DEPARTMENT OF SOILS INVESTIGATIONS

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PRESENT STATUS OF SOILS INVESTIGATIONS

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SYNOPSIS

A review of soil investigations during the past decade discloses that notable progress has been made in the use of indicator and compaction tests by the various State highway laboratories and the utilization of test data in subgrade treatment and in the design and construction of road surfaces, base courses and embankments. Use of consolidation and shear tests has lagged due to the fact that the correlation of test results with performance of soil structures and foundations has not been forthcoming. Needed especially in this respect are observations of settlements of bridge piers, sway or creep of retaining walls and abutments and of the stability of embankments. Yet, procedures of test and applications of test data for determining settlements have attained a more or less standard status and are particularly applicable for estimating settlements of embankments due to consolidation of the foundation soil because the relatively large size of the loaded area under the embankment approaches more nearly the conditions of test than is met in the smaller loaded areas under footings and bridge piers. By use of such estimates and also those furnished by the application of shear test data in theories now available as yard sticks, every highway embankment becomes a potential source of the information needed in the correlation necessary to bring the consolidation and shear tests to a practical status.

The past decade saw considerable effort devoted to investigations of the engineering properties of soil, and a commensurate amount of material published on the results attained. The findings applicable to highway construction were compiled under the title "Soil Stabilization and Soil Mechanics" and published as Part II, Proceedings, Highway Research Board, 1938 (1).[1] Included was a discussion of four general groups of tests as follows:

1. Indicator tests.
2. Compaction tests.
3. Consolidation tests.
4. Shear tests.

It seems appropriate at this time to review the status of these groups with respect to (a) apparatus and testing procedures; and (b) the utilization of the test data in practice.

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

Indicator Tests.

The determinations which indicate rather than disclose quantitatively the engineering properties of soil are:

- Mechanical analysis.
- Liquid limit.
- Plastic limit.
- Plasticity index.
- Shrinkage limit.
- Centrifuge moisture equivalent.
- Field moisture equivalent.
- Shrinkage factors.

The significance of these determinations for disclosing the properties of subgrade soils, and their use in a classification of subgrades based upon the performance of the soil was first discussed in *Public Roads*, May 1929 (2). Procedures for making the determinations were published in *Public Roads*, October 1931 (3).

By 1933, more than 30 State highway departments had included at least several of these determinations in their routine laboratory examinations of road materials. Procedures for making all the indicator determinations were adopted as standard methods by the American Association of State Highway Officials in 1934 (4); as tentative standard procedures by the American Society for Testing Materials in 1935 (5); and were advanced to standard status by the latter Society in 1939.

Specifications for stabilized road surfaces, and for stabilized base courses based on properties of the materials disclosed by the mechanical analysis, the liquid limit and the plasticity index were adopted as standards by the American Association of State Highway Officials in 1938 (4); and are now being considered as tentative standards by the American Society for Testing Materials.

Compaction Tests.

Tests which disclose the moisture content required by soil to attain maximum density at a given compactive effort produced by tamping, were described by R. R. Proctor in the Engineering News-Record in 1933 (6) for use in the construction of earth dams. Compaction tests employed by the California Highway Department and which utilize static pressures in the determination of moisture content-density relations of compacted soil for use in highway embankments were described by T. E. Stanton in 1938 (7).

Use of the Proctor tests generally in highway construction, and especially to determine the effect of chemical, portland cement and bituminous admixtures for changing the density and stability of soil was discussed in *Public Roads* in February 1935 (8), and May 1936 (9).

Procedures for performing the compaction tests on soils were adopted as standard by the American Association of State Highway Officials in 1938 (4), and are now being considered as tentative standard by the American Society for Testing Materials.

Specifications for the selection of embankment materials, based upon compaction test data were adopted as standard by the American Association of State Highway Officials in 1938.

Use of the compaction test data in the design and construction of soil-cement road bases was described in the Proceedings, Highway Research Board, 1937 (10). Procedures for performing the compaction tests on soil-cement road mixtures have received consideration as tentative standard by the American Society for Testing Materials and have been published for information by that Society in the proceedings of the 1939 annual meeting.

In the Proceedings of the Highway Research Board, 1938, Vol. 18, Part II, 24 State highway departments reported use of the compaction tests.

Consolidation and Shear Tests.

The consolidation tests were described by Charles Terzaghi in the Engineering News-Record in 1925 (11), and in *Public Roads* in 1927 (12). A complete exposition of the theory of consolidation, the consolidation tests and their practical application was published in *Public Roads* in 1937 (13).

As yet, no effort has been made to standardize the consolidation tests, and according to the Proceedings of the Highway Research Board referred to above, only nine State highway departments have obtained apparatus for making the tests.

Shear tests for soil have been described repeatedly since the time of Coulomb. The various types are described in the Compendium on Soil Testing Apparatus, Proceedings of the Eighteenth Annual Meeting, Highway Research Board, Part II (1).

As in the case of the consolidation tests, no effort to standardize shear tests has yet been made; and, according to the Research Board Proceedings (1) only nine State highway departments are equipped to make them.

Progress in Soil Mechanics has Lagged.

The rapid adoption of the indicator and the compaction tests suggest that the profession is only too eager to include as working tools those soil examination procedures which have practical value in the fields of design and construction. Its failure to include the consolidation and shear tests is the best proof that the subject of soil mechanics in which data furnished by the consolidation and shear tests are utilized, has not, with few exceptions, as yet emerged far enough from the realm of research to have general use in practice.

This is indicated by opinions included in a "Symposium on the Practical Application of Soil Mechanics," *Transactions* A. S. C. E. (14), with special reference to building foundations. Charles Terzaghi (page 1433) states:

> "Considering the present state of knowledge in the field of foundations, the settlement observations represent by far the most important type of research. The results of theoretical and of laboratory investigations in this field cannot claim more than an academic value until the importance of errors has been investigated thoroughly and repeatedly."

Reference is made also to discussions by A. Strieff, page 1496 and Jacob Feld, page 1466 (14).

Apparently then, the problem of changing the results furnished by research into workable tools of design now confronts the profession as a whole and furthermore, as an essential step in its solution, observations of the performance of structures in service must be made. By the same token it is equally important that yardsticks or gauges be set up as a basis of comparison, if the full value of settlement observations is to be obtained.

To illustrate, reference is made to the development of the subject of hydraulics. As a gauge or yardstick, it was accepted that the theoretical flow of water was in accordance with the relation

$$v=\sqrt{2gh}$$

in which

v = velocity of flow
g = acceleration due to gravity
h = pressure head

The actual flow under the many desired conditions met in service was then determined by observation and experiment, after which the basic relation was modified to read

$$v=C\sqrt{2gh}$$

in which the coefficient C expressed the ratio of the actual velocity to that indicated by the theory, and was found to have values of 0.5, 0.7 and numerous others less than 1.

Possibly progress in soil mechanics especially as applied to embankment construction can be expedited by following much the same course, utilizing as yardsticks estimates of performance computed by means of usable test data substituted in expressions of current theories of stress distribution in loaded soil. Such procedure utilized by the various State highway departments, supplemented by observations of the performance of structures after construction would make every highway bridge, retaining wall and embankment a potential source of the desired information. This combined with corollary large scale investigations of earth masses under controlled conditions of accelerated test should quickly furnish the needed values of the correction coefficient C.

Principally to assist in such possible approach to the design and construction of embankments, a review of suggested theories is presented. In its perusal, cognizance should be taken of the fact that soil mechanics is not an exact science. The theories of stress distribution have for their basis the assumptions that soil en masse is homogeneous, isotropic and either entirely elastic or entirely plastic. As a matter of fact, it is neither homogeneous nor isotropic, and its deformations depend upon some complex combination of the properties of elasticity and plasticity, and not on either of them independently. Therefore, the extensive use of approximations and short cuts to reduce the effort involved in the applications of the theories seems justified. The review considers two types of soil deformation: those due to consolidation of the soil, and those which depend upon its lateral displacement.

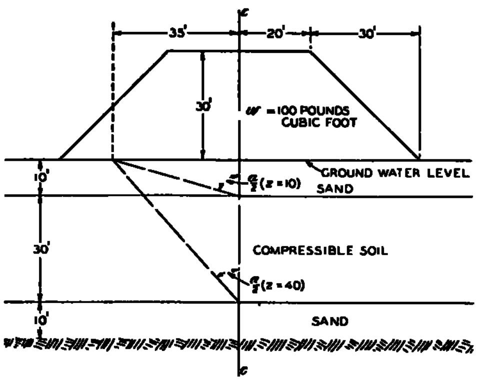


Figure 1. Soil Profile and Embankment Cross Section

TABLE 1

CONSOLIDATION DATA

Vertical pressures, lb. per sq. ft.		Voids ratios e	Average voids ratios
Inactive at top boundary	600	1.47	$1.33 = e_i$
Inactive at bottom boundary	2400	1.19	
Total at top boundary	3573	1.09	$1.05 = e_f$
Total at bottom boundary	4716	1.02	

SETTLEMENT OF EMBANKMENT DUE TO CONSOLIDATION OF THE UNDERSOIL

This material supplements the report "The Trend of Soil Testing" presented at the Eighteenth Annual Meeting of the Highway Research Board with special reference to the discussion, "The Consolidation Test."

The Problem.

Let it be assumed that an embankment of the cross section shown is to be constructed on the soil profile, Figure 1. Let it be assumed further that the natural soil is sufficiently old to have attained complete consolidation due to weight of the soil; that the weight of the immersed undersoil equals 60 lb. per cu. ft.; and that the fill material will weigh 100 lb. per cu. ft. It is considered that the sand is incompressible and therefore that only the compressible layer of soil will undergo consolidation when the fill is placed.

Computation of the Total Settlement.

The first step is to compute the inactive pressures or those under which the soil has been compressed and which consequently are no longer effective for producing further consolidation. In the case of the profile, Figure 1, the inactive pressure at the top of the compressible layer becomes

$$10 \times 60 = 600 \text{ lb. per sq. ft.}$$

and at the bottom of the compressible layer

$$40 \times 60 = 2400 \text{ lb. per sq. ft.}$$

These are recorded in column 1 of Table 1 as shown.

As a second step, the consolidating pressure or that which will be produced by the embankment when constructed, is determined by means of the Boussinesq formula (13) for pressures, p_z, along the center axis beneath a uniform strip load. It is as follows:

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

in which

p = pressure in excess of the inactive pressure exerted by the embankment at its base.

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

By substitution

$$p_{(z=10)} = \frac{3000}{3.142}(2.584+0.528)$$
$$=2973 \text{ lb. per sq. ft.}$$

In the same manner, the pressure at the intersection of c-c and the bottom of the compressible layer is found to be 2316 lb. per sq. ft.

The consolidating pressures added to the inactive pressures give total pressures

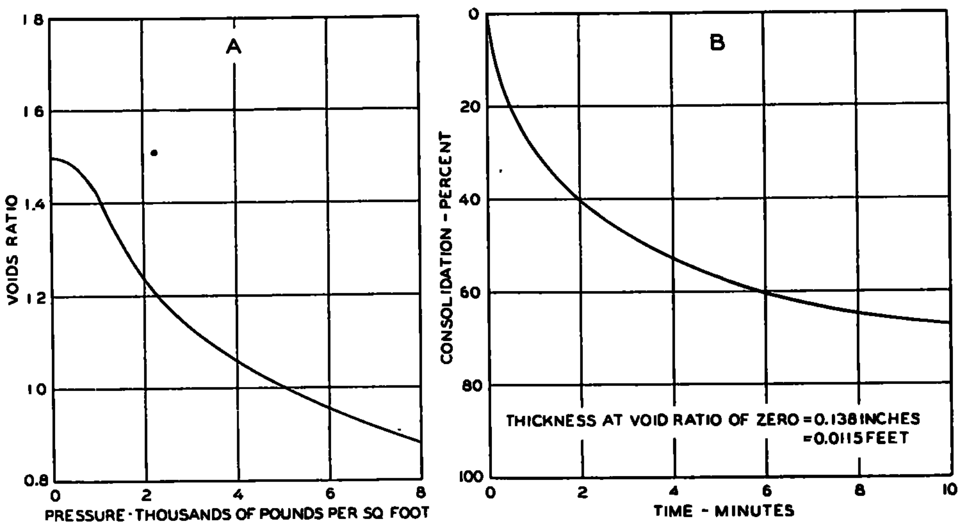


Figure 2. Consolidation Characteristics of Undisturbed Sample

In the case of the embankment, Figure 1, the side edge of the strip load is taken as the edge of a rectangle equivalent in height and area to the embankment cross section (15).

With p equal to 3000 lb. per sq. ft., the consolidating pressure exerted along the center axis (c-c) at the top of the compressible layer is computed as follows:

$$\text{Tangent}\frac{a}{2} = \frac{35}{10} = 3.5$$

$$\text{Arc tangent } 3.5 = 74.06°$$

then

$$a = 2 \times 74.06° = 148.1°$$
$$= \frac{148.1 \times \pi \text{ radians}}{180} = 2.584 \text{ radians}$$
$$\sin a = \sin 148.1° = \sin 31.9° = 0.528.$$

of 3573 lb. and 4716 lb. per sq. ft. acting at the top and the bottom boundaries respectively of the compressible layer, and these also are recorded in Table 1.

Reference is now made to the data, Figure 2, furnished by consolidation tests performed on an undisturbed sample of the compressible soil. From the pressure-voids ratio curve, Figure 2-A, the voids ratios at the pressures listed in column 1, Table 1, are determined and placed in column 2 as shown. The voids ratios of the inactive and the total pressures are then averaged separately as e_i, the initial voids ratio, and e_f, the final voids ratio, and placed in column 3.

With these data the total settlement Q

may now be determined from the expression

$$Q=\frac{e_i-e_f}{1+e_i}\times D$$

in which

D = thickness of compressible layer at e_i = 30 ft. in this case

By substitution

$$Q=\frac{1.33-1.05}{1+1.33}\times 30=3.6 \text{ ft.}$$

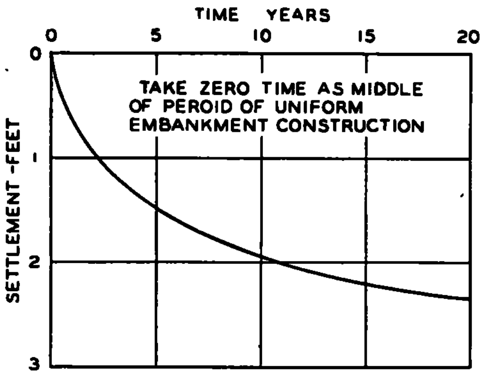


Figure 3. Estimated Rate of Settlement of Embankment

Rate of Settlement.

From the time-consolidation curve, Figure 2-B, the percentages of consolidation which the soil sample underwent at several arbitrarily selected times are listed in columns 1 and 2, Table 2. These percentages of the total consolidation of 3.6 ft. of the compressible layer may then be determined as in column 3. The estimated periods of time for the compressible layer to consolidate the different amounts shown are then computed from the relation

$$t_D=t_d\left(\frac{D_0}{d_0}\right)^2$$

in which

t_D = time required for the soil layer to consolidate a given percentage.

t_d = time required for the soil sample to consolidate the same percentage.

d_0 = thickness of soil sample at voids ratio, e = 0. It is furnished as part of the test data. See Figure 2-B.

D_0 = thickness of soil layer at e = 0. It equals $\frac{D}{1+e_i}$, equals $\frac{30}{1+1.33}$ = 12.9 ft.

Thus the time for the soil layer to consolidate the same percentage that required 1 min. for the soil sample is

TABLE 2

Rate of Settlement Data

Soil sample data Figure 2-A		Soil layer data	
Period of consolidation	Percentage of consolidation	Amount of consolidation	Period of time required
Min.	%	Ft.	Years
¼	12	0.43	0.6
½	20	0.72	1.2
1	29	1.04	2.4
2	40	1.44	4.8
4	53	1.91	9.6
8	65	2.34	19.2

$$t_D=1\times\left(\frac{12.9}{0.0115}\right)^2=1{,}260{,}000 \text{ min.}$$
$$=875 \text{ days}=2.4 \text{ years.}$$

Then the time corresponding to 2 min. is 2×2.4 = 4.8 years, etc. This gives column 4 Table 2 which with column 3 is used to construct a time-settlement graph for the embankment as shown in Figure 3.

The validity of such an analysis is indicated by work reported in *Public Roads*, February 1936 (16). Here the moisture contents of samples obtained from the soft foundation soil beneath a hydraulic fill placed at Four Mile Run on the Mount Vernon Memorial Highway were determined in the laboratory and were also computed according to the theory of consolidation. The results given in Table 3 show that the average of the computed values varied not more than

5 percent from the average of the determined values.

The foregoing gives the complete settlement analysis step by step for compressible undersoil with two drainage faces as in Figure 1. If the compressible layer has but one drainage face instead, the periods of time required for given percentages of settlement computed on the basis of two drainage faces must be increased by multiplication with the coefficients given in Table 4.

THE INFLUENCE OF SHEAR RESISTANCE ON THE STABILITY OF SOIL

The following discussion supplements material included under the title "The Trend of Soil Testing" presented at the Eighteenth Annual Meeting of the Highway Research Board, with special reference to the discussions of "The Theory of Shear Tests" and "Only Part of Soil Strength is Usable." Reference is made also to information on direct shear tests presented at the annual meeting of the American Society for Testing Materials, 1938 (17), and on the stabilometer tests published in the September 1939 issue of *Public Roads* (18).

The data, Table 5, are presented principally to illustrate the influence of the cohesion, c, and the angle of internal friction, ϕ, of soil on its supporting value, etc., computed from theory based upon Coulomb's classical conception of its shear resistance.

The deformations, m, and the corre-

TABLE 3

MOISTURE CONTENT DATA—FOUR MILE RUN INVESTIGATION

Undisturbed core number	Moisture content of the foundation soil	
	Determined by test	Computed from theory
	Percent	*Percent*
1	108	123
2	76	60
5	125	123
6	79	77
9	105	86
10	116	109
Average	101	96

TABLE 4

EFFECT OF BOUNDARY CONDITIONS ON RATE OF CONSOLIDATION

Consolidating pressure at drainage face ÷ consolidating pressure at impervious face	Time of consolidation with one drainage face÷time with two drainage faces for the following percentages of consolidation:					
	5	20	40	60	80	95
0	50	13	7.0	5.4	4.5	4 3
0 2	17	9.2	6.4	4.8	4 4	4 2
0.4	9.2	6.8	5 6	4.5	4 3	4 2
0.6	6.0	5.4	4 8	4.4	4.2	4 1
0.8	4 6	4 5	4 4	4.2	4.1	4.1
0.9	4.2	4.2	4.1	4 1	4.0	4.0
1.0	4.0	4.0	4.0	4.0	4.0	4 0
1.5	3.5	3.0	3.6	3.7	3.8	3 9
2	3.1	2 5	3.3	3.5	3 7	3 9
3	2.6	2.1	2.9	3 3	3 6	3.8
5	2.1	1.7	2.5	3.0	3.4	3 8
10	1 7	1.4	2.1	2.8	3.3	3 7
∞	1.6	1 1	1.6	2.3	3 1	3.6

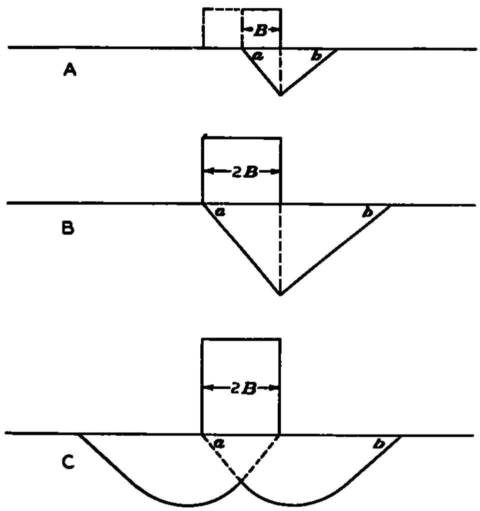
shown as 3 in Table 5 and in which

L = total lateral pressure per foot width of wall.
h = height of wall.
w = weight of earth per cubic foot.

The passive earth pressure P is expressed by the formula

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

shown as 4 in Table 5.



$B = \frac{1}{2}$ WIDTH OF EMBANKMENT

Figure 6. Surfaces of Slip Under Embankments ($\phi = 10$)

Supporting Value of Soil.

Formulas 3 and 4 it will be noted consist of two parts, a first term, which depends upon the weight of the earth in the wedge and is independent of the cohesion, c, and a second term which depends upon the cohesion c and is independent of the weight of the earth in the wedge.

If now we refer to the computed values of both L and P in Table 5, it will be noted that those for P are much larger than those for L. The additional pressure, q, which can be applied to the surface of the earth producing active pressure, to just equalize the excess of P over L is obtained by means of the formula

$$q = (P - L)\tan^2 a.$$

By substitution of the values of P and L, and simplifying, there results the expression

$$q = wB\frac{\tan^4 a - 1}{2\cot a} + c\frac{2\tan a}{\cos^2 a}$$

which is shown as formula 5, Table 5. (See diagram, Fig. 6-A) and in which $B = \frac{1}{2}$ width of loaded strip.

If it were desired to support a load, q, which would cause the active pressure to exceed the passive resistance, it would become necessary to apply a surcharge, q_1, to the surface of the earth furnishing the passive resistance, the magnitude of q_1, being given by the expression

$$q_1 \tan^2 a = q\tan^2 b + L - P$$

If now this equation is simplified there results the expression

$$q = wB\frac{\tan^4 a - 1}{2\cot a} + c\frac{2\tan a}{\cos^2 a} + q_1 \tan^4 a$$

and this will be recognized as an expression of the conception of supporting value which was first published in *Public Roads,* May 1929 (2).

Since the width B, can be taken as one-half the width of a loaded strip, the formula has been suggested for use in estimating the supporting value of the foundation soil of an embankment which fails by breaking in the middle.

If the embankment can be considered rigid enough to settle as a unit but fail by tilting to either side the formula becomes as follows:

$$q = wB\frac{\tan^4 a - 1}{\cot a} + c\frac{2\tan a}{\cos^2 a} + q_1 \tan^4 a.$$

If no surcharge is to be used, only the first two terms of the supporting value formulas are used. If the soil is cohesionless and without surcharge, only the first term is used; and if the soil is frictionless and without surcharge, only the second term.

Formula 6, Table 5, gives values of supporting value for an embankment which fails as a unit by tilting, and where no surcharge is to be used. See diagram, Figure 6-B.

When it can be considered that the embankment will act as a unit but settle vertically without tilting, there would be warrant for using twice the values obtained by means of formula 6, but in such case the more conservative values furnished by use of the Prandtl formula (15, 20), No. 7, Table 5, seem more appropriate. See diagram, Figure 6-C.

SETTLEMENTS OF EMBANKMENT DUE TO LATERAL DISPLACEMENT OF THE UNDERSOIL

If the distortion of soil required to build up values of c and ϕ behind retaining walls and beneath embankments were proportional to the deformations required in the test samples (1, 17), expressions for use in estimating the earth movements indicated in Figure 5, such as the following could be derived:

For wall moving outward due to active pressure—

Average horizontal displacement of wall in inches (Formula 8, Table 5)

$$L_l = .06mh \sin^2 b$$

Average settlement of earth behind wall (Formula 9, Table 5)

$$L_v = .06mh \sin b \cos b.$$

How much weight can be given the values of the deformations computed from these formulas as shown in Table 5 is not known. The principal purpose of including them here is to give at least a qualitative conception of the slow distortions which stressed plastic soil undergoes in the building up or mobilizing of its shear strength, and thus explain the slow sway or displacement of retaining walls and bridge abutments which have been designated as "creep." Also, they serve to emphasize the risk involved in the use of shear resistances in design which require large deformations of the soil to mobilize.

Regardless of the values shown in the last column, Table 5, which are based on the ultimate c and ϕ of the soil, structures built on the soil can safely undergo only a limited amount of settlement, L_v, and retaining walls, but a limited amount of lateral displacement L_l. If it were considered that the allowable values for L_v and L_l were those produced by sample deformations of m = 2 percent, then the c and ϕ with the corresponding computed values, Table 5, would be used. Such c and ϕ for the soil used in the foregoing demonstrations equal approximately $\frac{1}{3}$ the ultimate values and, therefore, it can be considered that in their use a factor of safety of 3 with respect to the total strength was employed.

Design of Slopes.

The safe slope of embankments placed on soft undersoil depends upon the shear resistance of the foundation soil; and of those placed upon good undersoil, upon the shear resistance of the fill material.

An expression which shows the relation of safe slope of embankment to the shear resistance of relatively thin layers of soft undersoil was published by the Ohio Department of Highways, 1937 (25). With modifications it is as follows:

$$S = \frac{Dw_s}{2c + Dw_u \tan \phi}$$

which is shown as formula 10, Table 5, and in which

S = slope = ratio of run to rise.
D = thickness of soft undersoil.
w_s = unit weight of fill material.
w_u = unit weight of undersoil.

In computing the values of S for the three thicknesses of the soft undersoil, Table 5, it was assumed that $w_s = w_u = 100$ lb. per cu. ft.

For determining the safe slopes with respect to the shear strength of the embankment material, a very laborious graphical method based upon the conceptions of Fellenius and others was described in *Public Roads*, December 1929 (19). Later the method was modified and very much simplified by D. W. safe slopes of embankments, the graphs, Figure 7, were introduced here to illustrate the enormous saving in time spent on computations, made possible by the use of such methods, which are equally applicable to the use of all the relations

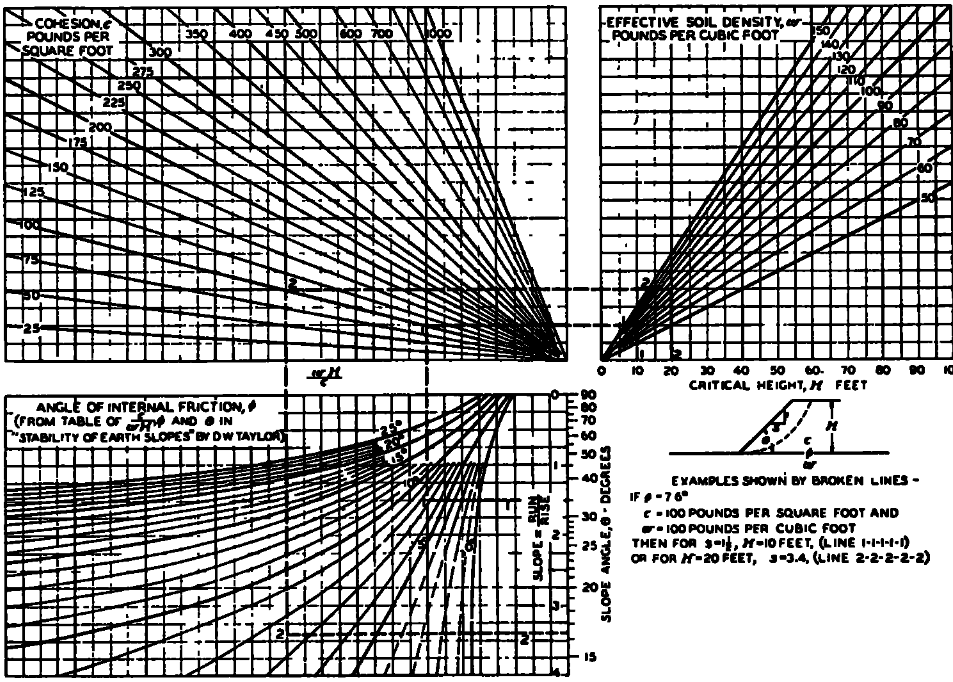


Figure 7. Chart for Critical Height of Slopes

Taylor (26). A further simplification by the junior author is made possible by use of the graphs, Figure 7.

When w, c and ϕ are known, the critical height of an embankment for any desired slope may be determined by working around through the three graphs clockwise; and the critical slope for any desired height, by working through the graphs counterclockwise. Values obtained in this manner for critical heights at a slope of $1\frac{1}{2}$:1; and the critical slopes for a height of embankment, H=20 ft., are shown in Table 5.

Additive to the information furnished on the effect of c and ϕ relative to the shown in Table 5, and to the others commonly used in soil work.

SUMMARIZATION

Thus it can be said that enviable progress has been made in the utilization of the indicator and the compaction test data. And the outlook in the field of soil mechanics as applicable to the construction of embankments is not so dark. With the exception of simplifications in the apparatus, the essential features of the consolidation test are for all practical purposes the same as when originally proposed by Doctor Terzaghi, and may well be standardized now for highway purposes. Within the range of the uniformity of the soil, it is indicated that for those conditions of soil profile and loaded area which approach the conditions under which the samples are tested in the laboratory, estimates of settlement of structures due to consolidation of the undersoil made in accordance with the theory of consolidation, are entirely valid, and are especially applicable to embankments due to their broad loaded areas. As the ratio of loaded area to depth of compressible soil decreases, the breach between field and test conditions widens, and the validity of the settlement estimates can be expected to correspondingly diminish.

Progress in shear tests has consisted principally in the development of suitable apparatus and a better understanding of the errors involved in the improper interpretation of test data. It is evident that the time is not yet ripe for the standardization of shear test procedures, although ideas are rapidly crystallizing with respect to simplified types of direct shear and stabilometer test apparatus. The validity of formulas based on Coloumb's conception of shear resistance cannot be determined until more adequate information is obtained on the performance of stressed earth masses.

It is appreciated that the use of partial values of c and ϕ introduces an error, for, in the theories expressed by the formulas a condition of ultimate failure of the soil is assumed. On the other hand, values of c and ϕ indicative of the ultimate strength of soil obviously could not be used because of the prohibitive deformations of the soil involved, even if they could be obtained. But no criterion for the determination of the c's and ϕ's indicative of the ultimate failure of samples during test has as yet been established, whereas, there is no difficulty in obtaining c's and ϕ's indicative of shear resistances of the samples at equal deformations.

It is expected that charts such as shown in Figure 7 to substitute for the formulas in the making of computations will greatly expedite a more general use of the theories. The next step is to encourage more controlled investigations such as have been in progress at the University of Michigan (27) and the Iowa State College (28, 29); and the applications of the theories in practice as was so well exemplified by the Bay Bridge approaches and the San Luis Obispo projects in California (1). Along with these the study of building settlement by Charles Terzaghi (14) and Gregory P. Tschebotareff (30); and of large bridges by W. P. Kimball (31) are furnishing valuable supplementary information.

In conclusion a word should be said on the desirability of utilizing controlled methods of compaction in embankment construction. Investigations disclose that embankments constructed under rigid specifications but before the advent of moisture control have densities ranging from 70 percent up to 100 percent of the Proctor maximum. By means of shear test data obtained in the Administration's laboratory and the chart Figure 7, it can be shown that unusually bad soils which have critical heights of not more than 10 ft. when compacted at 77 percent Proctor maximum may be used in embankments up to 60 ft. high at 90 percent, and 120 ft. high at 100 percent Proctor density. It is further indicated that the attainment of the required high densities in embankment construction does not necessarily mean increase in the cost of their construction.

The advantages of this method of construction are summarized by Thos. E. Stanton (32) as follows:

"Standard California practice requires that embankments be constructed and consolidated in layers not more than 8 in. thick before compaction and that each layer be consolidated by rolling to a relative compaction of not less than 90 per cent (California maximum) before subsequent layers are placed. By this method combined with rigid control over the moisture content it has been found possible to construct embankments to any height without subsequent appreciable settlement.

"Except for the fact that the specifications provide that no adobe shall be placed in embankments above one foot below the profile grade, there is no restriction in California specifications regarding the nature of the material which can be used.

"It has been found possible to secure relative compactions of 90 to 100 per cent with even the difficultly consolidated California adobe soils of the A-7 type, provided the construction procedure laid down in the specifications is rigidly followed.

"A number of major projects, each with many large fill and embankment quantities totalling in excess of a million cubic yards have been constructed from materials consisting largely of clay-shale of the A-7 soil type. Relative density in excess of 90 percent and a dry density of approximately 125 pounds per cubic foot were secured. One such fill, 130 feet high on the Cuesta Grade was constructed in 1937. Notwithstanding a severe wet winter subsequent to the construction of the fill, no appreciable settlement has taken place during the first year.

"Three projects constructed in 1929 in accordance with the California specifications have been under observation for nine years. The embankments ranged from 30 to 110 feet in height. During nine years of service no appreciable settlement of these large fills has occurred."

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DISCUSSION ON SOILS INVESTIGATIONS

MR. J. J. FORRER, *Virginia Department of Highways:* How many States are putting into practice the method described by Mr. Hogentogler?

MR. HOGENTOGLER: According to the compendium last year there are 24 States who reported making the Proctor tests. Both California and Ohio construct fills by controlled compaction methods.

How many of the other States that reported facilities for making compaction tests actually carry out all the details in construction, I do not know, but this I believe is true—that even though they have not gone so far as the two States mentioned above in actual control, just an appreciation of the idea that is back of these tests has done wonders toward improving the stability of fills in ordinary construction.

MR. FORRER: What are the additional costs on a yardage basis due to using the optimum moisture in fill construction?

MR. HOGENTOGLER: I cannot answer that but a discussion came up a year or two ago, the trend of which was that the contractor saves money by utilizing controlled methods of compaction during construction. Among other things, the grades constructed with sheepsfoot rollers have been compacted so highly that he does not have to wait for them to dry out after rain. Professor Casagrande brought out some illustrations in New Hampshire in which it cost the contractor less to utilize the compaction method than if he had not used it.

MR. J. J. PREECE, *National Park Service:* Maybe I can add a little more information to the last question. We are now constructing or reconstructing the C & O canal and in that we have some embankment work to do. The specifications were written up with laboratory control for the embankment construction. The first reaction of the contractors who were expecting to bid on that job was complete consternation. They threw up their hands, one of them who had done a great deal of work refused to bid at all and they came back at me and asked me if I had not written up something that was just so impractical that we could not expect to carry it through. I did not think that we had. As a matter of fact I think we had written up an extremely liberal specification.

We required no particular water control. We provide the Water Content—Compacted Unit Weight relation curve, and we tell the contractor we want compaction to a certain minimum unit weight. Now it is well known that for a given water content you get a higher compacted unit weight by more work. The unit weight which is obtained with a certain amount of work at optimum water represents the least work that must be applied in order to obtain that unit weight. We specified only the minimum compacted unit weight and told the contractor—you can change your water content to get this with less rolling or you can leave your water content and get the compaction by increased rolling, whichever is cheaper in any particular case. That I believe is rather a liberal provision. Finally the bids came in. To get the bids it was necessary to do some missionary work and convince the contractors that we were not going to tie them up in a knot. They finally bid on the work, relying on what we had told them we would do. The work has been going on now for some time and recently I asked the contractor how he felt about this control and he is quite enthusiastic about it.

I don't believe that any contractor who has any conception of what this soil mechanics laboratory control is all about, or has any idea of what he is shooting at and what the laboratories are trying to do, will feel that he is being put to any disadvantage. As a matter of fact, we are building all over the United States—although not by contract—most of it is CCC work. While we have not run into the bidding factor, we have encountered the opposition of men who have been constructing for a long time. They feel that they know how to do it and do not want any waste for refinements placed in their path. They do not want any fancy theories. Nevertheless, I don't know of a single one who has tried it who is not completely enthusiastic about it and believes that it not only permits him to keep a better control, and do better work, but also results in definitely lower costs.

C. M. Upham, *American Road Builders Association:* Moisture control was not specified on the Pennsylvania Turnpike. It was later recommended that moisture control should be used and when it was found from the contractor how much this additional operation would cost, it was found to be prohibitive. The ridiculous prices which were asked by the contractor made it impossible to use the optimum moisture content method. The contractor wanted from two to eight cents per gallon. You could buy gasoline cheaper than that. The high price was due to the fact that the moisture requirement was not included in the specifications in the beginning.

Prof. M. G. Spangler, *Iowa State College:* I have in mind a matter which might have a place in this discussion; that is the problem of subsidence of foundation material under high embankments. No matter how well the embankment has been compacted, we are still up against the matter of compression or consolidation of the natural ground or foundation material. In some cases that is a real factor. I myself have measured subsidences under an embankment 65 feet high which amounted to 2 feet, 8 inches at the maximum point.

I recall a story which Dean Marston tells in connection with his work on the Board of Review of the Mississippi Flood Control Works, in which he cited an example of a levee under which the subsidence had been as much as 80 feet. This of course is an extreme case, but subsidences measurable in feet are not uncommon and as embankments become higher due to reduced costs of earth-moving and grade and alignment requirements for adequate sight distances, the problem of subsidence is bound to become more noticeable.

Mr. Hogentogler: In that connection see the 1937 Civil Engineers Symposium of practical application of soil mechanics. The point is made by one contributor that at the present time about 85 percent of all failures of embankments are due to foundation trouble.

Mr. V. J. Brown, *Roads and Streets:* As I understand soil mechanics in its application, we have to analyze each type of soil independently. When we come to build a big embankment and we dig down into the cut out of which it is to be built, we are going to find several types of soil, shale and rock. This is especially true for deep cuts—and these are the kind that furnish the material for big embankments. It occurs to me that at the present time we are building our embankments upside down. We are taking the weathered soil from the top of the cut and putting it on the bottom of the fill. When we strike hard rock near the bottom of the cut, we put that on the top of the fill. It we were to shoot the full face of the deep cut, we would get a mixture of materials to place in the fill that would be uniform. What I am wondering about is, could our soils mechanics analyses be applied to that method of construction giving us definite results? In other words, could soil mechanics embankment control methods be applied when a construction method is used in which the full face of cut is shot and the mixed materials hauled into the embankment?

Mr. Hogentogler: Assuming that you have a combination of soils in which the optimum is changing and you place your fill at the optimum moisture content of one of the soils, there is the likelihood that the others will be adequately compacted.

Question: In line with Mr. Brown's query, approximately what size soil do you think was used?

Mr. Hogentogler: The Proctor test has to do with clay soils and is applicable whenever there is sufficient clay to float the granular material.

Mr. V. J. Brown: Has there been a correlation established in compaction between the tests on the soil as you make it and the results in the field where mixed soils, such as topsoil and rock, or shale and topsoil, etc., constitute the embankment?

Mr. Hogentogler: The compaction tests are used in the construction of topsoil and stabilized roads, although there is no relation between the test results and the number of trips of the compacting equipment.

Mr. V. J. Brown: I have talked with manufacturers in regard to compaction. Whenever I talk about the inefficiency of the sheepsfoot roller they come back with the old argument that for every soil there is a definite optimum water content which is related to the weights of the rollers and the amount of work done. There is also a different compaction density for different equipment and I maintain that we should eliminate the sheepsfoot roller in favor of some more efficient unit. We should have a unit that would give results comparable to our standard test method of compaction, which would be some kind of a machine that would hit every square inch of the surface uniformly.

Mr. Preece: The first thing that any laboratory is going to do with regard to soil mechanics is to set up for itself a procedure which is correlated to the work it is going to direct; that is the first requirement. The laboratory that is off by itself and is turning out page after page of data and is not interested in the job, is not going to get anywhere. We started some time ago to determine a method by which we could be sure our laboratory work was definitely correlated with our work in the field. I can tell you this and we have been using it for several years. In the meantime we have probably built about 45 to 50 dams, some of them going as high as 125 feet. We do have a definite relation, that is, on the curve that we give to the field.

PROF. DONALD W. TAYLOR, *Massachusetts Institute of Technology:* During the past decade there has been much progress toward a better understanding of the factors upon which the shearing strength and related characteristics of clays depend. Also there has been steady improvement in the types of apparatus which have been devised for determining the properties of clay soils. However, many of the complex phenomena which affect these properties are still but partly understood. This is especially true of the shearing strength, the most complex of soil properties and the one to which this discussion will be mainly devoted.

During the past few years numerous soil laboratories have been established by agencies anxious to make practical use of the latest developments in soil mechanics. Demands for the setting up of standardized procedures for various soil tests have been growing. At the same time there has been much agitation for the establishment of rules for interpreting shear test data and for recommended safety factors to be used in design.

There are two strongly contrasting beliefs with regard to the advisability of standardizations and the setting up of such rules. Regardless of which belief an engineer subscribes to, he will do well to understand both points of view and to keep both in mind in discussions of the subject.

First is the belief that in such an involved subject as shear in clays there are so many variables upon which the results depend, that, so long as new ideas are continually being developed which lead to alterations of those they replace, standardization of testing methods is undesirable. Any simple testing method cannot truly reflect all possible conditions of the many variables. Also, standardization may, in many cases, place a false appearance of dependability on test results. It will probably be generally agreed that for the best ultimate understanding of a complex problem, there is much logic in this viewpoint.

The second belief is frequently spoken of as a practical viewpoint. Engineering practice cannot wait for the development of perfect methods but demands at any time the best procedure which can at the time be set up. Trial in use will point out certain flaws quicker than can be found in any other way.

It is probable that most engineers will agree that both points of view are somewhat extreme and that the great majority of cases must be considered separately, each on its own merits. It is also probable that the standardization of many of the common soil tests is on the whole desirable. However, the time has not yet been reached when the standardization of shear tests on clays can be expected to be at all permanent and it is believed that the disadvantages of attempts toward standardization greatly outweigh the advantages. This is well illustrated by the current situation with regard to shearing strength investigations by triaxial or cylindrical compression apparatus, a type which has jumped into prominence in the last two or three years. Rapid strides have been made in methods of investigation on sands as shown by the new concepts of critical void ratios. Extension of this work to clays has only just begun.

One of the questions most frequently asked in soil mechanics work is the following. When tests for the determination of shearing strength have been conducted on samples from a given site, how does one determine from the test data the shearing strength values which should be used in the design or analysis of an embankment at the site, and if a design what factor of safety should be used? Surely, if a definite and dependable list of rules could be given they would be of great help to many engineers. Unfortunately, the question is much too complex to permit the setting up of definite general rules. If general rules were possible they could hardly be covered in one short paper. As has often been pointed out in foundation work there are conditions which are peculiar to any given job and as a general policy, each job unless it be a relatively simple one must be treated individually.

Since it is not possible to give a treatment which is as definite and general as would be desirable, the following discussion is limited to a few remarks on the subjects of factors of safety and factors affecting the shearing strength of clays.

Since failures may occur in many different ways, there are many possible types of factor of safety. In stability analyses of embankments it is especially important that the term "factor of safety" should not be used until its meaning for the given case is clearly defined. The question arises as to what shall be the criterion of failure; shall failure be associated with the condition wherein the most highly stressed point just reaches its limit of strength, or shall it be considered that failure has not occurred until the point which first attains the failure state has undergone further strain, at constant or decreased strength, until all reserve of strength of surrounding points has been expended?

Both concepts have been widely used. If a soil is of sufficient strength so that its most dangerously stressed point has a shearing strength which exceeds its shear stress by 50 percent, it may be said to have a "point factor of safety with respect to shearing strength" of 1.5. Complete failure in shear, however, would not occur by merely overstressing one point. Rather, all strength along a continuous surface would have to be overcome and the factor of safety which is in commonest use in stability analysis is sometimes called the "factor of safety with respect to total strength." It may be described as the ratio between total shearing strength and total applied shear on the continuous surface most susceptible to failure.

Field investigations by the Swedish Geotechnical Commission and other organizations on actual slides, supplemented by laboratory tests for shearing strength, have pointed out that it is seldom possible to use large factors of safety with respect to total strength in embankment design. Typical practical values are of the order of perhaps 1.1 to 1.5 while if higher values are attempted, prohibitively large increases of yardage may result. An illustration given by the writer at the Highway Research Board meeting of a year ago indicated this type of situation for a foundation failure of a highway fill. In this case a factor of more than 1.2 in the redesigned section would have required excessive yardage.

However, the following illustrates an important point. Let it be assumed that in a typical case the factor of safety with respect to strength is $1\frac{1}{3}$ and that in this same case the shearing strength determinations have a probable error of about 30 percent. This is, perhaps, a rough idea of the usual accuracy for tests on some types of soils. Here it is seen that the margin of safety is of the same order of magnitude as the probable error in the strength determinations, a more or less typical situation. This indicates the very practical fact that efforts along lines such as combined shearing tests and field investigations of failures, aiming at improved understanding of shearing strength values, are much more productive than are efforts to decide what factor of safety should be used.

The factors which affect the shearing strength of clay samples are numerous and complicated but they may be grouped under the following headings:

1. Porosity
2. Pressure
3. Strength due to natural structure
4. Condition of consolidation and other time factors
5. Degree of progressive action.

A thorough discussion of these items would fill a large textbook. The following is an attempt to outline the entire complex subject in a single paragraph.

While porosity has long been known to be an important factor, research by M. J. Hvorslev has demonstrated that an expression in terms of the porosity is also capable of expressing the effects of the past stress history of the sample. The total pressure has always been recognized as a factor of importance. Also the strength which is given by the natural structure and which may be largely lost if the structure is disturbed in sampling is a most important item. As a sample undergoes shear, the shear distortion in itself causes disturbance of structure and to a degree softens the sample, thus transferring part of the pressure on the sample into excess pressure in the pore water. Therefore, in addition to the first three factors mentioned, there must be added the fourth, "degree of consolidation." This factor will in turn depend upon many things, including properties of the natural structure, the speed at which the test is run and the size of the sample and other factors influencing the facilities for drainage of pore water from the sample. Other time effects such as a resistance akin to viscosity may occur, especially if shear is forced to take place quickly. Lastly, unless the soil is under uniform stress and strength conditions throughout, which probably is never the case in nature and which holds only approximately even in the best of laboratory testing methods, there will be progressive action, or weakening of strength caused by load concentrations which successively cause overstressing which passes from point to point.

The claim has sometimes been forwarded that soil mechanics can remove guesswork from foundation and embankment problems. To a degree this is true but there must always be a much larger degree of uncertainty in problems involving soils than there is in most engineering problems. In removing guesswork there are two ways in which soil mechanics has value. First, by a clearer understanding of the physical phenomena which govern the action of soils, designing becomes more rational even when soil testing is not resorted to for each specific problem. Soil testing in the form of fundamental research, however, contributes greatly to this first item which could be called qualitative soil mechanics. Secondly, laboratory determination of soil properties for use in analysis and design has proven to be of practical value in some types of problems. These two factors apply not alone to soil mechanics; a similar situation exists in the closely related field of geology. Development of both factors is in every way desirable. It is idle to speculate as to which is more important as both must be carried on together for best results toward practical and sound development.