarized in tabular form for convenience in plotting. Every possible combination of plot can then be made on arithmetical, semi-logarithmic, or log-log scales. Usually, the trend of the data , if it follows some set law, can be determined, and a curve or set of curves drawn to illustrate graphically the results of the tests.

A review of the various proposed methods of determining how much disturbance affects soil test values, indicates that no infallible methods are available. That disturbance affects both consolidation and strength values is a fact that cannot be overlooked. However, during any complete soil investigation, the scope of the problem is often such that only pilot tests to determine how much disturbance is being caused by the sampling methods, can be performed. On the basis of these tests one must either discard or improve the sampling technique or recognize the limitations of the soil test data obtained.

The geologic history of a soil often provides a guide to interpretation of unusual properties sometimes attributed to other causes. In consideration of all the factors affecting the accuracy of soil tests, it should be realized that it is impossible to set down on paper an exact procedure for interpreting soil test data. In view of the complexity of many problems associated with soils and the fact that the soils engineer is often confronted with materials that are too well consolidated to call "soil", but not hard enough to call rock, it becomes necessary to work out variation of accepted testing practice. These testing variations, while in the nature of research, will, if properly planned, provide guiding data that could have been easily overlooked, if the tests were performed in the usual manner. The prime factor in planning soil tests is the prearrangement of a testing program toward a definite end, after having thoroughly studied all known variables before the tests are started, otherwise hours of effort will be spent in trying to decipher the useless results of tests performed aimlessly.



CONSOLIDATION TESTS



TRIAXIAL COMPRESSION TESTS - CLAYS

FIG. 3

UTILIZING AVAILABLE SOILS IN AIRPORT CONSTRUCTION*

by

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Probably the most important one factor involved in airport construction is the existing soil condition: the types, characteristics, and arrangement of the soils on, and in the vicinity of, the site. This, in turn, transfers a tremendous importance to the preliminary soil survey. Although a major consideration must be given to many other factors and utilities, including the requirements of the Army Air Corps and Civil Aeronautics Administration, in chosing a site, the final choice between several sites is often predicated entirely on the existing soil conditions. In addition, a study of the soil report, in conjunction with a contour map, will have a tremendous influence on the choice of pavement design, cost of construction, and time required for completion of the project.

The stability and draining properties of the soils encountered on the site have a strong influence on the cost of constructing the finished airfield regardless of whether a high-type permanent field or a temporary auxiliary field is planned, and the cost of construction is always a major factor, even in times of national emergency. The tremendous influence caused by prevailing soil conditions is probably emphasized greatest in the construction of auxiliary fields where the minimum amount of construction necessary for all-weather landing is desired. The cost of constructing such a field approximately three-fourth miles square may vary from $5,000 to $1,000,000, depending

* Presented by Lieutenant T. A. Adams, Jr.

almost entirely upon the available soil and drainage. A factor which is likely to be even more important at the present time is the speed of construction, that is, the length of time required to get the field ready for operation. On some sites the field can be made ready for use within a few weeks, whereas in other cases several months are required and the progress may come virtually to a standstill during long periods of wet weather because of the soil conditions encountered. After the site is chosen, the engineer cannot begin to intelligently design the most economic pavement that will satisfactorily serve the purpose without a complete knowledge of the location and characteristics of the soils available on the site and of the high quality materials available in the vicinity.

Due to the difficulty of obtaining sufficient trained inspectors and field engineers and the large amount of construction in progress at the present time, it is necessary that the maximum amount of inspection and control be handled with a minimum number of men. For this reason it is impossible for these men to make the explorations during the progress of construction. These explorations and tests must be made in advance and furnished to the field inspector in a form ready for use at the start of the project.

SOIL SURVEY

The first stage in the preliminary soil survey consists of a thorough utilization of all available information. In addition to a visual inspection of the entire site using occasional preliminary borings, all local engineers should be contacted and soil survey maps and bulletins, prepared by the Soil Survey Division of the U. S. Bureau of Chemistry & Soils, should be studied.

The second stage involves the utilization of field parties in locating and sampling the existing soils and base materials and the utilization of the District Laboratory in testing and classifying these materials and compiling a soil report. The organization of the U. S. Engineer District contains two complete field crews, whose duties consist of making complete soil surveys of the proposed airfield sites. The practice is for one of these crews to commence explorations using hand augers and other hand-digging methods in advance of the topographical survey crews and well in advance of any design of pavement types for the project. If the general location of the runways, taxiways, aprons, etc., are not known, which is often the case, the field is marked off into a gridiron and a large number of holes are dug. The number of holes depends upon numerous factors such as the uniformity of the deposit as shown by the preliminary wide-spaced holes. As experience is gained, these field crews are able to use a considerable amount of judgment in the number of holes to dig and the depth to which the individual holes are dug. As a rule holes are dug to a depth of at least three feet below the expected runway grade. Occasionally holes are dug to a depth of eight or ten feet. Needless to say, these holes are deepest in the high points which will be the cut areas during construction.

A sample is taken of each different stratum of soil encountered and the exact depth represented by the sample is recorded. Each of these holes are spotted and marked and the elevation of the surface of the hole is obtained later by the crew making the topographical survey. A majority of the samples are of the minimum size necessary for making the Atterberg Limits tests or the Mechanical Analyses, while a large sample is taken of each of the different types of soil encountered on the field. These large samples are used for making the Proctor Test and any other tests that may be necessary at a later date.

Investigations are also made of sources of material in the vicinity which have possibilities for use as flexible base material in the runway construction. One of the crews engaged in these types of investigations carries the amount of portable testing equipment necessary for obtaining the percentage of soil binder and the linear shrinkage of the soil binder. The "soil binder" portion contains all particles smaller than 40-mesh in size. These two soil tests are used as a guide in determining which of the available sources shows the greater possibility of producing suitable base materials. A larger number of samples are taken from the most promising sources and, whereever possible, the holes cover an area considerably larger than is contemplated to be necessary for final use. Carefully chosen representative samples of all possible sources are submitted to the District Soils Laboratory for tests.

Upon arriving at the District Soils Laboratory, the samples are laid out and examined. Complete tests are made on a large number of representative samples, while identifying tests, such as the Bar Linear Shrinkage Test and the Mechanical Analysis, are made on the remaining samples. By comparing the visual appearance and results of the identifying tests with those of the samples which are completely tested, the essential test constants, such as plasticity index, are estimated for all of the samples. When all tests are complete, a soil plan is drawn up in the District Laboratory which shows the plot of each hole at its exact location on the plan of the airfield site, the essential characteristics, such as liquid limit, plasticity index, percent of soil binder, and depth limits of each stratum of soil encountered.

Each of the samples taken from the base material sources are completely tested for all of the tests included in the base material specifications. These tests usually include the percent soil binder (portion passing 40 mesh), Liquid Limit, Plasticity Index, Linear Shrinkage, and a complete or partial mechanical analysis. If the material will have to be crushed for use in constructing a base, the samples are crushed prior to being tested. The preparation and testing of flexible base materials follows the procedure used by the Texas Highway Department which consists of air drying, slaking, and then removing the soil binder portion (particles smaller than 40 mesh in size) from the sample by washing. It has been found that this is the only manner in which all of the soil binder actually present in the material can be removed and tested. Tests have been made which show that even a road gravel which appears to be easily separated from its soil binder by the dry-screening process producing a plasticity index of, for example, seven, the same gravel well slaked and washed will produce slightly more soil binder with a plasticity index of possibly 10 or 11. These additional particles of clay, which are revealed only by washing the sample, actually are in the mix coating the larger snad and gravel grains, and, under an impervious top, will absorb enough moisture by capillary attraction to help lubricate the mis. In the case of material such as caliche, it is impossible to tell which of the particles which look like soft rock will prove to be soil binder in the presence of water and which particles are actually the rocks.

After the tests results are completed, each hole is studied separately, and the depth and test results of the material in each stratum are considered in choosing the depth limits between which the deposit can be used during construction. If no sample was taken representing this full depth, a composite sample can be made and tested using an amount of each stratum proportional to its depth in the deposit. A locality map is drawn showing the location and extent of the available base materials. The location of each hole is marked and a log of the hole is included which shows the depth of stratum represented and the essential test constants.

DESIGN

After the topographical survey has been completed and a contour map is drawn, the runway grades and locations are chosen. It is possible in certain cases that a study of the soil plan will assist in locating the runways or choosing the grades. Due to the large area of earth excavation and grading involved, and the relatively flat grade required for finished airport construction, the runway elevations are usually chosen in an effort to obtain a balance between cut and fill in the final graded field.

The next step in choosing the design of the pavement to be used is a detailed study of the available soil that will be excavated, that can be excavated, and that will be normally in place at the proposed subgrade elevation. The first consideration is obtaining a stable subgrade, which is considered generally to be soils having a Liquid Limit less than 35, Plasticity Index less than 12 and a Linear Shrinkage less than 7. A study of the soil plan in conjunction with the contour map and runway excavations will show whether this type of stable subgrade will be automatically in place or whether it will be necessary to actually specify a layer of this material or sub-base. It will also be possible to determine whether there is sufficient quantity of this selected material on the site and whether it can be handled by the contractor in such a manner as to produce the desired quantity of select material under the finished runway. If sufficient select granular material is not available from the site, it is necessary to locate and test the closest sources of borrow material suitable for this purpose.

Wherever possible, this stable granular material should be on the surface of the finished subgrade regardless of whether the final design is to be a flexible base or a concrete pavement. In the case of flexible base design, the presence of a layer of stable granular material in the top layer of the subgrade will mean that a shallower depth of flexible base material can be used. Since the cost of selected granular soil is only a fraction of the cost of flexible base material, this design usually results in a considerable saving in the total cost of the project. Where a concrete pavement is to be placed, the subgrade problem is usually not one of stability, because the high flexural strength of concrete does not require as stable a foundation as do most of the flexible types. The granular material on the surface of the subgrade, in the case of concrete pavement, serves principally as a blanket for preserving the moisture in the clay undersoil, thus minimizing the danger of warped joints which will result when the clay soil is allowed to dry slightly prior to paving, followed by a subsequent absorption of water through the joints during rainfall, thus producing a high swelling force. For all types of pavement design, the provision of a stable granular layer on top of the subgrade will tend to preserve the moisture in the underlying clay both during and after construction, and will provide a surface on which construction can be resumed more quickly following periods of rainfall. A survey of the soil existing on the site and in the vicinity of the site may disclose the fact that there is no sandy soil within a distance of economical haul of the airfield, and if a flexible base is designed, the granular layer is omitted and the depth of base increased. This case, however, is quite an exception; at many of the airfield sites along the Gulf Coast where no sandy soil was known to be in the vicinity, a diligent search has usually uncovered sufficient quantities of nearby material.

The first type of design usually considered is a flexible base with a faiily thick asphaltic surface. If sufficient quantities of satisfactory quality flexible base materials such as gravel, crushed rock, caliche, iron ore, oyster sheel, etc., can be found near the site, this type of design will usually be more economical than any design of equivalent strength. For this reason a diligent search is made to determine the location and quantity of each possible source of flexible base material within an economical hauling distance of the field. Where a flexible base material is not available that is completely satisfactory in its natural state, a study is made to determine whether some of the deposits can be made satisfactory by any one of several methods, the most common of which consists of adding a fine sand or silty soil for the purpose of reducing the soil contents, such as Liquid Limit, Plasticity Index, and Linear Shrinkage, of the final mixture. If there are several sources of flexible base material which can be economically obtained, but some of which are considered bet'er than others, and effort is made to write the specifications to include only the better material.

Where no flexible base materials are economically available, some other type of design, such as Portland cement concrete, asphaltic concrete black base, or soil stabilization must be considered. On certain airfields where the present and future importance of the airport is great, both from the standpoint of volume of traffic and expected loads, a high type pavement may be chosen even though other materials are available. Each airport presents a separate engineering problem and all factors must be considered before the final design is chosen. To date no airfields involving soil stabilization have been constructed in the Galveston District, but it is believe that for future work some soil stabilization projects will be designed, based on the results of preliminary investigation tests made in the District Laboratory.

FIELD CONTROL OF QUALITY DURING CONSTRUCTION

The most complete preliminary explorations, preliminary testing and well chosen design will be of little benefit if the field control does not produce a pavement which corresponds to the original design. This applies both to obtaining the desired quantity and quality of material in each layer but also to obtaining the maximum possible stability in each one of these materials. No matter how well chosen the design may be, the final success of the pavement rests on the shoulders of the field engineers and the field inspectors. A Soils Manual has been written covering the exact testing procedure and the methods of control, and copies furnished the field inspectors. This Soils Manual has proven an invaluable aid to the relatively inexperienced personnel. Due to the large construction program and the relatively small number of trained inspectors, it became necessary to train the field forces as quickly as possible in order that the inspectors may be able to learn the use of these tests and their application. Each of the projects provide for a field laboratory and enough field laboratory testing equipment for making the desired tests.

An effort has been made to allow for part of the expected inexperience on the part of the inspecting forces by providing a safety factor in the specification limits over and above that which would be used with a highly trained inspection force. Also, a very complete preliminary soil survey and report is prepared with this thought in mind. The completeness of this preliminary report reduces the amount of tests that must be made on the field during construction. Some airport sites contain only two or three distinct types of soils which can be readily distinguished by eye and the characteristics obtained from the preliminary soils plan. In these cases very few quality tests need to be made by the field inspectors during the course of construction, which is very desirable in that the inspector is free to exercise closer control of construction methods and the placing of the soils. Where numberous types of soil are encountered, the Bar Linear Shrinkage Test is used by the inspector to identify the soil and thus determine its permissable location in the finished project. A few projects have trained inspectors and soil technicians who are capable of making the Liquid Limit and Plasticity Index Tests, but it has been found that untrained inspectors can attain more accurate results by making the Linear Shrinkage Test.

Where a flexible base is used, the borrow areas are selected from the preliminary survey described above, No matter how complete the preliminary survey on a flexible base material pit, it is always necessary to exercise constant testing and control during operation. Numerous layers and areas of the pit are uncovered which were not discovered by the preliminary testing; also, the effect of the crushing action may alter the quality and quantity of soil binder that was expected from the preliminary tests. On some pits it is necessary to add and mix a sandy soil in order to produce a satisfactory final product and this calls for constant control, testing and judgment on the part of the inspector. Wherever possible, a trained soil technician is used in the field control of flexible base material so that the Percent Soil

Binder, Liquid Limit, Plasticity Index and Linear Shrinkage Tests can all be made and compared against the specification limits. Wherever a trained soil technician is not available, the method of preparing and removing the soil binder is used as described above and the Bar Linear Shrinkage Test is used to control the quality of the soil binder.

FIELD CONTROL OF COMPACTION

Producing soils and base materials of satisfactory quality is only the first step in the construction of pavements. The moisture content of these materials must be carefully controlled before the highest final stability of each material can be utilized. Practically all engineers using soils in construction are now familiar with the relations between the compacting moisture and the density obtained, and the fact that a given soil, under a given compactive effort, will have one optimum moisture content, i.e., the moisture content which produces the highest "dry density". It is now always realized, however, that the control of moisture content <u>after compaction</u> is equally important and that this control is entirely different for clays than for granular materials. A brief review of the reasons for compacting soils will explain why this is true.

There has been considerable research done in the past few years concerning the factors influencing swell in the clay soils, which tests have been correlated in many instances with field behavior. <u>The results show definitely that the swell varies in proportion to the percentage of air voids present in the compacted soil prior to being allowed to absorb water and swell.</u> In every instance the condition of minimum swell was found to be consistent with a condition of minimum air voids. Thus, the contributing condition toward soil swellage is neither moisture content nor state of compaction singly, but that combination which results in the minimum air voids in a given soil compacted under a given pressure. If two specimens are molded at different moisture contents and different pressures so that both specimens have the same percentage of air voids, but the one molded at the highest pressure has the greatest density, this specimen of highest density will have the highest final stability after a period of absorption. Therefore, if we wish to obtain the highest final stability, as well as the minimum future swell, it appears highly desirable to compact the soil at or near the optimum moisture content and retain this compacting moisture. Tests have been made which show that if the specimens molded at optimum moisture are dried before being subjected to capillary water, the resulting swell will be considerably greater than for the same specimens which are not allowed to dry. This is particularly true of the plastic soils. Numerous tests have been made which show that if specimens are molded at various moisture contents under the same amount of compaction, then subjected to capillary action with or without being dried, the specimens molded at, or slightly wetter than, the optimum moisture content will absorb the least amount of water and will retain the highest final stability.

In the case of the plastic clays, therefore, it appears that after the soil have been compacted at its optimum moisture, or slightly wetter, the highest final stability, as well as the minimum future swell, will be obtained in that material if it is not allowed to dry but is covered while it still retains the compacting moisture.

In the construction of pavements the more granular type of soils and base materials will usually be placed near the top of the roadbed. In this case, particularly in flexible base construction, we know that the granular soils will not be subject to appreciable amounts of swell; therefore, our principal concern lies in obtaining the greatest amount of final stability in these soils in order that they may satisfactorily be able to support the unit pressure that is transmitted to them from the load at the surface. In these types of soils it appears that a higher final stability might be obtained by compacting the soil at, or a little wetter than, the optimum moisture content. The base materials should then be allowed to partially or totally dry before an impervious top is placed. The Texas Highway Department conducted a series of tests throughout the state in which moisture contents were taken at monthly intervals for three years from subgrades and bases under old asphalt pavements. A majority of these roads had served as open-surfaced roads for several years and thus had an opportunity to dry out prior to being surfaced. It was found that all of the granular soils and base materials (with P I values less than 12 or 15) consistently maintained a moisture content considerably less than their optimum moistures. Since the stability of a given compacted soil is largely a function of its moisture content, each of the granular materials maintained a stability considerably in excess of their stability at their optimum moisture contents. The modern practice is to include the base construction and the asphalt surfacing in the same contract, and numerous instances of "shoving" have occurred in a good base material where the base material was not allowed to dry after being compacted and before being surfaced.

All of the previous discussions point to the fact that it will be very desirable to compact all soils at the optimum moisture content, or slightly wetter, for the compactive effort being used. This applies regardless of whether the moisture is to be carefully preserved or the mixture is to be dried. Where the moisture content is to be carefully preserved, the mixture will contain less air voids if compacted at or slightly wetter than the optimum moisture, and will produce less subsequent swell, a lower final moisture content, and a higher final stability. Where the misture is allowed to dry, there will be a greater resulting gain in density in the mixture compacted wetter than optimum. This in turn will produce a higher final stability.

If for the purpose of adopting a rule-of-thumb procedure, a "granular material" is defined as a sandy soil with a P I less than 10 or a base material whose soil binder has a P I less than 15, the following schedule should be followed:

Under Concrete Pavement

Granular Soils - Compact at optimum moisture - then moisture may be either preserved or lost

Plastic Soils - Compace at optimum moisture - preserve the compacting moisture

Flexible Base Construction

Granular Soils - Compact at optimum moisture - allow to dry before topping.

Plastic Soils - Compact at optimum moisture - preserve the compacting moisture.

It is generally realized that the density obtained by rolling is influenced by many factors, such as, the type and weight of compacting equipment, size and spacing of feet in a sheep's-foot rooler, depth of penetration, depth of lift being rolled, number of passes, uniform distribution of moisture, and time at which the density is checked. It is very desirable that the engineer and inspectors become thoroughly familiar with these factors because this knowledge will enable him to obtain a better job with less trouble, work and expense.

It is unfortunate that the maximum weight of sheeps-foot rollers is somewhat limited in this state, because the early specifications for rollers used in pavement construction set the standard, and all pavement contractors own rollers with which it is difficult to obtain greater than 200 or 225 pounds per square inch pressure on the feet. The standard Proctor test is used, employing a 12 inch drop of the $5\frac{1}{2}$ pound hammer, because this compactive effort approximates very closely the compactive effort of the sheeps-foot rollers now in use.

All "fill" materials, all flexible base materials, and the top layer of all "cut" sections must be compacted under the supervision of the soil inspectors on the job. The usual specifications require at least 95% of the density obtained in the standard Proctor test, and further provide that the compacting moisture must be retained in the plastic clay soils. The more recent specifications include a clause to the effect that all flexible base materials must be totally or partially dried prior to the placing of an asphalt surface.

The inspectors are equipped with a field laboratory building located on the project and all the field testing equipment necessary for making the Proctor tests and for checking the moisture and density of the compacted material. The rubber balloon density apparatus is used on most of the projects for checking the density of the compacted material, but certain soils appear to lend themselves more satisfactorily to the use of other methods, such as the sand funnel method or the use of a small hand-driven monel metal cylinder.

Wherever it is possible, with the amount and type of inspection personnel available and the types of soil encountered, the usual standard method of controlling compaction is employed. This procedure has been discussed fully on many previous occasions and will be mentioned here only briefly. This procedure consists of first making a preliminary Proctor Curve, with the minus 1/4-inch portion, of every soil encountered on the project. An attempt is then made to compact each soil at its optimum moisture and obtain a chosen minimum percent of the Proctor maximum density. From the Proctor curve, illustrated on Figure 7, the minimum required density would be 110.7 lb. per cu. ft. for this soil. If the material contains plus 1/4-inch particles, additional tests and calculations must be made along with the density check tests in order to calculate the density of the minue 1/4-inch portion. Due to the rapidity of construction of the present contracts and the small size of the inspecting forces, very few Proctor tests can be made during the progress of the job. The moisture-density curves used by the inspector are those made in the District Laboratory on the samples taken in the preliminary soil survey. These curves are included with the other test data in the laboratory soil report; a copy of which is furnished the inspector at the beginning of the project.

The method of control described above is fundamentally sound but has two practical drawbacks when applied to field compaction where numerous shallow cuts and fills are encountered. Due to the rapid change in the types of soil, and the unexpected types encountered, it is often difficult, if not impossible, to accurately match each soil with the proper Proctor curve for that soil. If incorrectly matched the inspector may either (1) be attempting to obtain a density that is practically impossible to obtain for his embankment, or (2) be considering satisfactory a density that is below 95% of the true Proctor maximum density of his soil.

The second defect of this method of control is that it considers only the desired density for a soil and does not take into account the amount of air voids present. It is quite possible to have two different sections of the soil shown on Figure 7 compacted to a density of 113 lb. per cu. ft., with one section compacted at 8% moisture and the other section compacted at 14% moisture. When judged by density alone, these two sections appear to be of equal value, whereas, the section compacted at 14% moisture contains a much lower percentage of air voids and is actually much more satisfactory in practically every case. If the soil has a high plasticity, this lower percentage of air voids will result in a much lower degree of swell when subjected to capillary water in the roadbed. If the material is a granular soil or base material, it should be partially or totally dried before being covered, and the 14% section will have a greater gain in density and a higher final stability.

In order to overcome the drawbacks mentioned above, the Texas Highway Department has recently devised a method of controlling compaction by determining the percent of air voids in the compacted mixture. This method of control is extremely useful in the present airport construction program because by its use the small inspection forces, often containing men of limited training and experience, can obtain the maximum amount of control over the actual soil compaction. This method appears to be the simplest method of control and has the very material advantage of controlling the two features desired in a compacted soil; namely, (1) a reasonably high density and (2) a low percentage of air voids. Also, it is possible to control the density of materials containing large aggregate by checking the density of the total material, i.e., without having to separate, and make calculations for, the minus 1/4-inch portion of each density determination. This feature will be discussed later under

"Application to Materials Containing Large Aggregate".

Figure 7 shows the Proctor curve for a soil with an optimum moisture of 12.5% and a maximum density of 116.5 lb. per cu. ft.

At this moisture, the zero air voids density equals 124.4 lb./cu. ft.

95% of Proctor maximum density = .95 x 116.5 = 110.7 lb./cu.ft.

Therefore, the point A (containing optimum moisture and 95% of Proctor Density) has a % Air Voids equal to $100\ (1 - \frac{110.7}{124.4}) = 11\%$

If the line of equal air voids (11% in this case) is drawn through this point, any point on this line will have a density equal to 89% (or 100-% Air voids) of the zero air voids density at the same moisture content. This relationship is used to locate the line of 11% air voids. Several points are arbitrarily chosen on the zero air voids line and, for each point, the point representing 89% of that density is plotted beneath the line at the same moisture content. If the compacted soil is then required to have less than 11% air voids, it must fall in the shaded area above the curve of equal air voids (Figure 7). It is possible to meet the minimum air voids requirement and still not have 95% of Proctor density if the soil is compacted at a moisture content considerably higher than the optimum. It is believed that the check is somewhat automatic in this direction, however, because a soil considerably wetter than the optimum will usually be too sticky or too unstable to be worked. As more data is accumulated, it should be possible to set a minimum stability, as tested by the Proctor needle or similar method, which would prevent the compaction of soils with excessive moisture contents.

When the "air voids" method is used for controlling density, <u>the moisture and density tests must be made immediately</u> after compaction is completed, and before the mixture has a chance to lose moisture. Before the project is started, standard Proctor curves should be made on the predominant types of soils and when these soils are encountered, the inspector can check his ability to estimate and control the optimum moisture.

Once the rolling procedure has been adopted, a majority of the moisture content control will be based on the observation and experience of the inspector, checked by frequent air voids determination. When the density checks show too high a per cent air voids, either the rolling moisture or compactive effort must be increased for that soil.

It was hoped that a desirable minimum percent of air voids could be chosen that could be used for all the soils encountered in Texas, but this proved to be impractical. Figure 8 shows the plotting by the Texas Highway Department of optimum moisture vs. Proctor maximum density for 101 soils and base materials of various types and from various localities on which the Proctor results were available. On this chart the plotting number represents the PI of the material. Curve A represents a line of equal air voids (5.3%) which is an average line for a large number of the points. If the specifications require 95% of Proctor density, then curve B is drawn at 95% of the density of points on Curve A. Curve B is a line of equal air voids (10%), and the soils used in establishing Curve A must be compacted to less than 10% air voids, i.e., their compacted moisture vs. density must plot above Curve B.

A group of the fairly clean sands, identified by the low PI's (plotting numbers) have an average curve represented by Curve C (14% air voids). These sands should be compacted to fall above Curve D, which is 95% of Curve C and a minimum air voids content of 18.3%. There are also several clay soils (highPI's) in the lower right hand part of the graph which would require a separate minimum % air voids.

It is believed that, for the average project, a curve of equal air voids can be drawn that will be an average for all or most of the soils on that project, and a minimum air voids required that will insure 95% of Proctor density. One or more unusual soils may be encountered that will require a separate minimum air voids requirement such as Curve D but these exceptions can be noted and identified on the project.

This method of control is, of course, subject to errors. It can be seen that the points that fall below the average line of equal air voids (drawn through the points of maximum density) must be compacted to slightly greater than 95% of Proctor density in order to fall within the required minimum % air voids. Likewise, the points above the original line can meet the air voids requirements with slightly less than 95% of Proctor density. The standard method of comparing each soil with its original Proctor curve is subject to the same or greater errors, however, because of the difficulty in choosing the correct curve for each soil that is encountered, which will often include soil types not previously tested. <u>Application to materials containing large aggregate</u>: Assume that the Proctor curve shown on Figure 7 represents the tests made on the minus 1/4-inch portion of a base material and the point A contains 95% of Proctor density and 11% air voids. If the % of material passing 1/4-inch, the bulk specific gravity of the plus 1/4-inch portion, and the percentage of water necessary to "saturate surface-dry" the plus 1/4-inch portion are known, the <u>theoretical</u> moisture and density of the total material can be calculated with the following assumed conditions:

(1) The plus 1/4-inch portion is saturated-surface dry.

(2) The minus 1/4-inch portion is at optimum moisture and 95% of Proctor maximum density.

(3) The minus 1/4-inch portion fills all of the voids in the plus 1/4-inch portion.

This theoretical moisture and density of the total material will be at some point such as P on Figure 7, and will always contain less air voids than the point A because we have assumed no air voids in the plus 1/4 - inch portion. This would indicate that a lower "minimum per cent of air voids" should be chosen for checking the total material.

A large number of analyses have been made by the Texas Highway Department of actual moistures and densities obtained in several types of compacted base materials and it has been found that the percent air voids in the total material are approximately the same as the per cent air voids in the minus 1/4" material. These analyses also showed that, with few exceptions, the materials that tested below 95% of Proctor density in the minus 1/4-inch portion also showed a % air voids in the total material that was greater than the minimum air voids requirement chosen from preliminary Proctor tests on the minus 1/4-inch portion. This slight discrepancy between theory and actual behavior is undoubtedly due to the fact that the three assumed conditions, listed in the second paragraph above, are not all attained in a compacted soil or base material containing aggregate larger than 1/4-inch.

This is a very fortunate condition in that it enables the following method of control to be used in the compaction of base materials, except where there is practically no minus 1/4-inch portion as in a sledge stone base or rock fill:

(1) Make preliminary Proctor tests on the minus 1/4-inch portion of several samples of the materials to be used, and plot the optimum moisture vs. maximum density for all samples on the same sheet.

(2) Draw an average "line of equal air voids" through these points. Draw a line representing 95% of each of these densities; this line is also a

line of equal air voids and this minimum % air voids is chosen for use with the total material to be used from the deposit, pit, or project as the case might be.

(3) Immediately after each section is rolled, make a density test on the total material and either plot the % moisture vs. dry density on the original sheet or calculate the % air voids in the total material. If the per cent air voids is less than the required minimum or the plotted point falls above the "minimum air voids" line, the material is satisfactorily compacted. The method of checking the actual density in the minus 1/4-inch portion need be used only when density tests are made after the compacting moisture has been lost by drying.

The enormous saving in time, expense, and convenience of the air voids method of control is self-evident. In addition, the elimination of the numerous calculations involved in determining the minus 1/4" density removes a potential source of errors.

If the inspector wishes to know whether a given wetted soil is too wet or too dry for rolling, the following procedure can be used:

(a) Make a Proctor specimen of the soil in its existing moisture condition and obtain the wet density. Also examine specimen for stability.

(b) Obtain moisture content of the material by the "quick dry" method, using the correction factor obtained from preliminary comparative tests with standard drying procedure (quick dry method requires 15 to 30 minutes.)

(c) Calculate the dry density (weight of oven dry soil per cubic foot) of the Proctor specimen molded above.

(d) Plot moisture vs. density of the molded specimen on the chart and calculate the percent air voids or compare with the line of desired minimum air voids.

(e) If specimen shows too high a % air voids (plots below the desired minimum % air voids line) more water should be added. Passing an average shaped optimum moisture curve through the plotted point, the inspector can estimate the percent of additional moisture needed.

(f) If the percent air voids are low (plots above the desired minimum air voids line) but the molded specimen is obviously unstable, the moisture content should be reduced before the compacting equipment is allowed on the material. Inspectors can soon learn to judge the stability of a molded specimen with surprising accuracy.

(g) If both of the % air voids and stability are satisfactory, the material is ready to be compacted.

The writer is often queried concerning the differences between airport pavements and highway pavements, with respect to both design and construction. They appear to be very similar in practically every respect, with very few minor exceptions. Various types of pavements are used in both cases, depending upon the expected useage, loads, and the available materials. The maximum unit pressures at the surface of the pavement are approximately equal for the comparable classes of vehicles. The largest planes probably have a greater area of tire contact than the largest trucks and, therefore, a greater depth of base material and stable subgrade material is required. The depth of base used in both cases is still based largely upon experience gained by the proven past performance of similar materials. Numerous formulas, some combined with tests on the materials to be used, have been offered for designing the depth of base required under a given loading. The writer believes that an excellent start has been made towards the solution of this problem, but that none of the present methods are satisfactory because the greatest controlling factor is still the so-called "bearing value" of the subgrade and base material after they have been under an impervious surface for several years. The "bearing value" or shearing strength of a given soil may vary several hundred percent with only a small change in moisture content.

The actual construction of airport pavements has one distinct advantage over the construction of highways in that the operations are confined to a smaller area. This enables the inspectors to more easily and quickly move from one operation to another and thus keep a better control over the job. As a rule there are not nearly as many types of soils encountered in the construction of an airport as would be encountered in an equivalent square yardage of highway pavement, therefore the inspector can more quickly become familiar with the characteristics of the various soils. The large areas and relatively flat grades encountered in airport construction often present a serious handicap in the form of a drainage problem, during construction as well as after construction One of our airport projects which was delayed more than anyother by the high rainfall in 1941 contained a foot of uncompacted sandy A-2 soil underlain with an impervious clay.

One point that all types of pavements still have in common is the fact that the greatest economy of design can be used and the maximum final stability obtained in the chosen design by a proper utilization of the available materials. These available materials can, in turn, be properly utilized only by a thorough preliminary survey and a preliminary testing made prior to the designing stage.

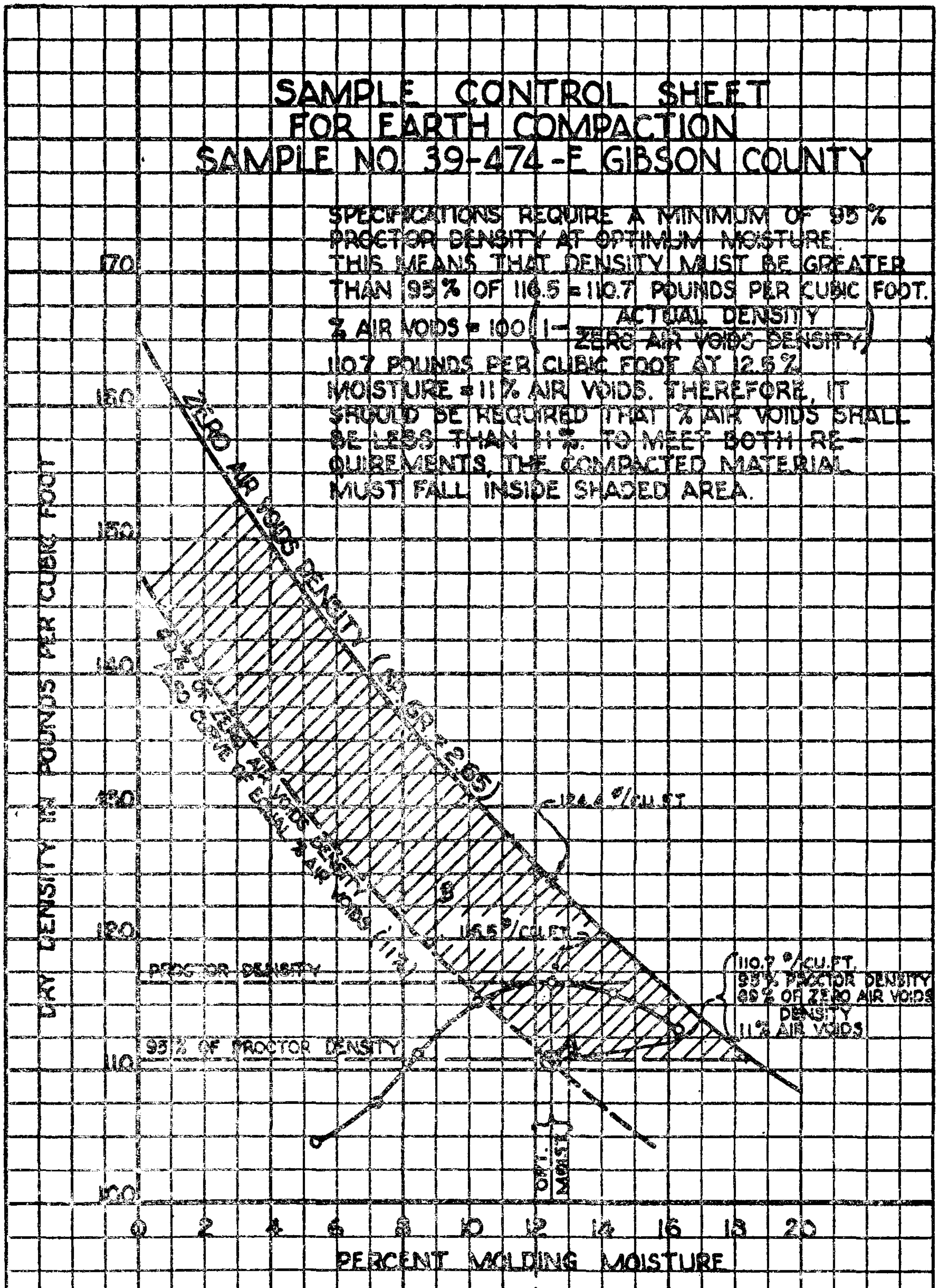
SAMPLE CONTROL SHEET
FOR EARTH COMPACTION
SAMPLE NO. 39-474-E GIBSON COUNTY
SPECIFICATIONS REQUIRE A MINIMUM OF 95 % PROCTOR DENSITY AT OPTIMUM MOISTURE.
THIS MEANS THAT DENSITY MUST BE GREATER THAN 95 % OF 116.5 = 110.7 POUNDS PER CUBIC FOOT.
% AIR VOIDS = 100 (1 − ACTUAL DENSITY / ZERO AIR VOIDS DENSITY)
110.7 POUNDS PER CUBIC FOOT AT 12.5 % MOISTURE = 11 % AIR VOIDS. THEREFORE, IT SHOULD BE REQUIRED THAT % AIR VOIDS SHALL BE LESS THAN 11 %. TO MEET BOTH REQUIREMENTS, THE COMPACTED MATERIAL MUST FALL INSIDE SHADED AREA.
ZERO AIR VOIDS DENSITY (SP. GR. 2.65)
89 % ZERO AIR VOIDS DENSITY
CURVE OF EQUAL % AIR VOIDS (11 %)
124.4 #/CU.FT.
116.5 #/CU.FT.
PROCTOR DENSITY
95 % OF PROCTOR DENSITY
110.7 #/CU.FT.
95 % PROCTOR DENSITY
89 % OF ZERO AIR VOIDS DENSITY
11 % AIR VOIDS
OPT. MOIST.
DRY DENSITY IN POUNDS PER CUBIC FOOT
170
160
150
140
130
120
110
100
0
2
4
6
8
10
12
14
16
18
20
PERCENT MOLDING MOISTURE

FIG. 7

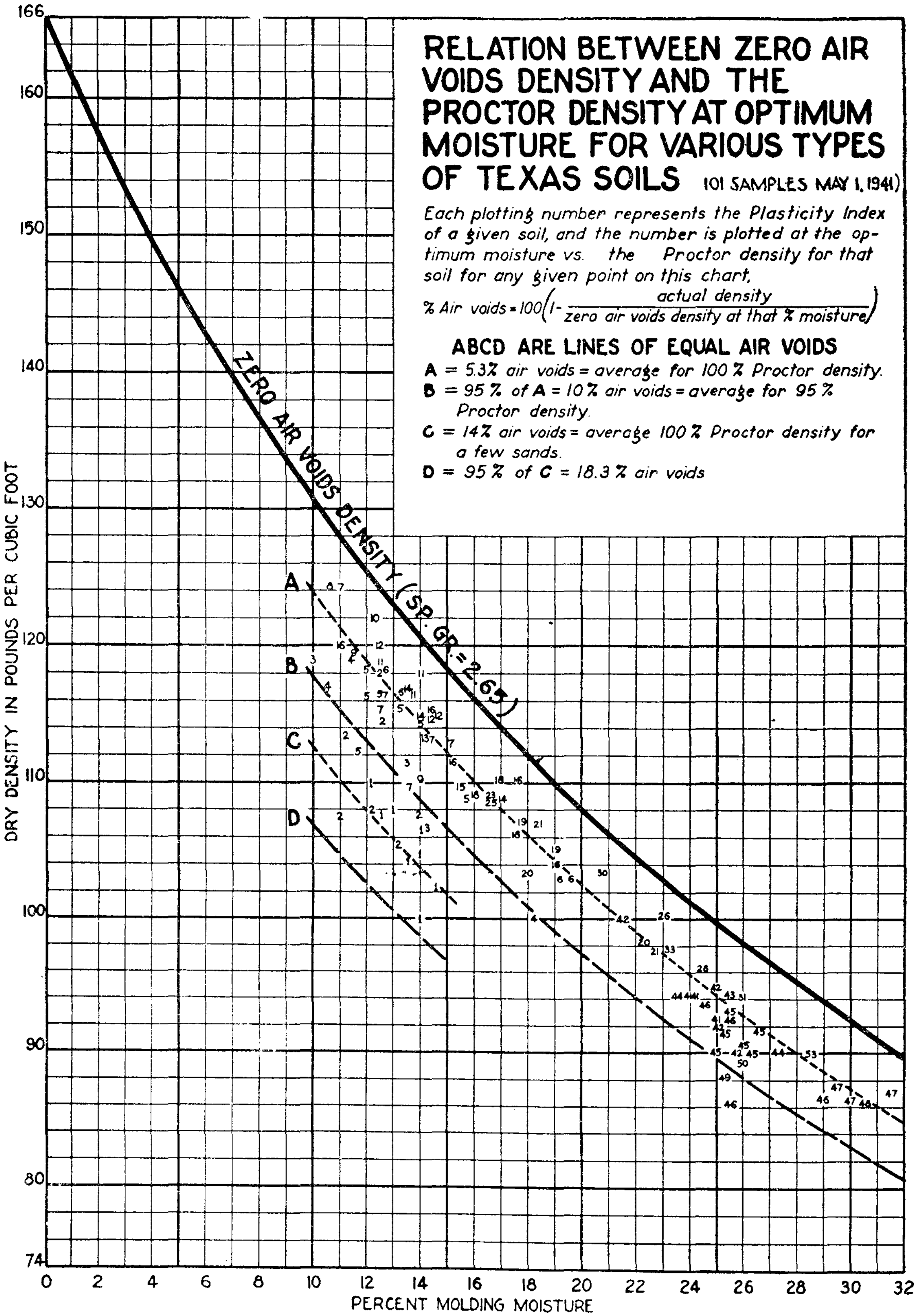
RELATION BETWEEN ZERO AIR VOIDS DENSITY AND THE PROCTOR DENSITY AT OPTIMUM MOISTURE FOR VARIOUS TYPES OF TEXAS SOILS (101 SAMPLES MAY 1, 1941)
Each plotting number represents the Plasticity Index of a given soil, and the number is plotted at the optimum moisture vs. the Proctor density for that soil for any given point on this chart.
$\% \text{ Air voids} = 100\left(1 - \dfrac{\text{actual density}}{\text{zero air voids density at that \% moisture}}\right)$
ABCD ARE LINES OF EQUAL AIR VOIDS
A = 5.3% air voids = average for 100% Proctor density.
B = 95% of A = 10% air voids = average for 95% Proctor density.
C = 14% air voids = average 100% Proctor density for a few sands.
D = 95% of C = 18.3% air voids
ZERO AIR VOIDS DENSITY (SP. GR. = 2.65)
A
B
C
D
DRY DENSITY IN POUNDS PER CUBIC FOOT
166
160
150
140
130
120
110
100
90
80
74
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32
PERCENT MOLDING MOISTURE

FIG. 8

DISCUSSION OF "LIMITATION OF THE FIELD LOADING TEST"

by

H. S. Gilette

Senior Highway Engineer, Public Roads Administration

The "Field Loading Test" which was conceived in an atmosphere of brisk generalization and ushered into the engineering literature with optimistic promises before the advent of soil mechanics as we now understand it has lingered too long in our field practices with a persistence that is amazing without a frank analysis of its shortcomings. During this interim these shortcomings have left much that is on the red ink side of the ledger of engineering practice. Your courageous analysis of its worthless shortcomings merits widespread engineering approval and hearty congratulations. I am in full accord with the conclusions expressed in your paper "Limitation of the Field Loading Test".

For a long time I have been of the opinion that the loading test on clays can be predicted from the inexpensive unconfined compression tests if the strain ratio $(\frac{Dc}{Du} = J$ is known.

Very recently I considered that it was possible to determine this ratio by the simple unconfined compression test alone and did so express myself in that way in the discussion following your paper at the Foundation meeting. In reviewing the literature subsequent to this discussion I have found that others have been initially of the same mind. Quoting from Mr. Austin B. Mason, Research Associate's paper E11 Vol. II of the Proceedings of the International Conference on Soil Mechanics and Foundation Engineering: At first it was considered possible to determine this ratio $(\frac{Dc}{Du} = J)$ by simple unconfined compression tests".

At the time of the meeting I was of the above opinion but subsequent check on my analysis has indicated that my enthusiasm was premature and in error. Presently at least the process of obtaining the strain ratio will require the utilization of the unconfined compression test in parallel and along with surface loading tests to define the strain ratio. In the absence of exact knowledge of the strain ratio for a particular case one can assume the strain ratio for an elastic-isotropic material, which assumption seems to be always on the safe side. This will have to be the procedure for the time being of the engineering practitioner without a large laboratory at his command. Of course, well-equipped laboratories will be able to utilize the tri-axial apparatus to their advantage.

Your paper very definitely discloses that the first essential necessary to predict the bearing power of a foundation soil is an ACCURATE SOIL PROFILE. Without such accurate data, predictions are pure guesswork and rule of thumb work. I wish to reemphasize this essential necessity - that until an accurate soil profile becomes a part and parcel of preliminary investigation, all foundation analyses will remain in the category of rule of thumb procedure.

The second necessity essential to progress in the theoretical analyses is continuing field measurements of settlements of structures in the field. Unless a cooperative effort is made to gather and to record this much needed data, progress in the direction of substantiated theory and still more economical coincident with safe design will be delayed.

The two above essentials stand out as beacon lights for our guidance to greater security in our analyses and greater economy in the ultimate objectives in the era ahead.

SOIL COMPACTION

by

Spencer J. Buchanan

The design, construction and ultimate behavior of earthern structures, regardless of the purposes they are to serve (that is, whether they are dams, levees, highway embankments or the bases for airport runways) are strongly influenced by the compaction of the soil used in them. The original conception and arbitrary standards for compaction, as set forth by Mr. R. R. Proctor* in 1933, have formed the basis for great improvements in the design and construction practices for earthworks. The effect of compaction on the major factors of design - namely, shear, consolidation and permeability - has not been fully explored; for examination of engineering literature shows this effect has been indicated only up to the present, except for cohesionless materials. Efficient construction is vital. For a structure to serve its intended purpose to best advantage, however, the two primary elements - design and construction - must be well balanced. It is the purpose of this paper to show the relationship between the major factors of design and compaction and how this relationship may be reflected in the completed structure.

Before going further, let us review what happens to a soil as it is compacted. Several explanations have been given for this action, all of which are essentially that offered by Proctor, which is described briefly as follows. The compaction of soils is achieved by forcing the fine grains into the voids of the coarse grains to increase the density of the mass. The friction between

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* "Fundamental Principles of Soil Compaction," R. R. Proctor
Four articles in E. N. R. Vol. 111, Nos. 9, 10, 12 and 13, 1933.

the grains must be overcome to accomplish this compaction. For a relatively dry soil, the resistance between the grains to a tendency to rearrangement is large, for the thin water films on the grains provide little lubrication. Also the effect of surface tension is pronounced, resulting in the compacting force being partially neutralized so that limited densification is accomplished. Increasing the moisture content improved the lubrication and neutralizes the capillary action to increase densification. This action proceeds until a critical point is reached at which a maximum of the fines have been forced into the voids of the coarse grains and the bulk of the remaining voids are filled with water, leaving a minimum of residual air. Increase of the moisture beyond the critical point does not further lubricate the mass, but provides an excess of water in the voids that cannot be driven out by the compacting force. Consequently the densification is decreased as the moisture content is increased and a greater plasticity results.

The picture of the behavior of soils undergoing compaction just sketched may be clarified. It is the writer's opinion that not only are the fine grains forced into the voids of the coarse grains as compaction proceeds, but also that a general interlocking or readjustment takes place for all of the grains, both fine and coarse. Further, for the range of low moisture contents, the water films on the grains bind them together into series of soil arches or groups, like flocculated sediment or bulked sand. These arches or groups are arranged in much the same manner as the structure of marine clay, described by Dr. Casagrande.* When the films are thin at the low moisture contents, the surface tension is so great and the arching action so pronounced, that the compacting force does not cause their collapse. Limited densification

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* "Structure of Clay and Its Importance in Foundation Engineering," A. Casagrande, Boston Soc. C.E. Journal, Vol. XIX, No. 4, April, 1932

thus is accomplished. Accompanying an increase of the moisture content, the effective surface tension and the strength of the arches are reduced, resulting in increased densification. At the critical or "optimum" moisture content the supply of water just is sufficient to neutralize the surface tension; the full effect of the compacting force is utilized in readjusting the relative positions of the grains to form a dense mass. The description previously given for the subsequent action from this point needs no repetition.

The arching within the soil is illustrated by extending the usual compaction curve, Figure 1, to the point of zero moisture content. In this instance the soil was oven dried to start with. Accompanying the addition of the first moisture the arching or bulking is pronounced, as is indicated by the abrupt decrease in density or unit weight. As the moisture is further increased, the films of water on the grains become thicker and it is natural that the capillary action effecting the behavior of the grains and arching would diminish.

The basic principles demonstrated by Proctor and subsequently verified by wide practice have been extended, as illustrated by Mr. L. W. Hamilton,* to show that the density of a compacted soil varies not only with the moisture content but also with the amount of dynamic energy applied in the compacting operation, as shown by the family of typical curved, Figure 2. If the major factors of design - shear, consolidation, and permeability - vary with density, as they obviously should, then it follows that the density or compaction may be adjusted to meet the requirements of efficient design rather than be held to an arbitrary standard that disregards these factors which are so vital to the success of a structure.

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* "Compaction of Earth Embankments," L. W. Hamilton, Pro. Eighteenth Annual Meeting Highway Research Board, 1938.

The shear strength of a soil should be increased by the extent of compaction, for the grains are forced into intimate contact, interlocking due to densification is increased, and greater friction results. Further, the voids are reduced, particularly for cohesive materials, so that the inherent shear strength of the material is increased.

To specifically illustrate the effect of compaction on the shear strength of a soil, a series of direct shear tests was made of a clayey-silt (see Figure 3 for general characteristics) whose compaction characteristics are shown by Figure 4. It may be noted that the arbitrary standard of 25 blows was not followed in this instance, but the compacting force was varied from 6 to 60 blows. The specimens for the shear tests were at the optimum density and moisture content. The results of the shear tests and effect of compaction are shown by Figure 5. Examination of the first curve of the family shows the strength gained during the first stages of compaction is rapid but for the later stages the gain is small. In this instance, 62% of the strenght at the arbitrary standard is produced by 50% (12.5 blows) of the compaction; however, by doubling the compaction (50 blows) a gain of only 23% is achieved. The family of curves shows further that the point of maximum return varies as the normal load; for a normal load 0.5 tons/sq.ft. the point corresponds to about 25 blows; for a normal load 1.0 tons/sq.ft. the point is about 27.5 blows; and for a normal load 2.0 tons/sq.ft. the point corresponds to 52.5 blows, thus indicating that the criterion for optimum compaction should be adjusted to the conditions to which the materials in question are subjected.

The benefit, insofar as strength is concerned, of compaction in this instance is shown by considering the increase from the initial point (6 blows) which is considered to correspond to the degree of compaction produced in placement alone without any rolling or other attention to densification, such as dragline construction. A relatively limited amount of compaction (12.5 blows) produced by hauling equipment, causes the strength to increase practically 100% for a range of normal pressures from 0.5 to 1.5 tons/sq.ft and by developing a compaction equivalent to the arbitrary standard of 25 blows the increase of the strength amounts to 215 to 245% for the same range of loads. This fact has not been given the attention it so worthily deserves, in my opinion.

In planning the compaction of the soil for a structure, advantage can be taken in both the design and construction of the relationship between compaction and shear strength to make the most effective use of the materials available. The use of the arbitrary standard (equivalent to 25 blows in some instances and 40 in others) dictates one optimum moisture content and density. In the case of a levee system or low dams in a flat river valley where the ground water is near the surface, the available construction materials may be too wet when compared to the "optimum" indicated by those standards, In that event, the compaction of such materials, when considered only in the light of the one standard, may not only be both impractical and costly but also delay construction seriously in times like the present.

However, examination of the two families of curves, Figures 3 and 4, shown that by reducing the criterion of compaction an increase of allowable moisture content is experienced while an appreciable portion of the shear strength is retained. For structures in which flat slopes are dictated by considerations such as maintenance or weak foundations, the reduced compaction allowable would be particularly advantageous from the standpoint of reduced cost provided the reduction is not so great as to permit detrimental shrinkage of the structure to occur. The broadening of the limits on compaction and consequent increase of the allowable moisture combine to broaden the limits on the usable materials available with economies in costs as well as to minimize delays in construction.

One of the prime purposes of compacting the soil in an embankment is to place the material in the condition that will ultimately develop upon the completion of the structure. The compaction of soil causes the material to be preconsolidated. The equivalent static load which would produce the same densification corresponds to the preconsolidation load as defined by Casagrande* in 1936. It is apparent that the equivalent preconsolidation load would vary as the densification produced by the dynamic compacting energy and the swelling characteristics of the material involved. To illustrate this relationshipm consolidation tests were made of specimens of the clay-silt previously referred to which had been compacted by using a range of blows varying from 6 to 60 and the preconsolidation loads for each were determined. The results of the tests are shown graphically by Figure 6. For situations such as highway embankments, bases for runways or the like where it is desirable, for economic reasons or due to the limitations of time, for the finished grade to be maintained the control of compaction on the basis of preconsolidation produced thus appears feasible. For large embankments such as bases of dams in which will be developed pressures in excess of the preconsolidation produced by compaction some shrinkage or settlement should occur as consolidation proceeds under the pressure in excess of that built in. However, as indicated in this instance, the shrinkage is limited.

For cohesive soils, the hazard from over compaction and the consequent expansion upon saturation or shrinkage upon drying must be guarded against.

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* "Proceedings International Conference on Soil Mechanics and Foundation Engineering," Arthur Casagrande - Vol. III.

The treatment of this phase of the planning of the compaction of a soil, as given in the Report of Sub-Committee No. 2 of the American Society of Civil Engineers* is so complete that further remarks relative to the matter are not warranted at this time.

The third critical factor of design affected by the act of compaction is the permeability. It has generally been conceded that because the voids of a compacted material are reduced, and as these voids serve as pipes through which water in the form of seepage must pass, there would obviously be a marked reduction of the permeability accompanying compaction. This is true but not to the extent ordinarily visualized. The yardstick for measuring permeability is a very long one, extending from one hundred centimeters per second to one billionth of a centimeter per second or through a range of eleven digits, for the four basic types of soil used for engineering construction; namely, gravel, sand, silt, and clay. The normal limits of the change of permeability produced by compaction will seldom involve more than two of the eleven digits on the scale. The permeability of the silty-clay used to illustrate other principles in this paper ranged between 13.0×10^{-9} and 2.8×10^{-9} cm. per second for the corresponding compaction 6 to 60 blows. Numerically, this change is sizeable; however, upon application in the design of a dam or levee the difference is actual quantities of seepage involved would be so small as to be insignificant. To meet this pahse of the design for a structure it would be far better to resort to elements formed of materials impervious by nature rather than depending upon densification.

The compaction of soil as related to its stabilization should be direct regardless of the form of stabilization; that is, mechanical or chemical.

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* "Report of Sub-Committee No. 2 on Consolidation of Embankments," Proceedings of the American Society of Civil, Vol. 62, No. 8, Page 1491.

In the first case, that of mechanical stabilization, the densification resulting from compaction causes the interlocking of the grains so that the binding agent (cement or asphalt) is in close contact with the aggregate. This same practice is followed in work with concrete in that, by vibration of the mass, densification results which is reflected in an increase of strength. Research in this matter has already indicated this tendency for stabilized soils. The criteria for the compaction for stabilized soils will no doubt continue to be a more or less arbitrary matter until more definite methods for pavement design based upon subgrade behavior are developed. However, the heart of any stabilized surface or pavement is its foundation or subgrade, the compaction of which goes a long way toward the prolongation of the life of both.

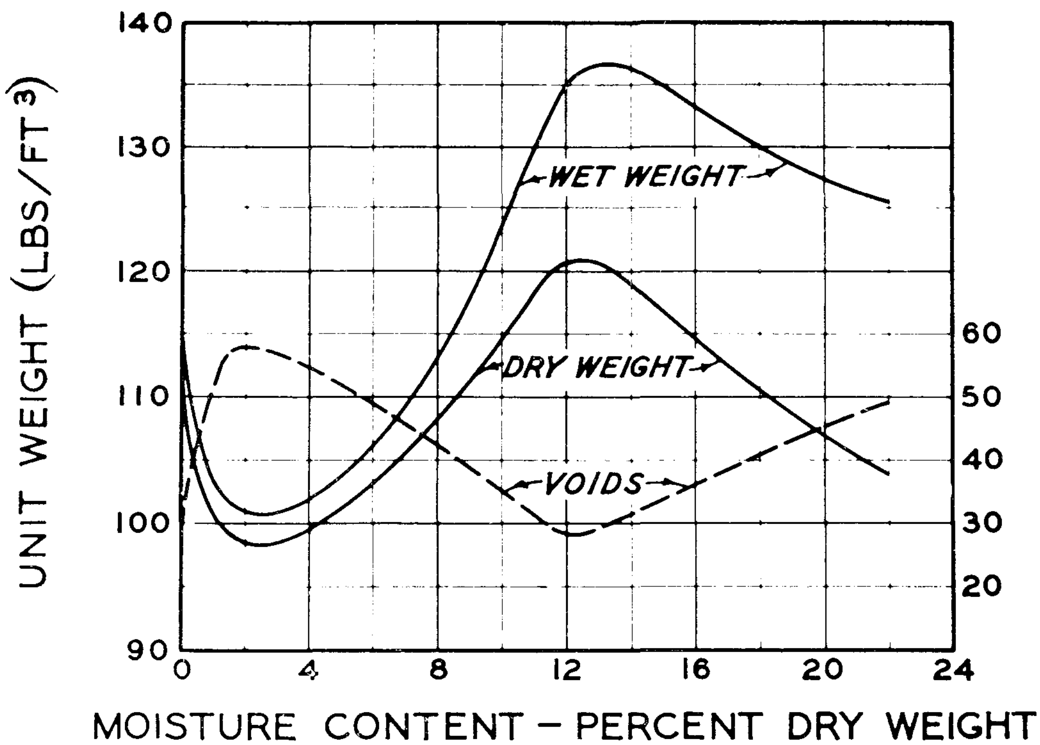


FIG. NO. 1 TYPICAL COMPACTION CURVE

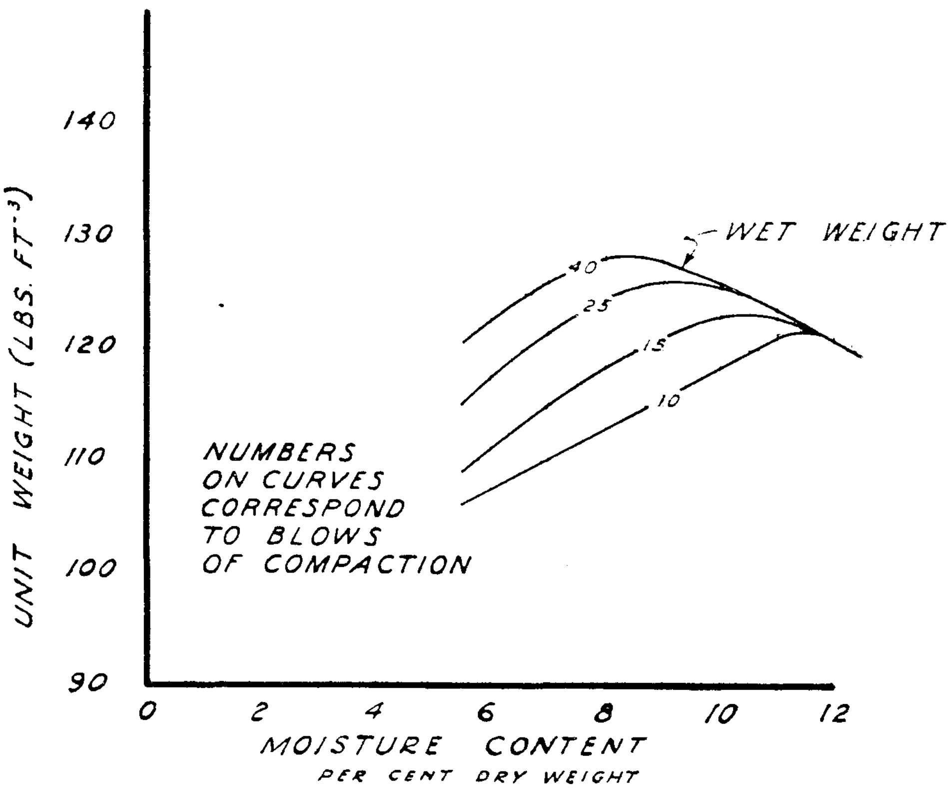
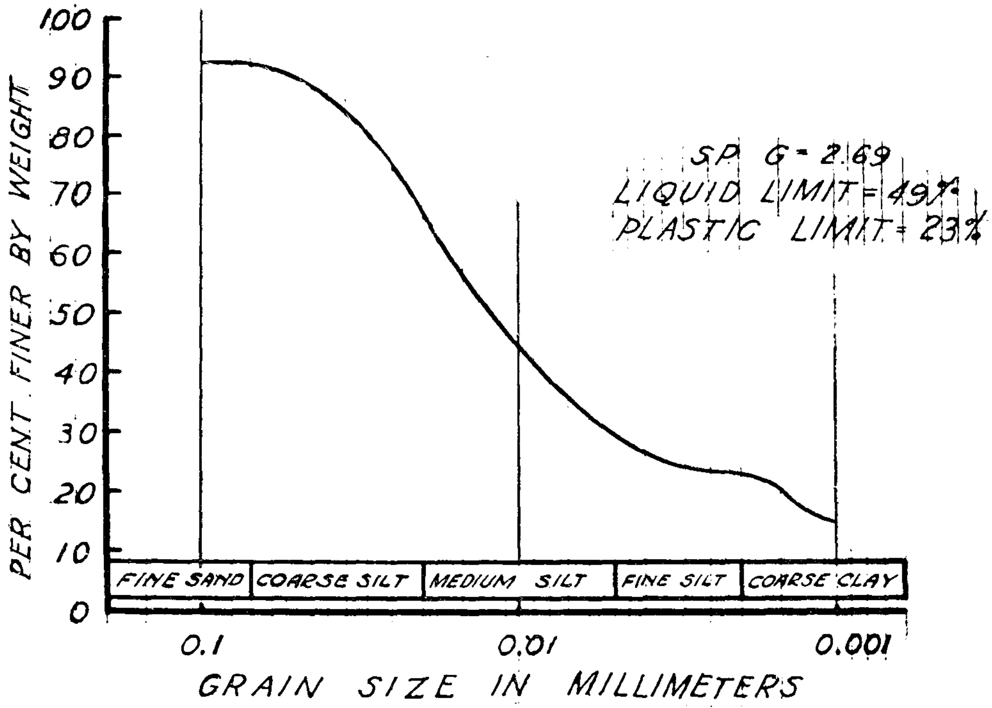


FIG. NO. 2 TYPICAL COMPACTION CURVES



MECHANICAL ANALYSIS DIAGRAM

FIG. NO. 3

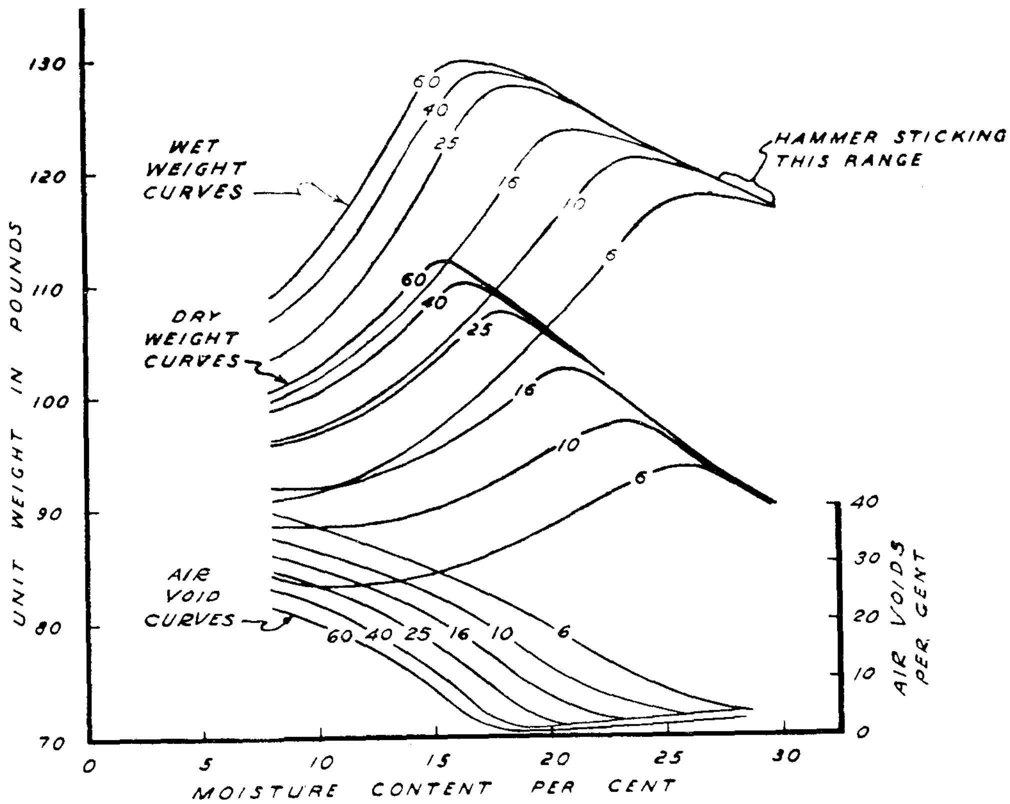
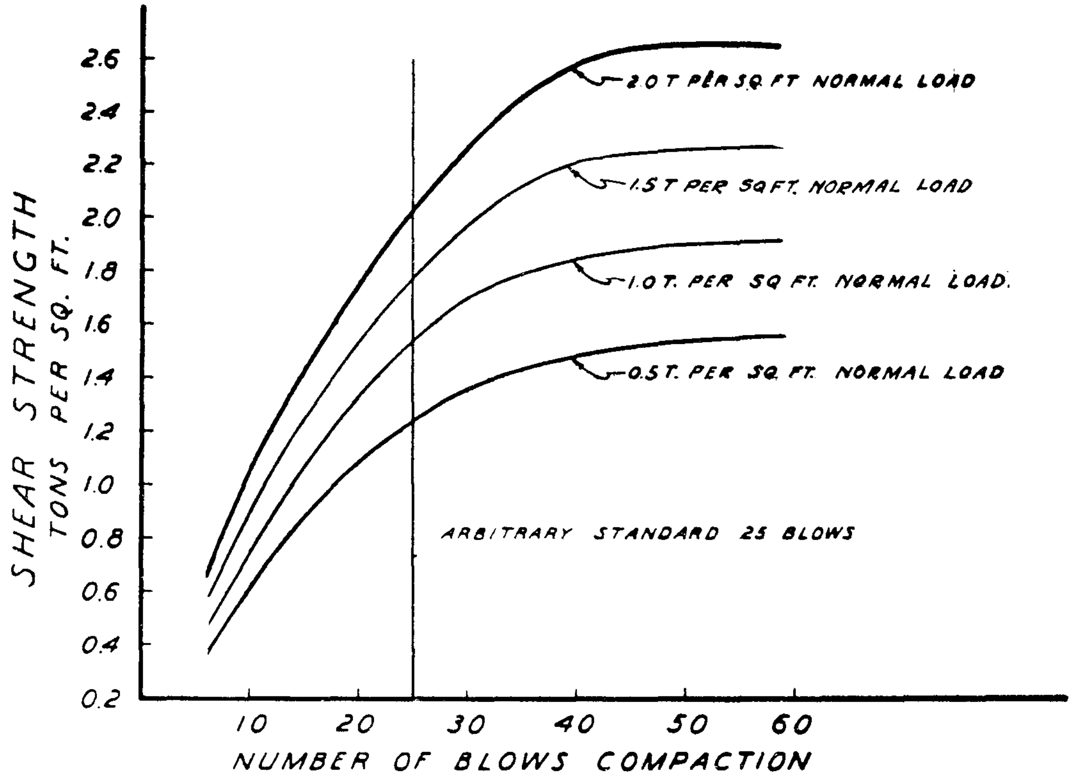


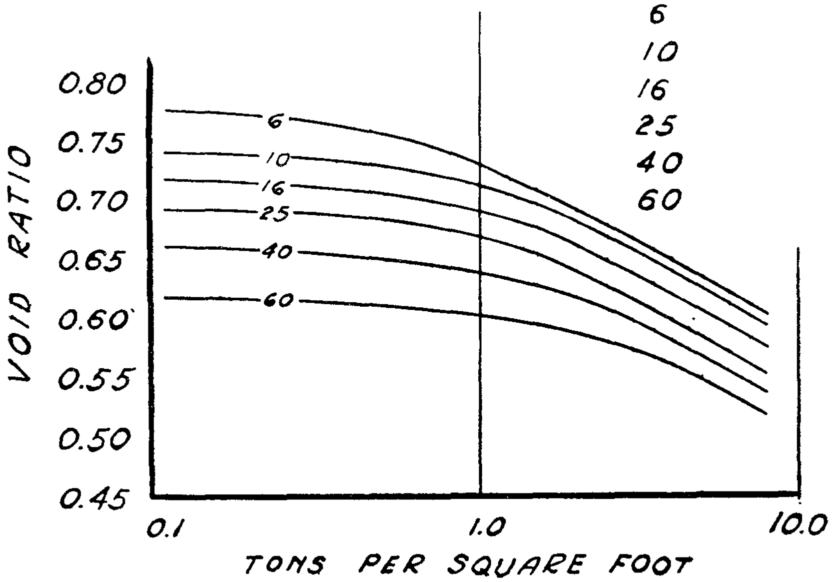
FIG. NO. 4 TYPICAL COMPACTION CURVES



RELATIONSHIP OF COMPACTION AND SHEAR STRENGTH

FIG. NO. 5

NUMBER BLOWS COMPACTION	PRECONSOLIDATION LOAD PRODUCED BY COMPARISON
6	0.82 T. SQ. FT.
10	1.30 " " "
16	1.65 " " "
25	1.90 " " "
40	2.50 " " "
60	3.20 " " "



PRESSURE CONSOLIDATION DIAGRAM

FIG. NO. 6