THE TREND OF SOIL TESTING

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The testing of soil samples under rigid conditions of control involves phenomena which explain the performance of soil in larger masses. Therefore, the material presented at the meetings of the Department of Soils Investigations is introduced by a brief résumé of soil test methods and the significance of test data.

Dominant Colloidal Phenomena. The electrical attraction which one material has for another causes some soil particles to become coated with air and others with moisture (1).[1]

Clays retain their plasticity when mixed with sand or other nonplastic material due to adsorption of clay by the sand. The clay is not distributed uniformly through the pores or interstices of the coarse particles, but most of it coats the nonplastic material permitting many of the pores to remain unoccupied even if there is more than enough clay to fill them.

In a similar manner, water coats the solid particles of wet soil permitting air to remain in the interstices between the films. See Figure 1 and reference (2). Only at moisture contents somewhere above the liquid limit does water displace the air and completely saturate soils. Such water has all the viscosities within the range from free water to solidified water or ice. See Figure 2 and reference (3).

As a result, size determinations by sedimentation methods disclose not sizes of the solids but of the solid particles encased in film; moisture capacity depends primarily upon surface areas of the particles and thicknesses of the films which coat them; effective volumes of fractions of soil become the combined volumes of solids and their films in the fraction; and the stability of soil at constant pore space depends upon the ratio of free to film moisture.

Films 0.000,011 in. thick add no appreciable size to sand grains say 0.1 in. in diameter and therefore film moisture affects the performance of sand fractions but little. Particles with decreasing diameters in the minus No. 200 sieve material, are increasingly enlarged to apparent diameters by their viscous coatings until at colloidal sizes volumes of the film moisture become so enormously great in comparison with the volumes of the solids that properties of the adsorbed moisture control performances of the soil.

The surface areas of particles which comprise a soil, depend upon their size and shape. If the volume of soil and shape of particle is kept the same, the surface area increases as sizes of the particles are decreased. To illustrate consider the surface areas of cubic feet of spheres of different diameters as follows: Spheres 1 mm. in diameter (sand size), about 1,000 sq. ft.; spheres 0.02 mm. in diameter (silt size), about 50,000 sq. ft.; and spheres 0.001 mm. in diameter (colloid size), about 1,000,000 sq. ft.

If volumes of soil and sizes of particles are kept constant, the surface areas increase as change of shape is made from the rounded or bulky grains of sands and silts to the scale-like and irregularly shaped clay, micaceous, diatomaceous or peaty particles.

Thicknesses of films on particles of equal size depend on their chemical character (4). The chemical composition of soil is indicated by the ratio of silica to the combined iron and aluminum oxides. This is termed the silica-sesquioxide ratio and is designated by the

* Since July 1, 1939 Public Roads Administration.

[1] Figures in parentheses refer to list of references at end.

symbol, $\frac{SiO_2}{R_2O_3}$. As a generalization, it can be said that films on clay particles of equal size become thicker as the silica-sesquioxide ratio becomes larger.

Figure 3 illustrates the combined effect of size, shape, and chemical composition

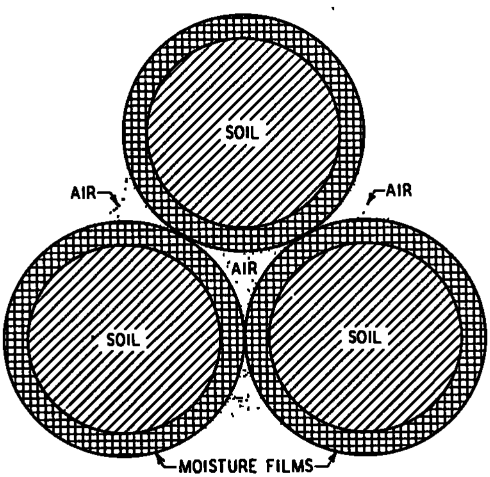

Figure 1

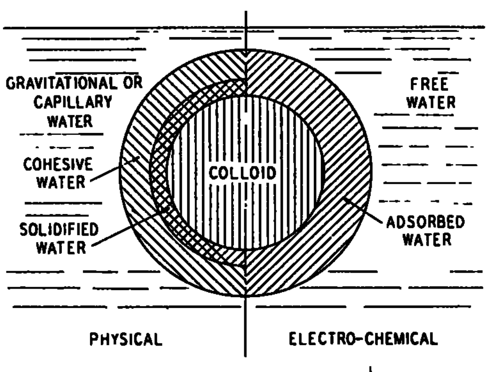

Figure 2. Colloid Encased in Film of Adsorbed Water and Suspended in Free Water

of particles on the relative volumes of solids and pore water in sediments of different materials.

The sands and rock powders represent the incompressible granular materials which contain but little film water and consequently have low moisture contents.

The silty soils, clay soils, muck, and colloids represent compressible colloidal materials which contain large amounts of film moisture and consequently have large moisture contents.

Apparent Volume of Soil Fractions. To illustrate the effect of film volume on

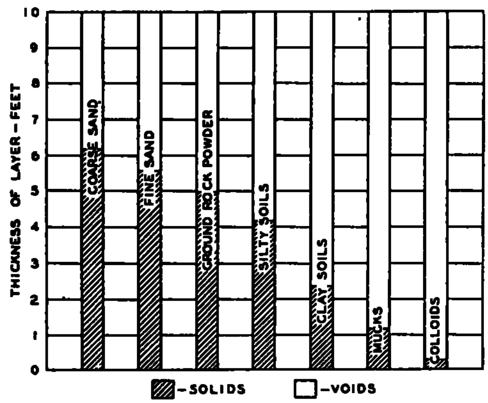

Figure 3. Relative Volumes of Solids and Water in 24-hour Sediments of Typical Materials

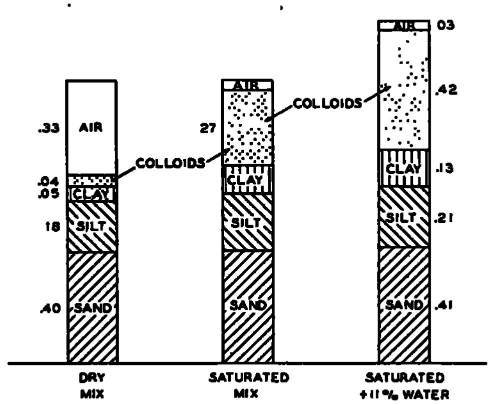

Figure 4. Fractions—Cubic Foot per Cubic Foot of Dry Mix

volumes of the individual fractions which comprise composite soil let it be assumed that the fractions of the properly graded sand-clay mixture, illustrated in Figure 4, consist of spherical particles and that the film moisture is distributed evenly over their surfaces. In the oven-dried state the relative volumes per cubic foot of

mixture will be as shown in the column to the left (dry-mix) If the original cubic foot of material were immersed in water and not allowed to swell, the effective volumes of the fractions would be increased as shown in the middle diagram If the moisture content were further increased to the field moisture equivalent (plus 11 percent moisture in this case), the volumes of the fractions would be further increased to those shown in the last diagram While, as can be seen, the volume of the sand fraction increased from 0 40 cubic foot to but 0 41 cubic foot, the volume of the colloidal fraction increased from 0 04 cubic foot to 0 42 cubic foot (6a)

Effect of Film Moisture on the Stability of Soil Decreasing the ratio of free to film moisture increases the stability of soil This ratio in any particular soil may be decreased as follows

1 At constant pressure and temperature, by reduction of the moisture content

2 At constant pressure, temperature and moisture content, by replacement of the natural ions with others which have greater attraction for water

3 At constant pressure and moisture content, by decrease of temperature

4 At constant temperature and moisture content, by decreasing the pressure

The gradual increase of stability as soil sediments dry out illustrates the moisture reduction effect The abrupt change from the plastic to the semi-solid state at the plastic limit represents the type of "critical" moisture content met so commonly in soil investigations

In *Public Roads*, July 1931, it was shown that when the moisture content of a soil is increased above the "critical" moisture content deformations of loaded cylinders increase at a very much greater rate than for similar moisture increases below the "critical"

Effect of Electrolytes The potassium ion represents those which have relatively small attraction for water. It has a true diameter of 0 000,000,008 in and in suspension becomes associated with 16 molecules of water, thus attaining an apparent diameter of 0 000,000,02 in See Figure 5 and reference (5) The lithium ion represents those which have great attraction for water It has a true diameter of 0 000,000,006 in and be-

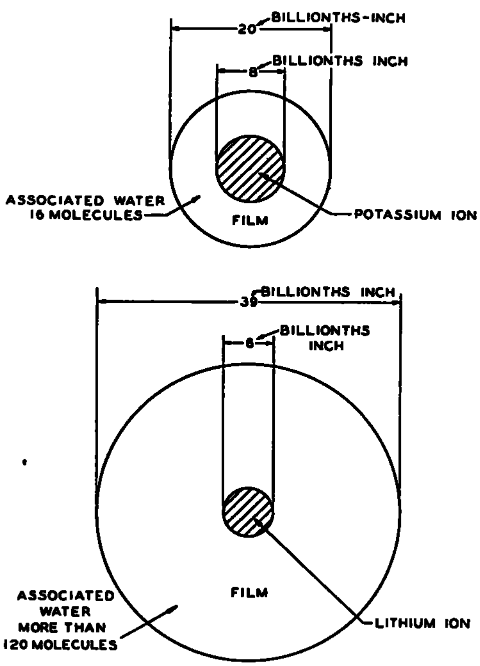

Figure 5

comes associated with more than 120 molecules of water, thus attaining an apparent diameter of 0 000,000,039 in Between potassium and lithium, other of the more common metallic ions can be arranged in the order of their attraction for water

Consider for instance the Hagerstown soil investigated by Dr Hans Winterkorn (4). At a moisture content of 28 percent it can support a load of 1 12 tons per sq ft with no further settlement due to consolidation Saturation by aluminum ions which have small

attraction for water causes its supporting value to drop to 0 78 ton per sq ft , and by the sodium ions which have great affinity for water to increase to 2 73 tons per sq ft

Since flow of moisture is through spaces between films and not between the particles as such change of film thickness changes at the same time the permeability of soil which controls the speed of consolidation Thus, the coefficient of permeability which for the natural soil at a moisture content of 28 percent was 5 55 cm per sec (10^{-8}), became 6 45 cm per sec (10^{-8}) when the soil was saturated with aluminum ions and 0 45 cm per sec (10^{-8}) when saturated with the sodium ions Use of an electrolyte with thin films to facilitate compaction of soil, which may be leached out later should according to the above be beneficial

Increase of temperature reduces thicknesses of films and therefore the stability of soil It has been shown that the stability of compacted samples, without change of total moisture content, was reduced from 1,400 to 960 lb per sq in when the temperature of the sample was raised from 42°F to 130°F (2) Also, the permeability of compacted samples was reduced by lowered temperatures far more than could be explained by increased viscosity of water (6) These facts explain the abrupt drop in the stability of subgrades and embankments occasioned by sudden temperature rises in the spring of the year

Increasing the pressure likewise reduces thickness of films and therefore the stability of soil It has been shown by shear tests that for every moisture content there is a normal pressure at which the given moisture content becomes the "critical "

Internal Forces Retain High Density of Soil The binding of clay particles into enormously strong masses after all the water, productive of capillary pressure, has been evaporated from thoroughly dried soil samples represents the great tenacity of practically solid moisture films Under such conditions granular soils fall apart

Colloidal cementing action holds the particles of soil at low moisture contents together by forces exerted within the soil, which accounts largely for the high density of compacted and consolidated samples

Many soil strata consolidated to low moisture contents by enormous overburdens retain the high compaction thus attained after geological processes have greatly reduced the overburdens Customarily samples of undisturbed soil in the laboratory retain much of the compaction which they had in situ deep in the ground

Samples consolidated in the laboratory likewise retain high density after the consolidating pressure has been removed and all external pressure due to surface tension has been eliminated by complete immersion This is the effect which facilitates interpretation of test data and furnishes vast possibilities in the field of stabilization

Stability of Soil Determined by Test It is the purpose of soil tests to disclose in some manner the effect of moisture and air content of samples on the bearing value and stability of soil masses

The four general types of physical tests are

1 Consolidation
2 Shear
3 Compaction
4 Indicator

The effect of temperature is eliminated in the laboratory by performing the tests under approximately constant temperature conditions

The ratio of pores to solids in the mass is expressed by either the voids ratio or the moisture content Voids ratio, designated by the symbol e, is defined as the ratio of the volume of pores to the volume of solids in the soil mass

The moisture content, w, is defined as the weight of moisture expressed as a percentage of the weight of the solids in a soil mass.

In soil with pores completely filled with water, the voids ratio, e, and the moisture content, w, are related according to the following expressions:

$$e = \frac{w \times SG}{100} \qquad (1)$$

and

$$w = \frac{e \times 100}{SG} \qquad (2)$$

in which SG = specific gravity of the soil solids. See Table 1.

TABLE 1

SPECIFIC GRAVITIES, VOIDS RATIOS, AND MOISTURE CONTENTS OF MATERIALS SHOWN IN FIGURE 3

Material	Specific gravity	Voids ratio, e	Moisture content, w
			percent
Coarse sand	2.65	0.6	23
Fine sand.........	2.65	0.8	30
Rock powder	2.67	1.0	37
Silty soil..........	2.70	1.4	52
Clay soil..........	2.76	3.5	127
Muck..........	2.71	7.2	266
Colloids...........	2.83	30.0	1060

The term saturated soil as used hereafter refers to soil which has been submerged in water, although all the air has not been displaced.

THE CONSOLIDATION TEST

Essential features of the Terzaghi consolidation test are shown in Figure 6. Samples of the compressible soil are encased in the apparatus between two porous stone filters, with lateral flow excluded. Static loads are applied to the piston in several increments, totaling somewhat more than the maximum unit support required by the particular foundation. The sample is permitted to attain equilibrium under each increment before the next is applied.

Water squeezed from the soil sample passes through the stone filters and escapes from the overflow orifices, a and b. This simulates a compressible stratum of soil sandwiched between two porous strata, except that in the field the possibility of lateral flow is not excluded.

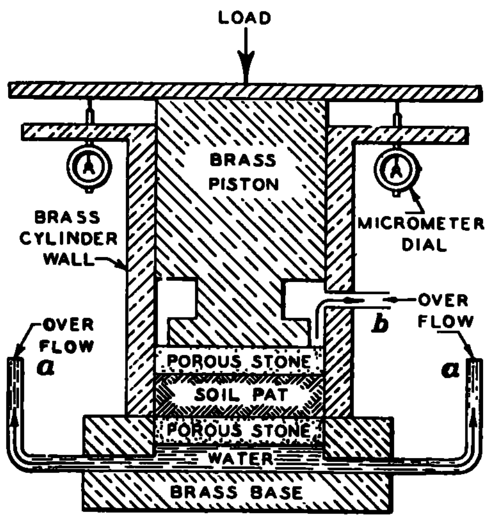

Figure 6. Essential Elements of Terzaghi Consolidation Device

The test furnishes the data shown in Figure 7. The pressure-moisture curve shows the moisture content, w, to which the sample will be consolidated by a given pressure, p. The time-consolidation curve[2] shows the speed at which the test sample consolidates.

For example, according to the pressure-moisture curve (Fig. 7) a pressure of 0.5 ton per sq. ft. will compress the sample to a moisture content of 79.5 percent. According to the time-consolidation

[2] A time-consolidation curve is obtained for each load increment. The one shown in Figure 7 is the average obtained from the three increments of loading used. Investigations have indicated that no appreciable error is introduced by making use of the average curve for all increments of load.

curve, 28 percent of the ultimate consolidation will occur in the laboratory sample in one minute.

From the moisture content-pressure curve of Figure 7, additional data which will be referred to later in this report may be obtained as shown in Table 2.

exerts pressure just equal to that produced by the excavated earth.

Depending upon a number of variables, such soil, if thus subjected to additional pressure, may consolidate vertically, displace laterally, or both; and thereby cause piers to fail by settlement, and abut-

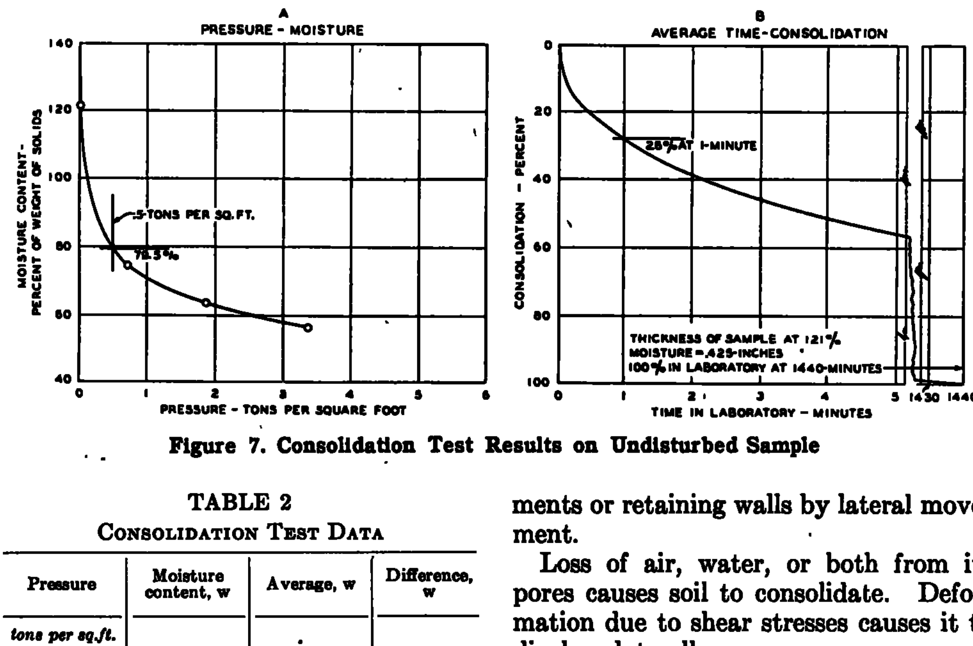

Figure 7. Consolidation Test Results on Undisturbed Sample

TABLE 2
CONSOLIDATION TEST DATA

Pressure	Moisture content, w	Average, w	Difference, w
tons per sq.ft.			
1.1	69.8	67.1	
1.7	64.4		
			9.2
3.1	56.6	57.9	
2.7	59.2		

ments or retaining walls by lateral movement.

Loss of air, water, or both from its pores causes soil to consolidate. Deformation due to shear stresses causes it to displace laterally.

The theory of consolidation, including pressure distribution, has been discussed at length by Palmer and Barber in *Public Roads* (7). Consolidation of saturated soil, which only is considered in the theory, is due entirely to loss of water.

To illustrate the phenomenon of consolidation, consider a fully saturated sample of soil which has been consolidated to equilibrium under a pressure p_1. Next let it be assumed that the sample is now so confined that no more water can escape from it. Under this condition let the pressure be increased from p_1 to p_2. The tendency of this increase of pressure, p_2-p_1, is to squeeze out an additional amount of water and thus bring the grains closer together. But since the water is

Application of Consolidation Test Data Illustrated

Consider the soil profile illustrated in Figure 8. If sufficiently old, the layers have attained all the consolidation possible due to weight of soil. In such deposits, every particle has a fixed position relative to its neighbors. A bridge pier constructed as in Figure 8 will destroy the equilibrium of the deposit and thus cause the soil particles to seek new positions relative to each other unless the pier

prevented from escaping from the soil, it in turn must carry the new increment of load and in consequence must exist under a hydrostatic pressure equivalent to the new increment.

To illustrate, reference is made to Figure 7 and the resulting consolidation test data in Table 2. At a pressure of 1.1 tons per sq. ft, the sample was consolidated to equilibrium at a moisture content of 69.8 percent. If, now, further escape of water were prevented and the pressure increased to 3.1 tons per sq. ft., a hydrostatic pressure equal to 2.0 tons per sq. ft. would be produced within the sample. If, under similar conditions of

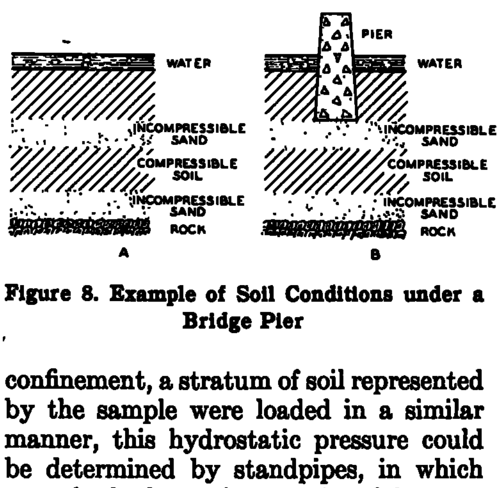

Figure 8. Example of Soil Conditions under a Bridge Pier

confinement, a stratum of soil represented by the sample were loaded in a similar manner, this hydrostatic pressure could be determined by standpipes, in which case the hydrostatic pressure of 2.0 tons per sq. ft. would be sufficient to maintain an elevation of water in the pipes 64.1 feet above the ground water elevation.

However, in the test sample and in soil formations to which the theory of consolidation is applicable, water can escape from one or both faces of the compressible soil. Under these conditions the hydrostatic pressure within the layer is, at the instant of loading, the same as for the totally confined compressible material noted above. As the water escapes, this pressure is diminished until it becomes 0, when equilibrium is attained under the new pressure. If, as assumed, the total pressure of p_2 equals 3.1 tons per sq. ft., equilibrium will be attained when the moisture content of the soil has been reduced to 56.6 percent.

How the speed of embankment construction is regulated by the hydrostatic pressure in the foundation soil is described in reports by T. K. Huizinga and O. J. Porter, both of which are included in these proceedings.[3]

Consolidating Pressure Is Only Part of the Pressure which Acts on Soil

The total pressure on the soil (Fig. 8) is comprised of two parts: that due to weight of soil within the layers and that produced by the pier. On the assumption of no further consolidation due to weight of soil, the vertical pressure it produces is considered "inactive." That produced by the pier in excess of inactive pressure is the "consolidating" pressure[4] because it alone can effect further consolidation of compressible strata.

Inactive Pressure. Pressure at any point within the saturated layers prior to the pier construction is equal to the buoyed weight of soil above the point.

The buoyed weight of a cubic foot of saturated soil is the weight of a cubic foot of the mass, solids plus water, minus the weight of a cubic foot of water. The additional head of water above the saturated soil is disregarded, for obviously the depths through which soil particles settle can have no effect on the densities of resulting deposits.

The weight per cubic foot of the incompressible sand layers is likely to be fairly constant. Consequently the verti-

[3] See pages 81 and 129.

[4] The term "active earth pressure" has a particular significance relative to earth pressures considered in the design of retaining walls. Therefore the term "consolidating" seems to be an advisable substitute for the term "active" which has been used also to indicate the pressures productive of consolidation.

cal pressures within can then be expected to increase proportionately with the depth of the layer, making the relation of pressure to depth a linear one.

Because of higher moisture contents, a compressible material is generally lighter than an incompressible one and its weight per cubic foot becomes greater as the pressure on it increases and consequently as the depth of the material increases. Therefore, the relation of vertical pressure to depth in the compressible layer is not linear but curved as discussed by F. A. Robeson in *Public Roads* (8). This makes the diagram of inactive pressure have the form of the crosshatched area in Figure 9B.

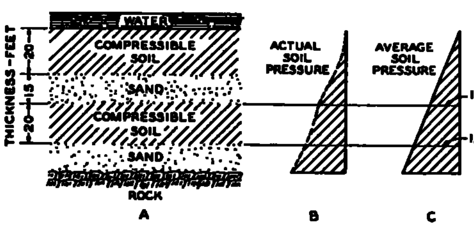

Figure 9. Vertical Pressure Distribution in Soil

As an approximation, however, the buoyed weight of soil in the deposit is assumed to be constant and equal to 60 lb. per cu. ft. thus making the diagram of inactive vertical pressure triangular as shown in Figure 9C.

In such case the inactive pressure at the top of the lower compressible layer becomes—

$35 \times 60 = 2{,}100$ lb. per sq. ft. or approximately 1.1 tons per sq. ft.;

and at the bottom of the lower compressible layer—

$55 \times 60 = 3{,}300$ lb. per sq. ft. or approximately 1.7 tons per sq. ft.

The consolidating pressure varies with the shape of the pier. If, for the purpose of demonstration, the pier is assumed to be 20 ft. wide at its base and long enough to constitute a strip load, the pressures p_z, along the center axis (c–c), Figure 10, are computed from the Boussinesq formula (7).

$$p_z = \frac{p}{\pi}(a + \sin a) \qquad (3)$$

in which

p = pressure in excess of the inactive pressure exerted by the pier at its base, and

a = twice the angle in radians between the vertical and a line from the side edge of the base to any point along the center line at a distance z below the base of the pier.

If p is taken at 3 tons per sq. ft., the pressure exerted along the center axis (c–c) at the top of the compressible layer is computed as follows:

$$\text{Tangent } \frac{a}{2} \text{ (see Fig. 10)} = \frac{10}{15} = 0.667$$

$$\text{Then } a = 1.176 \text{ radians } (67.4°)$$
$$\sin a = 0.923$$

By substitution

$$p_{(z=15)} = \frac{3}{3.142}(1.176 + 0.923)$$

$$= 2.005 \text{ tons per sq. ft.}$$

In the same manner, the pressure at the intersection of c–c and the bottom of the compressible layer is found to be 1.035 tons per sq. ft. The vertical pressures along the axis c–c at other distances below the pier are shown by the curved line of Figure 10.

The total pressures, consolidating and inactive, on the compressible stratum as indicated by the diagrams in Figures 9C and 10 are shown in Figure 11.

Amount of Consolidation. If the stratum 20 ft. thick has the compression characteristics of the data in Figure 7, the moisture content of the layer at its upper boundary, at an inactive pressure

of 1.1 tons per sq. ft. (see consolidation test data, Table 2), equals 69.8 percent; at its lower boundary, at a pressure of 1.7 tons per sq. ft. 64.4 percent. The average for the layer is taken as the average of the moisture contents at the boundaries, or 67.1 percent.

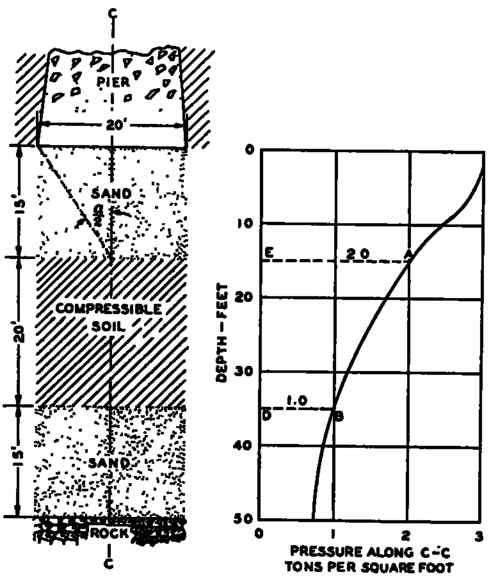

Figure 10. Pressure Distribution under a Bridge Pier

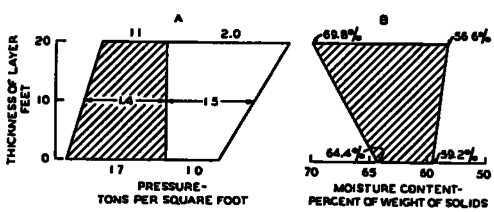

Figure 11. Vertical Pressures and Moisture Contents, Compressible Layer

The total vertical pressure, due to both pier and soil, would be 3.1 tons per sq. ft. at the upper boundary and 2.7 tons per sq. ft. at the lower boundary, with corresponding moisture contents of 56.6 percent and 59.2 percent, respectively, or an average of 57.9 percent.

Consequently, it is considered that the consolidating pressure produced by the pier will eventually reduce the average moisture content of the compressible undersoil from 67.1 to 57.9 percent.

If the solids have a specific gravity of 2.65, a soil mass at a moisture content of 67.1 percent according to formula 1, will contain $0.671 \times 2.65 = 1.78$ times as much water as solids by volume. Therefore, a column 20 ft. high (see Fig. 12) will consist of 7.19 ft. of solids $\left(\frac{20}{1 + 1.78}\right)$ and 12.81 ft. of water. When the moisture content is reduced to 57.9 percent, the column will contain 7.19 ft. of solids, but only 11.05 ft. of water $\left(\frac{0.579}{0.671} \times 12.81\right)$. The difference of 1.76 ft. represents, according to theory, the ultimate distance the pier could settle without lateral displacement of the undersoil.

Speed of Consolidation. Samples and strata of similar soil with two permeable faces, and consolidated by equal pressures, attain an equal percentage of consolidation at times proportional to the squares of their thicknesses.

The time of settlement of any layer of soil with two permeable boundaries may be computed by means of the expression

$$t_D = \frac{t_d \times D^2}{d^2} \qquad (4)$$

in which

t_D = time required for consolidation of the stratum

t_d = time required for consolidation of the soil sample

D = thickness of the stratum, and

d = thickness of the soil sample.

The soil sample (see Fig. 7) had a thickness of 0.425 in. at a moisture content of 121 percent. Consequently it contained $1.21 \times 2.65 = 3.21$ times as much water as solids by volume. Therefore, of the 0.425 in. the solids comprised 0.101 in. — $\frac{0.425}{1 + 3.21}$ —, and the water

0.324 in. When the moisture content was reduced to 67.1 percent, the thickness of the water was reduced to

$$\frac{67.1}{121.0} \times 0.324 = 0.18 \text{ in.},$$

thus making the total thickness 0.28 in. or $\frac{0.28}{12}$ ft.

This is considered as the thickness of the laboratory sample when, at an initial moisture content of 67.1 percent, it would consolidate according to the time-consolidation curve Figure 7, 28 percent in one minute. The time required for the soil stratum 20 ft. thick to settle 28 percent of the total of 1.76 ft. or 0.49 ft. then equals

$$\frac{20^2}{\left[\frac{0.28}{12}\right]^2} \times 1 \text{ min.} = 734{,}000 \text{ min. or } 510 \text{ days}$$

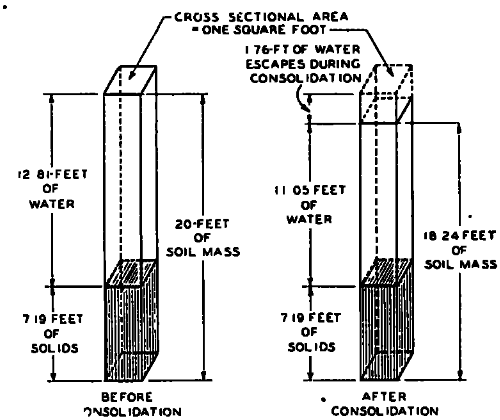

Figure 12. Diagram of Results of Loading on Compressible Soil Stratum

In a similar manner the time required for any other amount of settlement may be determined.

A soil stratum with but one drainage face will, if the pressures at the top and the bottom are equal, require four times as long to consolidate as a similar stratum with two drainage faces.

Strata with but one drainage face, and which are subjected to pressures which are different at the top and bottom faces, consolidate at speeds which vary as discussed in the report in *Public Roads* (7). In such cases the times, as determined for similar strata with two permeable faces (formula 4) are first multiplied by four and then further qualified by coefficients which have been published in tabular form elsewhere (9).

For strata with impermeable or but partly permeable faces, procedures for determining the speed of consolidation are not yet available.

THEORY OF SHEAR TESTS

The purpose of this discussion is to present clear-cut, theoretical approaches to the evaluation of shear strength of soil. The development is based upon wide experiences of laboratories and every illustration is devised from published data. We begin with a block of rock or metal which, if resting on a horizontal plane, has no tendency to slide.

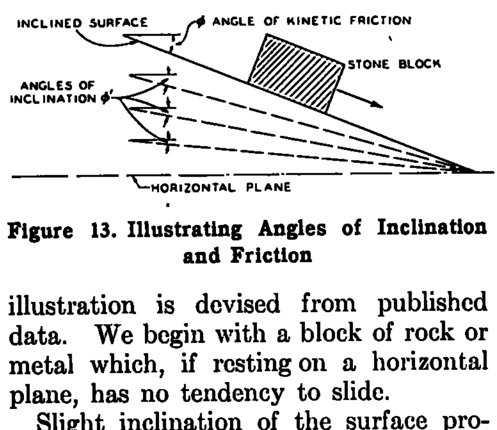

Figure 13. Illustrating Angles of Inclination and Friction

Slight inclination of the surface produces such tendency, and when the inclination becomes great enough the body will, if given a start, slide down the surface as indicated in Figure 13. The angle of inclination at which the body will move at a relatively slow but constant speed is termed the angle of kinetic friction and is designated by the symbol ϕ. Angles of inclination less than ϕ are designated by ϕ'.[5] Similarly, if the sur-

[5] Angles of inclination have been termed also angles of "obliquity."

face remains horizontal, a horizontal force must be applied to the body to make it slide. Such horizontal force per unit of the area of contact between the block and surface is termed kinetic friction and designated by the symbol s.

According to generally accepted conceptions, as stated by Anderson (10), s is directly proportional to n, the force per unit of area in contact which presses the surfaces together, and is independent of both the areas in contact and the rate of sliding. Its magnitude is expressed by the formula:

$$s = n \tan \phi$$

in which ϕ = angle of kinetic friction whose tangent $= \frac{s}{n}$.

The force per unit area required to start the sliding and termed the *static* friction may be larger than kinetic friction.

Any cement which binds the surfaces together and has a resistance per unit area, c, will, if rigid like a hard glue, affect the resistance required to start the block sliding. Plastic cements like bituminous and soil binders have somewhat different effects as follows: (a) they may influence, at least for some distance of block sliding, the force required to maintain as well as to start the sliding; (b) their full resistance is not developed until some sliding has occurred; and (c) their c may be varied by the rate at which the block is slid.[6]

The magnitude of s at the instant of maximum c is expressed by the formula

$$s = c + n \tan \phi$$

If a number of plates, arranged as shown in Figure 14A, and fixed at the bottom, have their positions changed at a constant rate by a force per unit area s applied to the top plate, s becomes the shear stress; d, as shown, the deformation; ϕ, the angle of internal friction; and c, if present, the cohesion of the system. For convenience, all values of cohesion less than the maximum are designated by the symbol c′.

The relation of s to deformation may then be shown as in Figure 15, which also shows the effect of static friction. The static friction receives no further mention in this report.

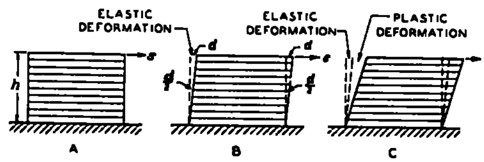

Figure 14. Deformations Produced by Shear

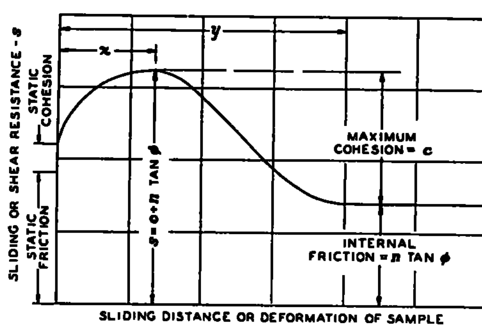

Figure 15. Illustration of Friction, Cohesion and Shear

The distance x, Figure 15, representing the movement of one surface on the other required to develop maximum c, multiplied by the number of plate surfaces in contact, Figure 14A, equals the deformation of the system at maximum s; and y multiplied by the same number equals the deformation at which all effects of cohesion have been spent.

Therefore, the thinner the plates in the same height or the greater the height of the system having plates of the same thickness, the larger will these deformations be.

[6] Change of rate of load application has been found to cause values of c but not ϕ of soil bituminous mixtures to vary also. See reference (11).

The sketches of Figure 14 may illustrate also the phenomenon of shear in homogeneous materials of high molecular cohesion like metal, which it may be considered, according to Anderson (10), are comprised of horizontal layers of molecules. If the sample held fixed at the bottom is deformed from the rectangular (Fig. 14A) to the parallelogram

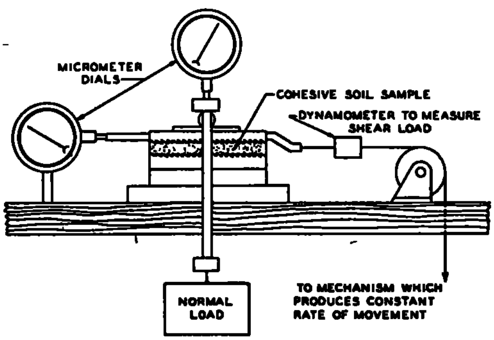

Figure 16. Essential Features of Shear Test

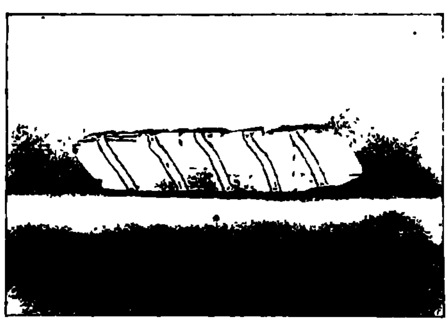

Figure 16A. Deformed Shear Test Specimen

cross sections (Figs. 14B and 14C) by a shear stress, s, applied at the top, each layer of molecules will be shifted to the right a distance which, if d at the top, will be d/2 half way down, and so on to 0 at the bottom; and thereby the molecules of each layer have a position slightly changed with respect to those in adjacent layers.

If not stressed beyond the elastic limit, the metal sample will resume its rectangular cross section upon the removal of s; or, if stressed continuously beyond the elastic limit, will attain permanent deformation and rupture finally if s is sufficiently increased. The deformations at which ductile steel fails may be many times as great as those at which brittle metal fails.

If stressed as indicated in Figure 16, soil shears in similar manner; sand more abruptly like brittle metal; and clay after considerable deformation, like a ductile metal. Soil has sizable particles instead of molecules comprising its layers and

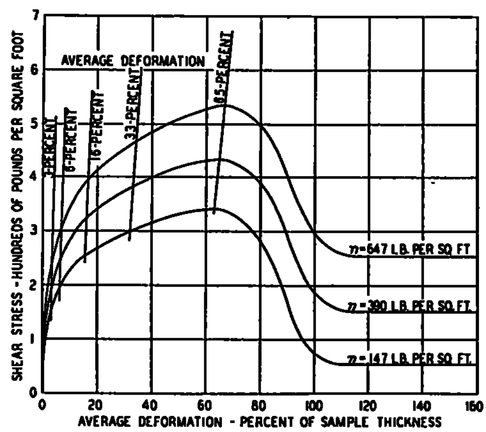

Figure 17. Illustrating Stress—Strain Relation, Shear Tests

considerably less and more variable strength. A deformed sample is shown in Figure 16A.

The smaller the soil particles, making in turn the thinner the layers which slide, or the more plastic the soil, the larger will be the deformations (x and y, Fig. 15) through which the cohesion contributes to shear strength.

Usually some readjustment of soil particles must take place before any internal friction and cohesion is developed and, as a result, shear resistance-deformation curves of soil samples may have the typical characters shown in Figure 17 (12, 13, 14).

At maximum s, the sample with

n = 147 lb. per sq. ft. deformed 63 percent of its thickness; that with n = 390 lb. per sq. ft., 65 percent; and the third

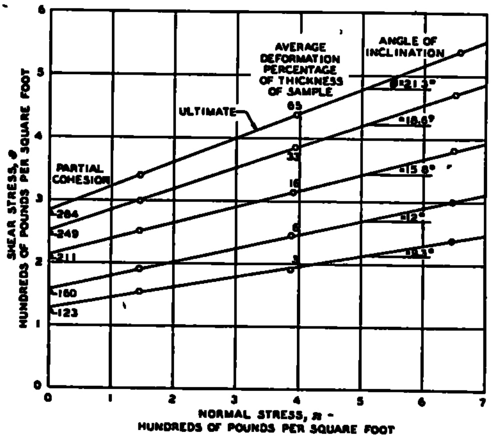

Figure 18. The Relation of Normal and Shear Stresses for Variations in the Deformations

18 (15) whose angle with the horizontal is the angle of internal friction of the sample, ϕ, which equals 21.3 deg. in this case, and whose s, at n = 0, is the cohesion c, which equals 284 lb. per sq. ft.

By making similar use of the shear resistance at deformations of 33 percent, 16 percent, 6 percent, and 3 percent, which are also shown in Figure 17, the relations of s to n at average deformations less than those at maximum, s, may be shown as by the other four lines in Figure 18.

As a next step, the data of Figure 18 may be used to show the relation of internal friction and cohesion to sample deformation as in Figure 19.

Similar treatment of shear test data obtained by means of increment loading has been presented before the American Society for Testing Materials (16).

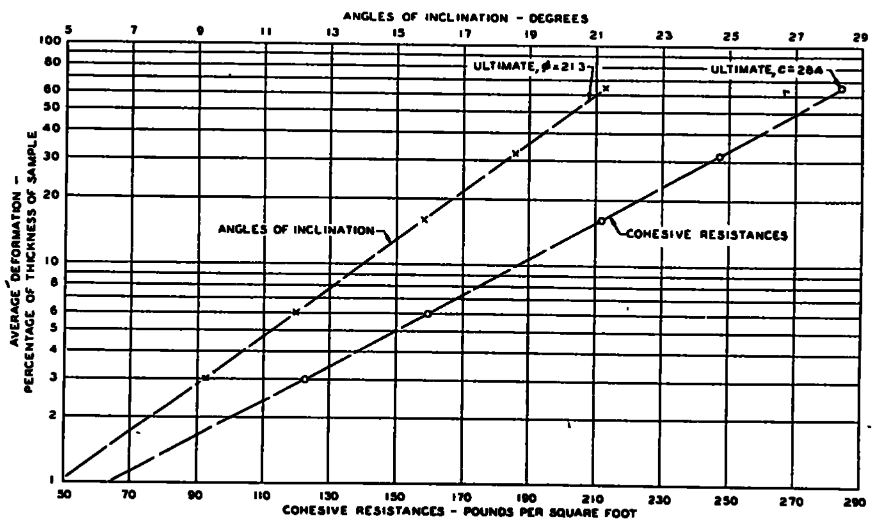

Figure 19. Relation of Sample Deformation to Shear Constants Obtained in Shear Tests

sample with n = 647 lb. per sq. ft., 67 percent. This makes the average for the three at their maximum shear resistances, 65 percent, which is not unusual for very plastic materials.

The relation of maximum s to n can then be shown as by the top line of Figure

Failure of a vertically loaded cylinder is illustrated in Figure 20A. In this case it may be considered that cones formed, as shown at each end, move closer as vertical deformation proceeds and cause the soil material to first bulge and ultimately fail by splitting off the sides of the

cylinder. The cylinder might fail along any planes which are between the cones and parallel to the cone surfaces.

The angle, a, made by surfaces of the cone with the horizontal equals, according to theory, 45 degrees + $\frac{\phi}{2}$; (16a, 17) ϕ being the angle of internal friction along the surfaces of the cones at the instant the cylinder fails.

If ϕ equaled 21.3 deg. as in Figure 18, a should be equal to 55.65 deg. and according to the theoretical relation origin O at the angle a = 55.65 deg. with the horizontal, until it intersects the arc at some point M. The vertical projection of OM then gives graphically, according to Mohr, (19) the shear stress (s = 387 lb. per sq. ft.); and the horizontal projection of OM gives the normal stress (n = 265 lb. per sq. ft.) along the surfaces of the cones at the instant the cylinder fails.

A straight line drawn tangent to the arc at the intersection point M shows the relation of s to n, and, as shown, gives the

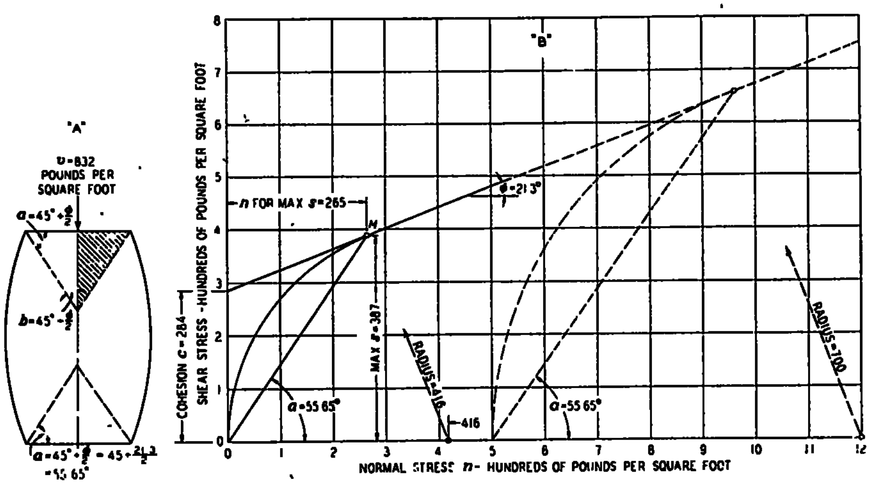

Figure 20. Graphical Representation of Stresses in Cylinder at Failure

v = 2 c tan a, (18) the ultimate vertical pressure, v, should equal 832 lb. per sq. ft.

If such were the case, the data could be analyzed by means of Mohr's circle as follows: With a center at a distance[7] $\frac{v}{2}$ = 416 lb. per sq. ft., from the origin, an arc of a circle with radius $\frac{v}{2}$, is constructed as shown in full lines in Figure 20B; and a line is drawn through the

value of ϕ = 21.3 deg. and c = 284 lb. per sq. ft., which were assumed.

Unfortunately, however, the angle of fracture is not always defined clearly by the tested sample. Therefore it is necessary to have at least another set of Mohr's circles and draw tangents to corresponding circles in each set in order to determine the s-n relations.

The desired data are obtained by testing cylinders of soil at predetermined amounts of lateral restraint. The many types of apparatus used for this purpose are described in the compendium on apparatus included in these proceedings. One type has the essential features shown in Figure 21. The sample may be tested

[7] The distance from the origin to the center of the circle is $\frac{v + l}{2}$, in which l is the lateral pressure. The radius of the circle is $\frac{v - l}{2}$. In unconfined cylinder test l is zero.

as an open system with porous stones as shown, or a closed system with metal disks instead, depending on the purpose of the test.

Data obtained from tests at several lateral pressures are used. For demonstration only one test supplementary to that of the unconfined cylinder is used.

If from such test, with the lateral pressure, l, kept constant at 500 lb. per sq. ft., it was found that v at failure of the cylinder was 1900 lb. per sq. ft., the cor-

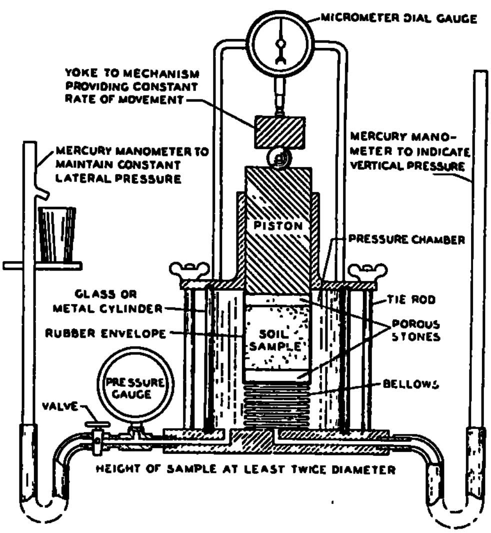

Figure 21. Schematic Diagram of Confined and Constrained Compression Test

responding stress circle could be constructed with a radius of $\frac{1900 - 500}{2}$ and a center at $\frac{1900 + 500}{2}$ distance from the origin (20). This is shown by the broken lines in Figure 20.

Let it be assumed that the stress-strain relations of the cylinders were as shown in Figure 22. With the v's at percentages of the maximums from Figure 22, or Table 3, a series of diagrams could be constructed as in Figure 23. These would disclose c′ and ϕ' values as shown in Figure 18 and the values of *a* in Table 3. From such, the shear constant-deformation diagram could be constructed

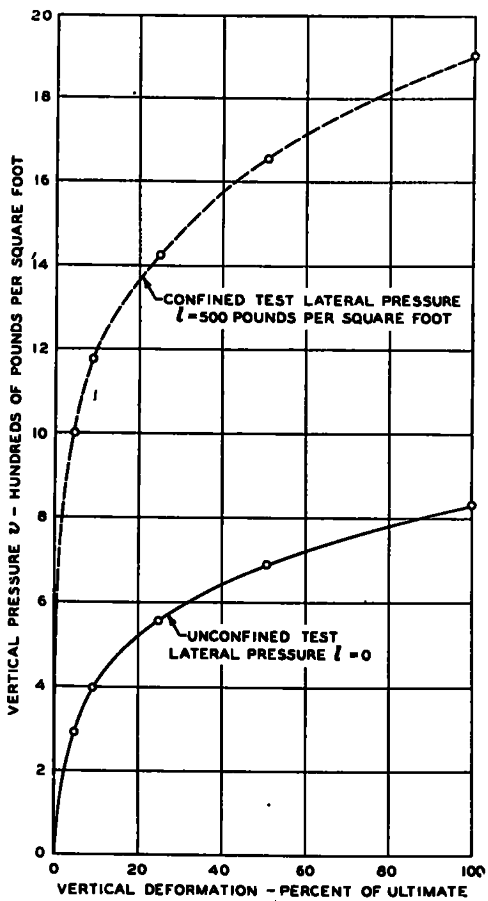

Figure 22. Stress-Strain Relation for Cylinder Test

TABLE 3

ASSUMED VALUES OF a FOR DIFFERENT DEFORMATIONS

Vertical deformation	Vertical pressure, v		Angle, a
percent of ultimate	*lb. per sq.ft.* l = 0	l = 500	*deg.*
4.6	290	982	49.65
9.2	396	1160	51.0
24.6	558	1430	52.9
50.7	690	1660	54.3

similar to Figure 19, but with different deformation scale.

Only Part of Soil Strength is Usable

The linear s–n relations shown in the foregoing are valid only up to the particular normal pressure which causes the moisture content at which a given soil is tested to become the "critical." For normal pressures greater than this, the relation of s to n may approach or even coincide with the horizontal.

Performing confined compression tests on samples at different moisture contents and in a closed system furnishes desired relations of moisture content of soil to c and ϕ through any range of normal pressures. From such data, usable values of c and ϕ can be selected with respect to pore pressures in the undersoil. This information permits the speed of em-embankment construction to be so controlled that pore pressure, disclosed by standpipes inserted in the undersoil, will not be developed to a magnitude which makes failure imminent.

Testing samples at constant rate of strain instead of constant rate of load application, facilitates determination of the maximum shear strength of cohesive materials.

Usable strengths of materials which comprise a particular structure depend also upon the deformations permitted in the structure as a whole.

If the sole purpose of steel reinforcement in concrete pavements were to prevent cracking due to tensile stresses of the concrete, only a part of the great strength of the steel could be utlized because the concrete would crack at smaller deformations that those required to develop the total strength of the steel.

So in the design of retaining walls, bridge piers, abutments and the like, where soil is an influencing factor, no more of the ultimate soil strength can be utilized than that which is developed at maximum safe deformations permissible in the structure as a whole.

For this purpose a shear stress-deformation relation, such as shown in Figure 19, may prove helpful. If, for instance, the bank of earth, Figure 24A, were allowed to deform gradually, its mobilized cohesion and internal friction would be increased, thus diminishing the pressure against the wall, until when the maximum values, c and ϕ, had been developed the wall would be required to resist only what has been termed the "active"

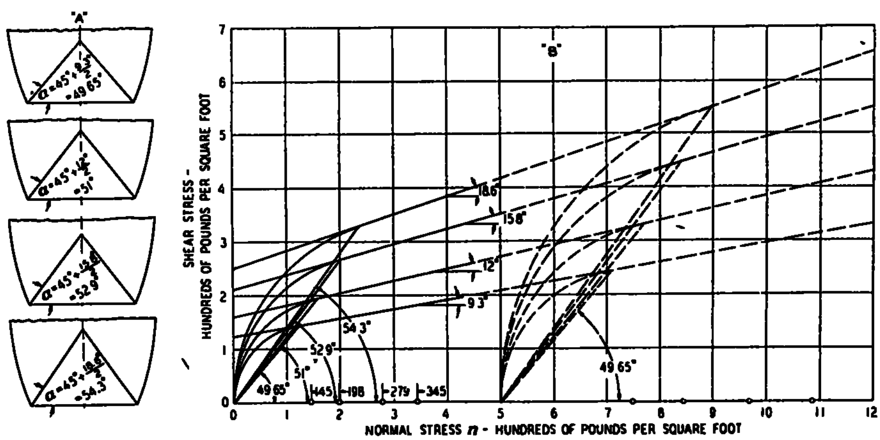

Figure 23. Graphical Representation of Stresses in Cylinder Before Failure

earth pressure. This is expressed by the formula (19):

$$L = h\left(\frac{wh}{2}\tan^2 b - 2c\tan b\right)$$

in which

L = total lateral pressure per foot width of wall

h = height of wall, and

w = weight of earth per cubic foot.

If comprised of soil which has cohesion the embankment should according to theory require no wall for heights up to

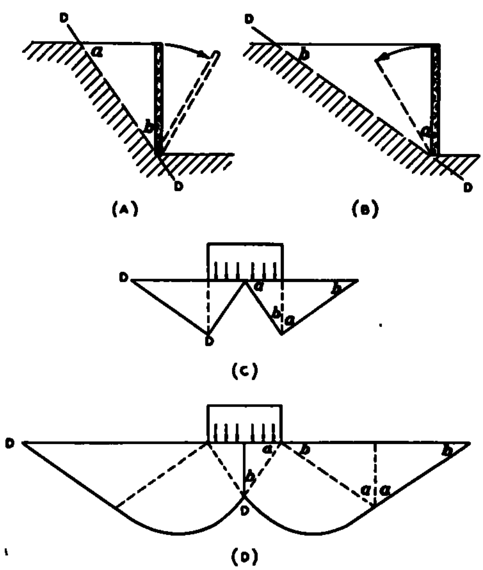

Figure 24. Illustrating Planes of Slip—(ϕ = 20°)

H which is determined by the formula (15) $H = \frac{2c}{w} \times \tan a$, and this it will be noted equals the compressive strength v of the cylinder divided by the weight per cubic foot of the earth. For the soil, Figure 20, v = 832 lb. per sq. ft. and

$$w = 100 \text{ lb. per cu. ft.}$$

$$H = \frac{832}{100} = 8.32 \text{ ft.}$$

But in order that full shear strength be developed, the embankment, if of the soil of Figure 19, would have to incline outward as shown in Figure 24.[8]

For heights greater than 8.32 ft., the required retaining wall would have to be inclined similarly in order that full "active" earth pressure be developed.

If, prior to ultimate failure, the soil in Figure 24A and B, performed as indicated in Figure 14 for molecular materials and in Figure 16A for soil in a laboratory shear test, and, if the amount of distortion were relatively the same as for the soil sample in Figure 19, then the movement of the wall, if it rotates as shown, can be computed from the expression

$$\tan b' = \frac{1 - \sin\phi}{\frac{200}{m} - \cos\phi}$$

in which

b' = angle the wall rotates

m = percentage of sample deformation (Fig. 19)

ϕ = angle of internal friction

At the ultimate, with m = 65 percent and ϕ = 21.3 deg.,

$$\tan b' = \frac{1 - 0.363}{\frac{200}{65} - 0.932} = 0.296$$

$$b' = 16.5 \text{ deg.}$$

This would cause the top of a 20-ft. wall to move outward about 5.9 ft. (20 × 0.296).

If the wall were forced backward as in Figure 24B until the full shear strength of the soil were developed, the wall would be inclined backwards similarly and the pressure against it would be that which

[8] Wall movement as affecting earth pressures was discussed by Darwin in 1882, see reference (21).

has been termed "passive" earth pressure, P This is expressed by the formula (19)

$$P = h\left(\frac{wh}{2}\tan^2 a + 2c \tan a\right)$$

With the same assumptions used in estimating the movement due to active earth pressure, wall movement required to develop passive earth pressure, is indicated by the expression

$$\tan a' = \frac{1 + \sin \phi}{\frac{200}{m} + \cos \phi}$$

By substitution of the ultimate values as above, the movement of the top of the wall becomes 6 8 ft

For instance, for a deformation of 5 percent, which in turn would be expected to cause the top of the 20-ft wall to incline outward 0 41 ft from the vertical, c = 150 lb per sq ft and ϕ = 11 3 deg For a deformation of 2 percent, which in turn would cause the top of the 20-ft wall to incline but 0 18 ft from the vertical, c = 100 lb per sq ft and ϕ = 7 6 deg

Relatively high percentages of the ultimate strengths of sandy, silty and many undisturbed soils, which fail at small deformations, are usable

It will be noted from Figure 24, that the angle *a* is significant in theories of design The earth mass considered in the design may be conical, as under circular footings, pyramidal, under square bases, or wedge

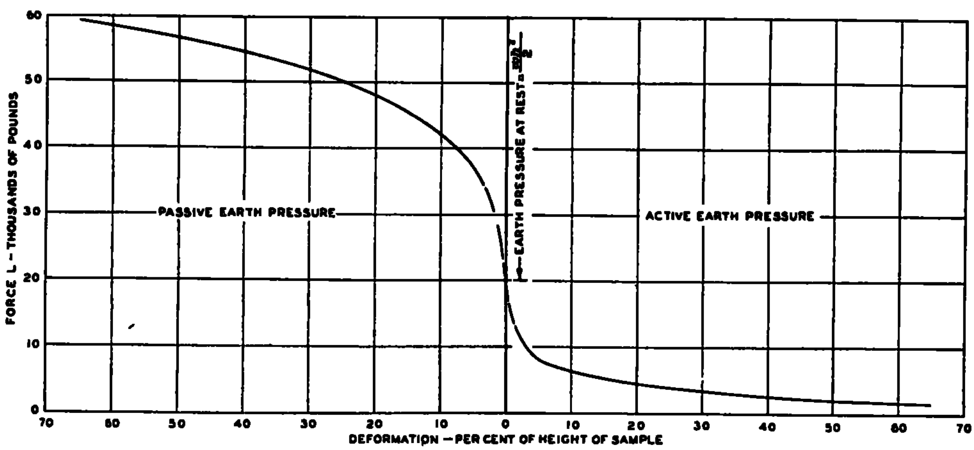

Figure 25 Pressure Diagram for Wall 20 Feet High

It is evident that no such inclinations of the wall could be allowed So at the outset it becomes necessary to decide how much the wall can be allowed to incline and still remain satisfactory for the purpose intended by its construction Then usable values of c′ and ϕ' can be selected As a further guide a graph which shows the complete range of passive and active pressures with respect to soil sample deformations as in Figure 25 might be helpful In the construction of this graph, the values of c and ϕ shown in Figure 19 were used

shaped under strip loads and behind retaining walls, but its vertical projection is persistently a right triangle which has the acute angle *a* and its complement *b*, obtained from the cylinder tests

The wedge (17–19) assumed in the design of retaining walls (Fig 24) has a lower boundary D–D which is supposed to become the plane of slip when shear strength of the soil has been exceeded If the failure is produced by the weight of earth which tends to force the wall outward, D–D will make the angle *a* with the horizontal and *b* with the vertical If,

however, the wall were forced backward, as indicated in Figure 24B, D–D would make the angle *b* with the horizontal and *a* with the vertical.

In the determination of supporting value of soil under a strip load, the triangles considered in a formula published previously in *Public Roads* (22), are those shown in Figure 24C, and if by the method suggested by Prandtl (20), those in Figure 24D.

That indicated in 24C is to provide against failure of the under soil when there is a possibility of the embankment splitting in the middle. That shown in 24D is to provide against failure when the embankment is stable enough in itself to settle as a whole.

In each case the soil under the loaded area is subjected to "active" earth pressure and the resisting soil, outside the loaded area, to "passive" earth pressure.

If no deformation of the earth (Fig. 24) were permitted, the pressure exerted against the wall, would become what has been termed "earth pressure at rest."

This can be determined from the expression

$$L = K \times \frac{wh^2}{2}$$

in which

h = height of wall in feet.
w = weight per cubic foot of earth, and
K = ratio of the lateral pressure l, to the vertical pressure v of the soil.

The expression for earth pressure "at rest" is similar to the expression for what has long been called "equivalent fluid pressure" (19). It may be defined as the pressure produced by a hypothetical fluid, equal in effect to the pressure exerted laterally, by vertically loaded soil. The coefficient K has been found to vary from 0.8 for clays to 0.4 for sands (23). Goodrich found in 1904 (24) that the resultant may act as high as 0.4 h above the bottom of the wall. See also report by Jacob Feld (25). Quite recently it was found by investigations carried on in Berlin that the resultant may act from 0.4 h to 0.46 h above the bottom of the wall.

By means of the apparatus shown in Figure 21, a completely constrained compression test may be performed also in stabilometers. In this case the pressure gage, instead of the adjustable manometer is used to determine the pressures in the chamber. If it were possible to prevent horizontal deformation of the sample, and if the voids were completely filled with water, there could be, excluding the deformation of the water itself, no vertical deformation of the sample. In such case the value of K (12, 14, 15) depends on the elasticity of the soil and is not related to shear strength of the sample. Therefore materials differing widely in strength may have K's of the same value. Professor D. P. Krynine (26) and F. L. Plummer (27) and others (28) have called attention to this pertinent fact. Professor Krynine states: "The value of K in monotonously isotropic elastic bodies (confined water, some metals, etc.) is equal to 1." Mr. Plummer states that "soil pressure at rest depends not on cohesion and internal friction but solely on the elastic properties of the backfill."

While Mohr's circles could be constructed from the data which furnish the K values, it can be seen they would have no significance with respect to c and ϕ values.

COMPACTION TESTS

Essentially the compaction tests consist of densifying the same soil, at different moisture contents, with a given compactive effort and from the resulting data determining the maximum density which can be obtained and the corresponding optimum moisture content.

The tests have wide use and procedures for performing them by means of both

tamping pressures (the Proctor method) and static pressures (the California method) are described in reports included in these proceedings.[9]

The purpose in referring to them here is to stress their significance with respect to the consolidation and shear test data.

Consider first Figure 26. This shows the weight per cubic foot (solids + moisture) of a particularly troublesome soil, at different moisture contents when the pores are completely filled with moisture (wet weight-zero air voids curve),

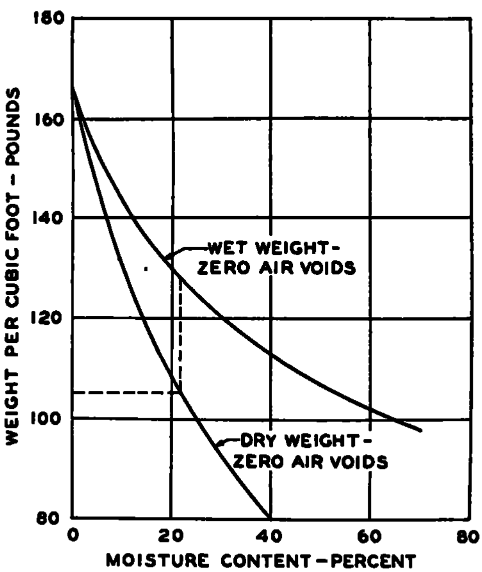

Figure 26. Relation of Density to Moisture Content

and the weight of the solids alone (dry weight-zero air voids curve).

The soil contains only 18 percent of sand and its specific gravity (SG), is 2.65.

The wet weight per cubic foot (W) is determined from the expression

$$W = \frac{1 + \frac{w}{100}}{\frac{1}{SG} + \frac{w}{100}} 62.4 .$$

[9] See pages 151, 154, 315.

The dry weight per cubic foot (W_o) is determined from the expression

$$W_0 = \frac{62.4}{\frac{1}{SG} + \frac{w}{100}}$$

Curve 1, Figure 27B shows the relation of weight of solids per cubic foot of wet soil to moisture content as determined by the Proctor method of test. At this compaction a density of 105 lb. per cu. ft. is attained when the moisture content of the soil equals 19.6 percent. If a considerably greater amount of compaction, either tamping or static (California method of static pressure equals 2000 lb. per sq. in.) were used, a similar relation such as shown in curve 2, Figure 27B, would be expected. In such case the density becomes 125 lb. per cu. ft., and the optimum moisture content 10 percent. The broken line drawn through the peaks of curves 1 and 2 then represents the relation of any desired density to the corresponding optimum moisture contents (3).

Since the maximum density curve is for all practical purposes parallel to the zero air voids curve (3), tests which furnish the relation shown in curve 2, Figure 27B should be just as satisfactory a basis of estimate as the Proctor tests.

The equivalent static pressures required to produce the given densities, if determined by test can then be shown as in Figure 27C.

The next step is to show a relation between height of embankment and consolidating pressures as in Figure 27A. Here it is assumed that the weight per cubic foot of embankment is equal to that of saturated soil at the dry weight density of 105 lb. per cu. ft. This wet weight from Figure 26 equals 128 lb. per cu. ft.

The pressures exerted at the bottoms of fills of different heights would under these conditions be as follows: For 20 ft., 2,560 lb. per sq. ft.; for 50 ft., 6,400 lb. per sq. ft.; and so on. The moisture contents

to which these will consolidate the soil are obtained from moisture content-pressure curves for compacted samples. The dry weight densities corresponding to these moisture contents (from Fig. 26) are plotted against the heights of embankment which produce the given pressures, in Figure 27A.

These curves indicate that from the standpoint of consolidation pressure this particular material as placed at the Proctor density should be safe for embankments up to 50 ft. high. But for a greater height, say 100 ft., the density to be safe should be at least 109 lb. per cu. ft., with a corresponding optimum moisture content of 17 percent. The density of 109 lb. per cu. ft. is expressed equally well by either 87 percent of the California maximum, curve 2, or 104 percent of the Proctor density, curve 1. From the relation in Figure 27C the compactive effort would have to be increased from an equivalent of 200 lb. per sq. in. to 350 lb. per sq. in.

In connection with the provision of adequate shear resistance, it has been found by investigation that:

(a) 98 to 100 percent maximum density as indicated by the Proctor method of compaction is attainable in the construction of embankments without prohibitive effort on the part of the builder.

(b) Troublesome fills, which had been compacted by methods not utilizing designed moisture control, contained at times soil which had densities of but 65 to 85 percent of the Proctor maximum.

Computations of critical height of embankment with 1½ to 1 slopes by Resal's (15) or Taylor's method (28a) indicate the relative high stability of fills at 100 percent Proctor density as compared to that of fills which have considerably less density if they become saturated.

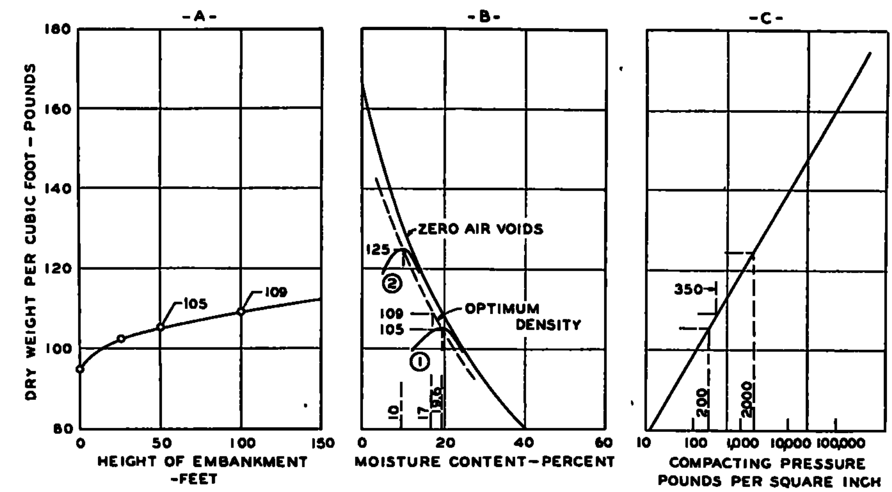

Figure 27. Compaction Data

By use of the ultimate values of c and ϕ as determined by direct shear tests, it was indicated in the case of the soil, Figure 27, that the critical heights were as follows (by Resal's method): At a density of 79 percent Proctor, 34 ft.; at 100 percent Proctor, approximately 2,000 ft. In the case of another extremely troublesome soil, they were: At 66 percent Proctor, about 10 ft.; and at 100 percent Proctor density, slightly less than 400 ft.

In the preceding discussion of usable

shear values, data on a highly plastic soil indicated that at values of c' and ϕ' equal to approximately 1/3 of the ultimates, the sample had deformed 2/65 or but 3 percent of the amount required to develop full shear strength. Making use of 1/3 of the ultimates as usable safe values, the critical height for the soil, Figure 27, becomes at 79 percent Proctor, 6 ft.; at 100 percent Proctor, 41 ft.; and for the second soil at 66 percent Proctor, 2 ft.; and at 100 percent Proctor density also 41 ft.

A factor of safety of 1.5 instead of 3, indicates safe heights well above 100 ft. for 100 percent compaction. Therefore, it would seem that for soils generally and for embankments of moderate height, compaction to densities close to the Proctor should have adequate shear resistance and not exceed the critical consolidation pressure.

For high fills greater compaction seems desirable. A particularly troublesome problem may develop if the lower part of the fill becomes saturated before the upper part is placed. In such case it may be necessary to install standpipes and regulate subsequent speed of construction with respect to the observed pore pressure in the part already constructed.

Finally, adjustment of moisture content on the job may effect a saving of rolling effort or attain greater compaction with the same effort. If as has been found in some cases, the compacting equipment produces more than 100 percent Proctor density at the Proctor optimum moisture content, there is the possibility that with the same effort, still greater density may be obtained by reducing the moisture content.

To determine this, the density attained at a lower moisture content should be plotted on a graph such as shown in 27B. Through the point thus shown draw a curve similar to those shown as 1 or 2. The intersection of such curve with the broken-line maximum density curve discloses the new optimum moisture content to be used and the density which may be attained.

INDICATOR TESTS

The most commonly used of those tests which indicate rather than disclose quantitatively the engineering properties of soil are the plasticity tests and the mechanical analysis.

The significance of these determinations and standard procedures for making them have been published elsewhere (29, 30).

The purpose of this discussion is to demonstrate their use in the combining of soils in order that mixtures which have desired properties may be obtained.

In this connection reference is made to the Virginia clay, the Missouri clay and the Pennsylvania soil; Table 4.

The results as usually reported include the liquid limit, the plasticity index and the percentages of sand, silt, and clay in the mixture. But it has been suggested previously in *Public Roads* (31) that the properties contributed to the mixtures by the sand, silt and clay differ widely. The sand fraction, which contributes the internal friction serves at the same time to reduce the capillarity and the cohesion. The silt and clay combined serve to provide the capillarity and the clay fraction to provide the cohesion in the mixture. It has also been found as a generalization that the liquid limit indicates the capillarity of the mixtures. Consequently as the combined silt and clay fractions are increased, the liquid limits are increased proportionately. In a similar manner, the plasticity index indicates the cohesion of the mixture, and by the same token is increased proportionately as the clay content is increased.

If the liquid limit is divided by the combined percentage of silt plus clay, the result will indicate the effect of one percent of the combined silt plus clay content on providing capillarity in the mixture.

If as an average value, a liquid limit of 20 and a plasticity index of 0, can be taken for sands containing no silt or clay, then the effect of adding any percentage of combined silt plus clay of that particular soil to such sand can be determined by means of the expression

$$LL = 20 + SI \times (\text{clay} + \text{silt contents}) \quad (5)$$

in which SI may be arbitrarily called the capillarity index.

The effect of adding clay alone to the mixture of sand and silt can be estimated in a similar manner by means of the formula

$$PI = CI \times (\text{clay content}) \quad (6)$$

in which CI may be termed the cohesion index (32).

Percentages of clay needed in a mixture to provide a given plasticity index may be determined in a similar manner by use of formula 6.

The combining of two soils in such a manner as to provide a given liquid limit or plasticity index in the resulting mixture is facilitated by means of a chart as shown in Figure 28 (33).

Here the Missouri clay is considered as an admixture to the Pennsylvania soil. The plasticity index of the Pennsylvania soil is plotted as a moisture content on the line for 0 percentage of admixture and the plasticity index of the Missouri clay is plotted as a moisture content on the line for 100 percent admixture. A line connecting these two points represents the plasticity index for any range of admixture from 0 to 100 percent.

TABLE 4

ILLUSTRATIVE INDICATOR TEST DATA ON SOILS AND SOIL MIXTURES

Soil type	Liquid limit	Plasticity index	Sand	Silt	Clay	Capillarity index	Cohesion index
			percent	*percent*	*percent*		
Virginia clay	132	101	6	6	88	1.19	1.15
Missouri clay	68	42	11	35	54	0.54	0.78
Pennsylvania soil	26	6	27	51	22	0.08	0.27
Missouri clay, 33 percent and Pennsylvania soil, 67 percent	40	18	22	45	33		
Same mixture 55 percent with sand 45 percent	31	10	57	25	18		

The values of SI and CI for the three soils referred to above are shown in the last two columns, Table 4.

To illustrate the use of these relations let it be assumed that the Virginia soil is to be changed by an admixture of sand so that the resulting mixture shall have a liquid limit of 50.

By substitution of 50 for LL and 1.19 for SI in formula 5, we have

$$50 = 20 + 1.19 \times (\text{silt} + \text{clay content})$$

and this gives a required silt plus clay content of $\frac{50 - 20}{1.19} = 25$ percent.

Therefore the sand content of the mixture must be increased from 6 to 75 percent.

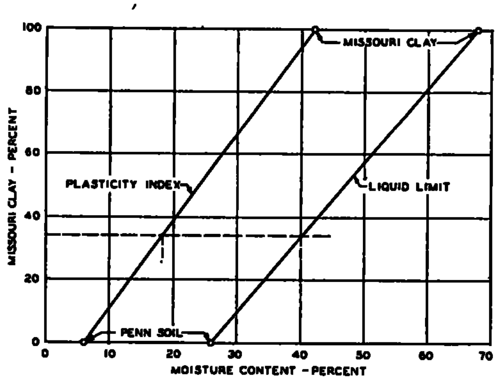

Figure 28

In a similar manner the line represent-

ing the liquid limits for any range of admixtures from 0 to 100 percent is also constructed.

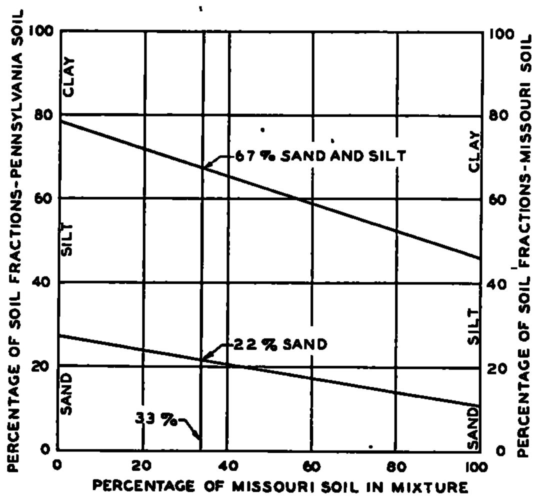

Figure 28A

For any desired liquid limit then, say 40, it is only necessary to find the ordinate at the intersection of the 40 percent moisture content line with the liquid limit line. In this case such ordinate indicates an admixture of 33 percent and a corresponding plasticity index of 18.

The grading of the mixture is then disclosed graphically by Figure 28A, which has been used by E. F. Preece, National Park Service.

Such a mixture, shown also in Table 4, has the characteristics of an A-7 subgrade.

If it were further desired to reduce the plasticity index to 10 by an admixture of sand having a liquid limit of 20 and a plasticity index of 0, a similar procedure with appropriate graphs will disclose a required admixture of the sand of 45 percent and the resulting mixture will have the characteristics of an A-2 subgrade soil. (See Table 4.)

To illustrate a similar type of problem, let it be considered that in the reconditioning of an old road surface 800 cu. yd. of material having a plasticity index of 4 is required, and furthermore that in the old road surface there are 350 cu. yd. of material with a plasticity index of 2.

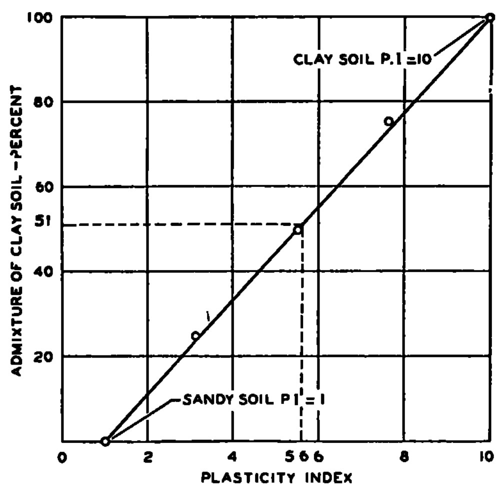

Figure 29

The plasticity index of the 450 cu. yd. of borrow material may then de determined by means of the formula

$$PI_M = PI_A + P_B(PI_B - PI_A)$$

in which

PI_M = plasticity index of the mixture

PI_A = plasticity index of the old road material

P_B = ratio of the borrow material to the total yardage, and

PI_B = plasticity index of the borrow material.

By substitution we have

$$4 = 2 + \frac{450}{800} (PI_B - 2)$$

and

$$PI_B = \frac{2500}{450} = 5.6$$

If two borrow pits are available, one of a soil with a plasticity index of 1 and another with soil having a plasticity index of 10, a graph similar to that referred to above is constructed as shown in Figure 29. From this chart it is found that for the resulting mixture to have a plasticity index of 5.6, it must have 49 percent of sandy soil and 51 percent of clay soil.

Therefore the 800 cu. yd. of required material must be comprised of

Old road surface	350 cu. yd.
Clay pit soil.	229 cu. yd.
Sandy pit soil . . .	221 cu. yd.

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