SOIL MECHANICS

FOR

CIVIL ENGINEERS

BY

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PREFACE

This book has been written from a conviction that its subject matter is basic for all civil engineers and architects. Its purpose is that of a work of post-graduate standard, but it has been written also for those without any previous knowledge of the subject. It is not intended for the specialist, and with a technical public in view, its chapters have been arranged so as to be more or less self-contained.

Complicated mathematical proofs have been omitted, and every effort has been made to keep the mathematical portions as simple and self-explanatory as possible, although for the benefit of those readers who wish to delve more deeply, a full set of references has been included.

Considerable stress has been laid on applications of soil mechanics to highway engineering problems, partly because the highway engineer is especially concerned with foundation problems, and partly because of the work upon which the writer is chiefly engaged. But the subject concerns many engineers other than highway engineers, and the scope of the book has been extended so as to cover as much as possible of other engineers' foundation problems.

It should be mentioned that a large literature is constantly appearing dealing with this subject, and that no finality can be claimed for much of what is offered in the present volume. It is, however, thought that the latter should be of real assistance to those who have not yet had an opportunity of coping with the voluminous literature available already, a great deal of which has been made use of herein.

Acknowledgments

It is desired to acknowledge very gratefully the co-operation and assistance which the writer has received from many sources in the compilation of this volume, including the Council and Principal of the University of the Witwatersrand, Johannesburg, Professor W. G. Sutton, Head of the Department of Civil Engineering in that University; Mr. W. H. King, and several past and present research workers in that Department; the National Road Board and the South African Iron and Steel Corporation, South Africa, whose financial support has made the work of the writer possible; the South African Torbanite Mining and Refining Company; the Automobile Association of South Africa; the Cement and Concrete Association of South Africa, who, with the South African Iron and Steel Corporation, have made scholarships in highway engineering available; the Highway Research Board, U.S.A., and many American universities and research institutes; the Provincial Road Engineers of the Union of South Africa; the Director of the Road Research Laboratory, Harmondsworth, for permission to use work produced by his Department; the Controller of H.M. Stationery Office, London, for permission to reproduce Figure 54A; the Director of the Building Research Station, Watford; Mr. D. E. Davis, a former student, who prepared the figures in the book; Mrs. M. A. Davis, who painted the colour pictures of clays shown in the frontispiece, and last, but not least, Mrs. R. G. Knight, who helped very greatly in checking the work chapter by chapter. To each and all, the writer tenders his sincerest thanks.

CONTENTS

FOREWORD

The Engineering Aspects of Soil Mechanics denote the study of the engineering properties of soil, i.e., the capacity of soil to resist external loads or the forces induced in it by such loads as roads, railways, aerodrome runways, bridges, dams and buildings of various kinds. Unless they are built on hard rock, all structures of these kinds have ultimately to be supported by soil, and unless the effects of loads on soils are fully understood, either waste of money may occur from the use of too conservative a design, or failure of the superincumbent structure from settlement may result. Examples of failures of structures, due either to the then undeveloped state of soil mechanics as a subject or to ignorance of its principles, are the Leaning Tower of Pisa in Italy, the failure of a large cutting in clay soil on the Southern Railway near Sevenoaks, portions of the cuttings of the Panama Canal, a portion of the Chingford Reservoir dam near London, and many foundations of roads, railway embankments and aerodrome runways in various parts of the world.

Failures of this kind should be avoidable if the principles of soil mechanics are understood and utilised in the design of the structure to be carried by the soil; in fact, it is no exaggeration to say that the study of soil mechanics is fast becoming a basic subject in the scientific training of civil and structural engineers and architects.

Soil mechanics is no new science. Thomas Telford, a pioneer in highway engineering, made use of it, for in his road specifications he laid down that " a vertical section should be made, and the nature of the soil, to be ascertained by boring, over which each apparently favourable line passes, should be shown ; for it is by this alone that it can be determined and calculated at what inclination the slopes in cutting and of embankments will stand. If bogs or morasses are to be passed over, the depth of the peat should be ascertained by boring, and the general inclination of the country for drainage should be marked."

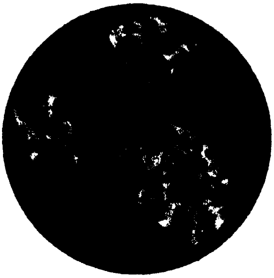
Prior to 1916, when soil tests were suggested by A. Casagrande, civil engineers relied almost entirely on rule-of-thumb methods when considering the loads which could be placed safely upon soils, although long before this A. Coulomb and H. Boussinesq had developed mathematical theories as to the nature of the stresses induced by loads on soils, while W. M. Rankine had dealt also with earth-pressure theories. Later pioneers in this rapidly developing branch of science are K. Terzaghi (Austria), C. Atterberg (Sweden), L. Casagrande (Germany), D P. Krynine (U.S.A.), L. F. Cooling and A. W. Skempton (Great Britain) ; the pace of research thereon has been accelerated greatly by urgent problems caused during World War II, more especially in connection with aerodrome runways.

One of the difficulties encountered in the application of soil mechanics to engineering problems is the variability of soil as a material. Soil is not a manufactured product like iron or steel ; its properties can change in a distance of a few feet, or even sometimes in an inch or so, and this has led to the necessity for detailed soil surveys and subsequent laboratory testing of samples obtained from such surveys, before the design of the structure which the soil has to carry can be embarked upon. Another difficulty is the part played by moisture in the behaviour of soils under load. It can, in fact, be said that moisture is probably the most important single factor governing this subject. Yet another difficulty is the impossibility of reproducing in the laboratory processes of consolidation which have taken hundreds, if not thousands, of years to accomplish under natural conditions. But in spite of all these and other difficulties, the engineering aspects of soil mechanics have now reached a stage when they can be of definite use to the practising engineer or architect.

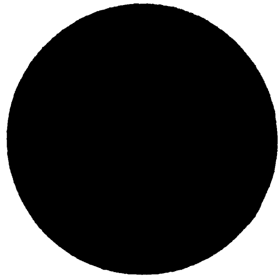
The subject has been described by one of its leading exponents as a new chapter in engineering science, and this is so much the case that it shows every promise of conforming to the great Charter of the Institution of Civil Engineers, London, in becoming yet another means whereby the civil engineer may practice his calling for the use and convenience of man.

ILLUSTRATIONS SHOWING COMMON CLAY MINERALS FOUND IN SOILS

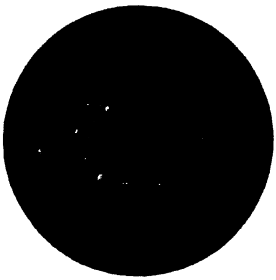
(See Chapter Three)



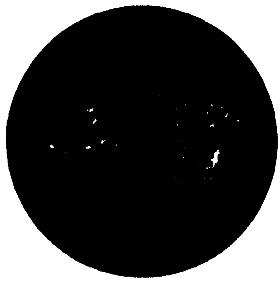
Pure Kaolinite Clay. × 220. Crossed nicols. Showing low polarisation colours and aggregation of crystals.



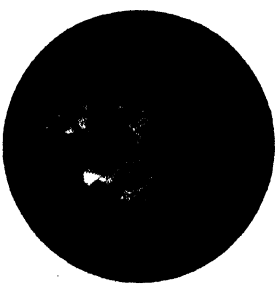
Nontronite Clay (Montmorillonite Group) × 175. Crossed nicols. Showing formation of nontronite from augite (grey patches). The nontronite is a speckled aggregate of crystals, with bright polarisation colours.



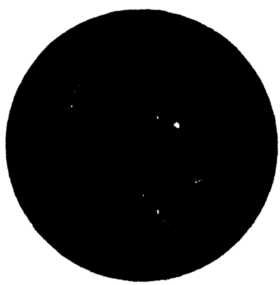
Kaolinite Clay × 400. Crossed nicols. Showing masking of body colours by adherent limonitic coating. The edges of the crystals show the usual characteristics of kaolinite.



Montmorillonite Clay. × 140. Crossed nicols. Showing aggregate of crystals and medium polarisation colours. The refractive index is low and the crystal shape vermiform.



Nontronite Clay × 110. Crossed nicols. Showing high refractive index, bright polarisation colours and undulose extinction.



Pyrophyllite Clay × 120. Crossed nicols. Showing high refractive index, bright polarisation colours and aggregation of needle-shaped crystals.

CHAPTER ONE

THE BASIC MATHEMATICS OF SOIL MECHANICS

This chapter has been planned in such a way as to introduce the senior student and the practising engineer to the fundamental mathematical basis of soil mechanics. The normal mathematical equipment of the civil engineer is assumed, but the theories on which modern soil mechanics are based have been simplified as far as possible so as to bring the subject within the grasp of those who are not familiar with the more advanced mathematical methods employed by specialists.

The basic mathematics now to be dealt with should, if possible, be understood by the reader prior to a study of the later chapters dealing with the mechanical and other properties of soils, but this is not absolutely essential, since the later chapters of this book can be read and used without following the present chapter. It is, however, preferable to try to understand the latter if possible.

Symbols. At an early stage in the compilation of this work an attempt was made to use only so-called accepted codes of symbols throughout, but great divergencies in these codes soon became apparent, and for this reason use has been made of symbols commonly accepted in current published works.

Definitions.

The following definitions should be known by the reader :—

Angle of Repose. The angle of repose of a soil is the maximum angle which the exterior face of a granular mass can make with the horizontal.

Angle of Internal Friction. The angle of internal friction of a soil is the angle, the tangent of which is equal to the ratio of the resistance to sliding on an internal plane to the pressure normal to that plane. Strictly, this term can only be applied to sandy soils, and it should be noted that it is not the same as the angle of repose of a soil.

Cohesion. The cohesion of a soil is the resistance which it offers to external forces by reason of the property whereby the soil particles tend to cohere or to stick together. Strictly, this term can only be applied to clay soils.

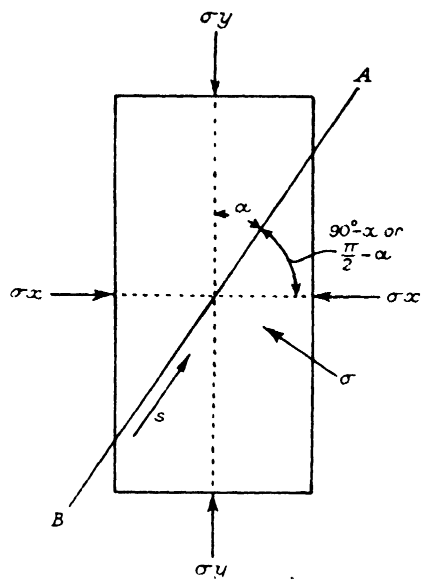


Fig. 1.—Showing normal and shear stresses on plane $A \cdot B$.

The Stress Analysis of Soils.

Mohr's Circle. This is a graphical construction which enables the state of stress at any point in a body to be shown. Consider the equilibrium of any element of a solid subjected to principal stresses σ_x and σ_y. The normal and shear stresses on a plane AB (Fig. 1) are given by

$$\sigma = \frac{\sigma_y + \sigma_x}{2} - \frac{\sigma_y - \sigma_x}{2} . \cos 2a$$

$$\text{and } s = \frac{\sigma_y - \sigma_x}{2} . \sin 2a$$

where a is the angle between plane AB and the direction of the principal stress σ_y.

In Mohr's Circle (Fig. 2), OP is drawn proportional to σ_x, and OS proportional to σ_y, and the circle constructed on PS as diameter, with centre V. From point S, a line SQ is drawn, making an angle a with

SP and cutting the circle at point Q, whence line QR is drawn perpendicular to PS, meeting PS at R. Then

$$QR = \frac{\sigma_y - \sigma_x}{2} . \sin 2\alpha = S$$

$$\text{and} \quad OR = \frac{\sigma_y + \sigma_x}{2} - \frac{\sigma_y - \sigma_x}{2} . \cos 2\alpha = \sigma$$

Thus for any plane AB at an angle α to the Y-axis, there is a point Q on the circle such that the angle $PSQ = \alpha$, QR is proportional to the

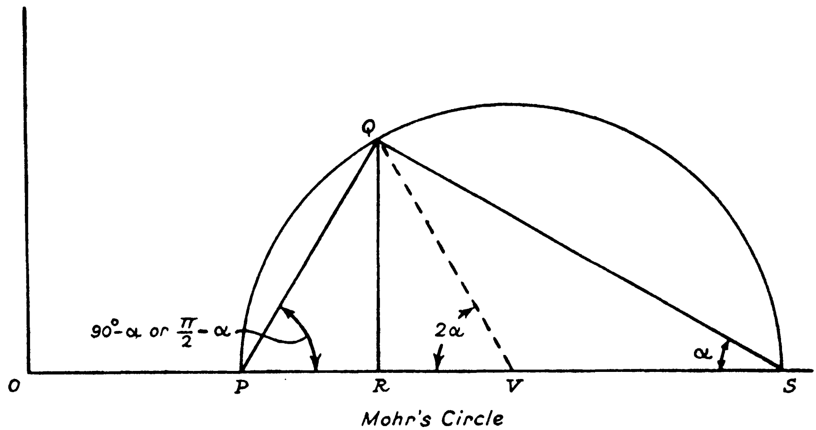


Fig. 2.

shear stress, and OR is proportional to the normal stress on that plane. Hence the state of stress in the element is fully defined.

Envelope of Failure or Line of Rupture.

From a knowledge of the principal stresses acting on a material, it is possible to construct a series of Mohr's Circles for the various ultimate

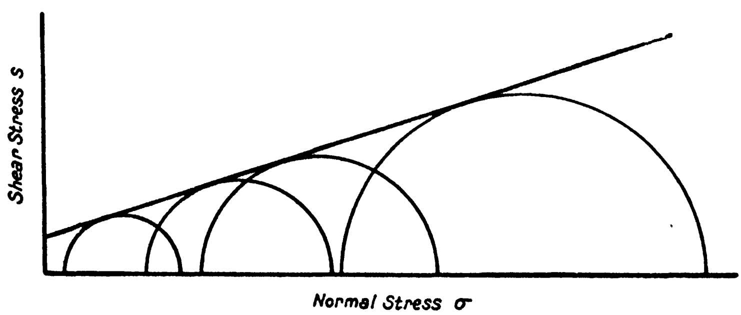


Fig. 3.—Showing envelope from Mohr's Circle.

loadings of a particular soil, and such a series of circles will be found to form a family of circles for which an " envelope " can be drawn to touch the sides of the circles (Fig. 3). This envelope may be curved, but in many cases it is reasonably straight, and then Coulomb's equation applies :—

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

where s = shear strength of the soil,
c = cohesion of the soil,
σ = stress on the sheared face,
ϕ = angle of internal friction of the soil.

For cohesionless soil such as clean sand, the envelope is a straight line passing through the origin (Fig. 4), while for a completely cohesive soil such as clay, the envelope is a straight line parallel to the datum line (Fig. 5). Few soils are either purely cohesionless or purely

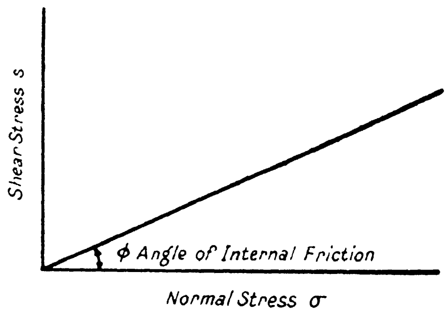


Fig. 4.—Showing envelope for cohesionless soil.

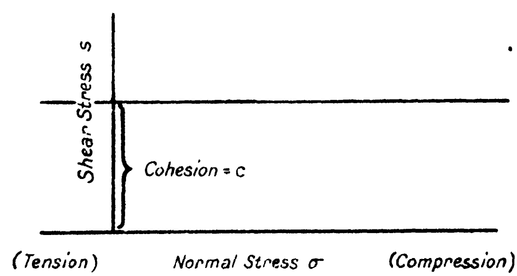


Fig. 5.—Showing envelope for cohesive soil.

cohesive, so that the normal diagram will correspond to Figure 6. Even clean sand will often stand at quite a steep angle in its undisturbed state, showing that it possesses cohesion as well as internal

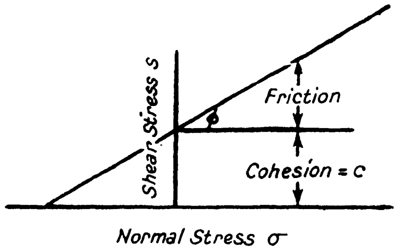


Fig. 6.—Showing envelope for soil possessing both cohesion and internal friction.

friction ; this cohesion persists even when water has dried out from the soil, so that it cannot be attributed entirely to capillary tension of the water filling the voids.

Bearing Capacity of Plastic Soils.

Prandtl's Method for finding the bearing capacity is based on the following assumptions :—

1. That in a strip-loaded medium (Fig. 7), wedges GAF, ABC and BED behave as rigid wedges, while the sectors AFC and BCD deform

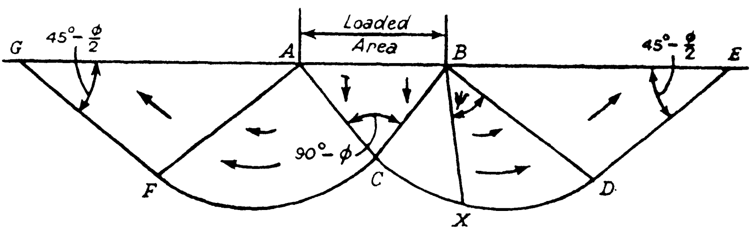


Fig. 7. Showing plastic deformation of soil under loaded strip A-B.

plastically, the angles and lengths in the figure being determined mathematically.

2. That the line of rupture or Mohr's Envelope for the material is a straight line.

3. That in the plastic sectors AFC and BCD, the stresses are constant along the radii vectors such as BX, but vary from one radius vector to another, that is with the angle ψ.

With the above assumptions, the Prandtl formula for the bearing capacity of a strip-loaded soil is :—

$$\text{Bearing Capacity} = q = \frac{c}{\tan\phi}\left[\frac{1+\sin\phi}{1-\sin\phi} \cdot e^{\pi\tan\phi} - 1\right]$$

where c and ϕ have the same meanings as before, and e is the base of Napierian logarithms. From this equation, it will be seen that if $c = 0$, i.e., for a sandy soil with no cohesion, the bearing capacity is also zero.

This result is not borne out by experience. It is known that the unit weight of the soil is of importance, especially in granular soils, and Terzaghi has therefore suggested that a correction should be made in Prandtl's equation whereby $c + c'$ is written in place of c, where

$$c' = hw \tan \phi$$

h being $\dfrac{\text{area of wedges and sectors}}{\text{length } GE}$ and w being the unit weight of soil.

It is possible to allow for the effect of surcharge (i.e., a load outside and adjacent to the loaded strip), for it has been shown that if a pressure

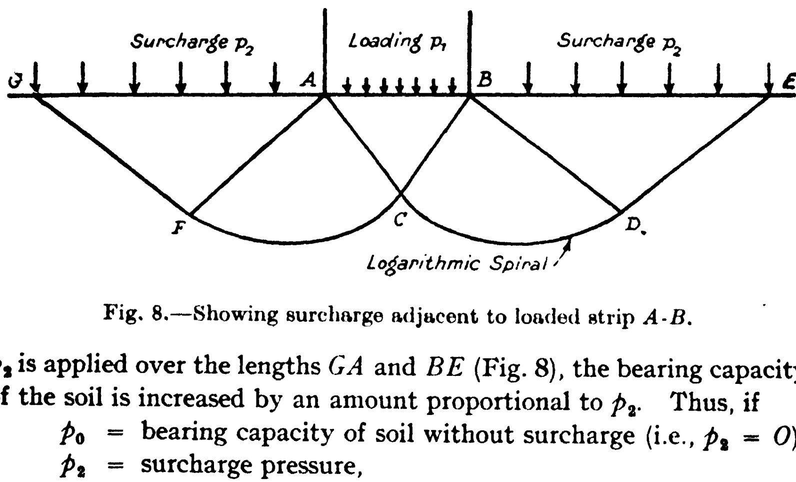


Fig. 8.—Showing surcharge adjacent to loaded strip A-B.

p_2 is applied over the lengths GA and BE (Fig. 8), the bearing capacity of the soil is increased by an amount proportional to p_2. Thus, if

p_0 = bearing capacity of soil without surcharge (i.e., $p_2 = 0$),
p_2 = surcharge pressure,
p_1 = bearing capacity with surcharge p_2

and $$\lambda = \frac{1 + \sin \phi}{1 - \sin \phi} \cdot e^{\pi \tan \phi}$$

then $p_1 = p_0 + \lambda p_2$.

The relation between λ and ϕ is as shown in Fig. 9. It is clear from this figure that the effect of even a small surcharge can be quite considerable in the case of a soil having a large value of ϕ, but negligible for a soil in which the value of ϕ is small.

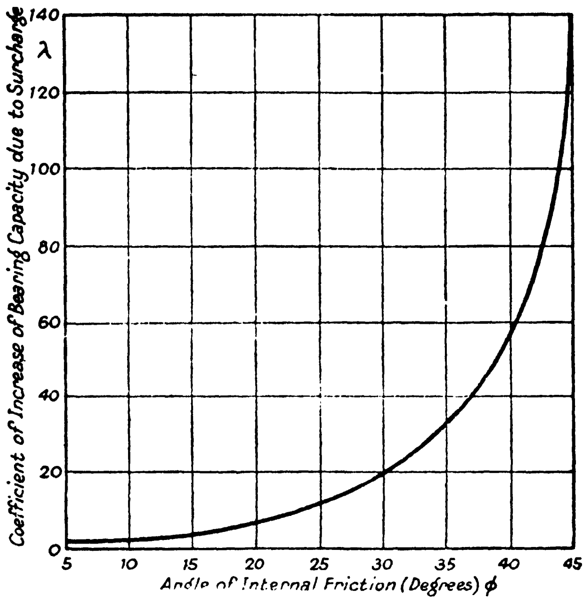


Fig. 9.—Showing effect of increase in angle of internal friction ϕ on increase of bearing capacity due to surcharge.

Terzaghi's Method of calculating the bearing capacity of soils is developed from an elementary wedge theory in which for a width of load equal to $2b$,

$$p = \frac{bw(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{2c}{\tan \beta \sin^2 \beta}$$

where p = unit load which the soil will support without lateral displacement,

w = weight per unit volume of soil,

$\beta = 45° - \dfrac{\phi}{2}$

If a surcharge pressure be added, then

$$p = \frac{bw(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{2c}{\tan \beta \sin^2 \beta} + \frac{p_1}{\tan^4 \beta}.$$

These equations take into account the effect of the size of the bearing area, a factor which is significant in sandy but not in clay soils.

Housel's Perimeter-Shear Method, which is based on the effect of the shearing resistance of the soil along the perimeter of the bearing area, utilises the following relation :

Ultimate bearing capacity $= p = xs + p'$

where x = ratio of perimeter of loaded area to area of loaded area,

s = shear along perimeter of bearing area,

p' = estimated bearing capacity or resistance to compression of soil.

Bearing Capacity of Elastic Soils.

Boussinesq's Theory. Most of the approaches to the various problems of loaded areas at the surface of an elastic medium depend on Boussinesq's solution for the stresses in an elastic medium caused by the application of a point load at its surface. Any elementary area under uniform pressure can be considered as a point load, and hence by applying the principle of superposition the stresses caused by a loaded area can be found by integration, this integration being more or less difficult according to the shape of the loaded area. For a

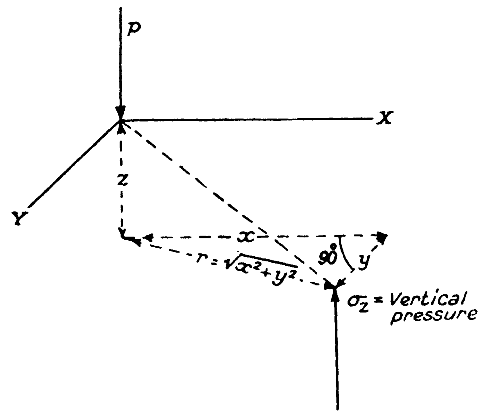


Fig. 10.—Vertical pressure at a point in an elastic medium caused by a point load at its surface.

point load P (Fig. 10), Boussinesq showed the vertical stresses σ_z at a point in the medium at a depth z below the point of application and having horizontal co-ordinates of x and y to be equal to

$$\sigma_z = \frac{3P}{2\pi} \times \frac{1}{z^2\left[\left(1 + \frac{r}{2}\right)^2\right]^{\frac{5}{2}}}$$

where $r^2 = x^2 + y^2$. In the case of spherical surfaces which touch the surface of the medium at the point of application of the load P

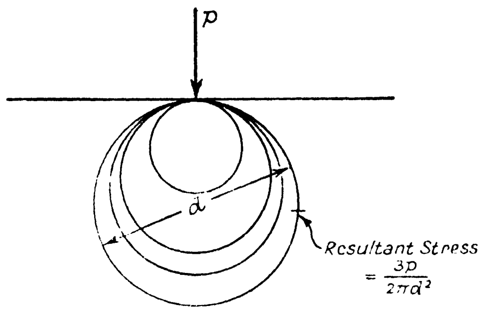


Fig. 11.—Resultant stresses on spheres below a point load acting at the surface of an elastic medium.

(Fig. 11), the resultant stress on a horizontal face is constant and equals $\frac{3P}{2\pi d^2}$ where d is the diameter of the sphere.

The vertical stress distribution under a concentrated load P is calculated from the above formula and is shown plotted in Fig. 12. The distribution is bell-shaped, and becomes flatter as the stressed plane lies further from the point of application of the load.

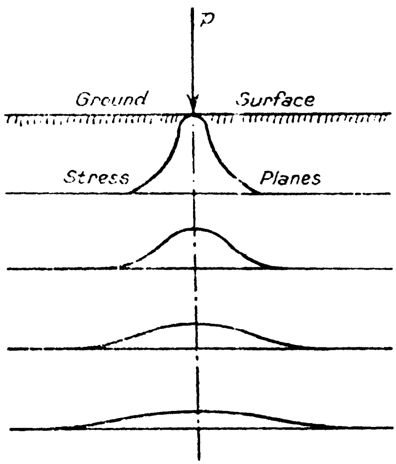


Fig. 12.—Showing bell-shaped distribution of stress below a point load at the surface of an elastic medium.

The Bulb of Pressure.

If contours of equal vertical stress are plotted from the vertical stress-distribution diagram, a series of isobars is obtained, known as the " Bulb of Pressure." For a strip load, this is of the form shown in Fig. 13, and for a circular load the form is as in Fig. 14. In both these figures the stress is shown in terms of the applied pressure. The stress trajectories for triangular loading are shown in Fig. 15, and are of interest when applied to embankments (see Chapter 10).

Stresses in Soils.

Different workers have deduced different values for the relationship between the unit bearing capacity of a soil and its unit cohesion, such values depending on the assumptions made.

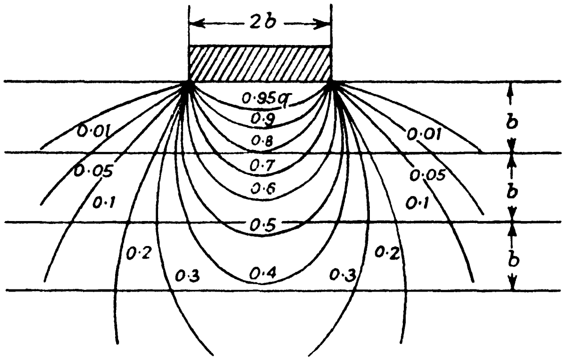


Fig. 13.—Showing bulb of pressure in an elastic medium loaded with a strip load.

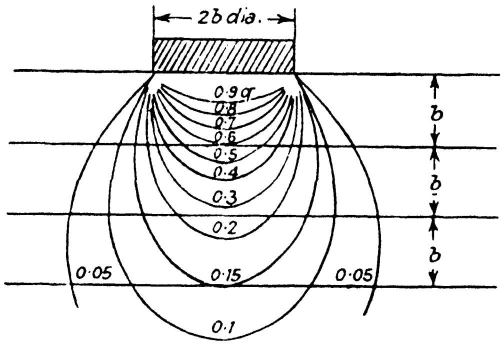


Fig. 14.—Showing bulb of pressure in an elastic medium below a circular load

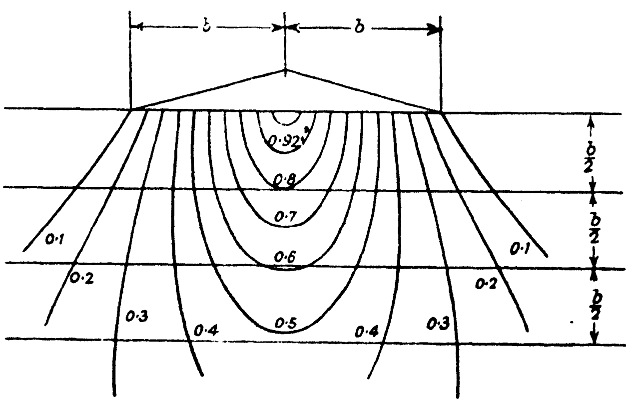


Fig. 15.—Showing distribution of pressure in an elastic medium below a triangular load.

Thus Terzaghi, using the approximate method, and including in his deduction the unit weight of the soil, gives

Unit bearing capacity of soil $= q_c = 4c$, where c is unit cohesion of soil as before.

Prandtl, using the Theory of Plasticity and ignoring the unit weight of the soil, gives

$$q_c = 5.14c$$

Krey, using the same assumptions as Terzaghi, gives

$$q_c = 5.52c$$

while Boussinesq, using the same assumptions as Prandtl, gives

$$q_c = 3.14c.$$

All these values apply only to purely cohesive soils, and they all express the unit bearing capacity as a function of unit cohesion; this latter property can be determined as described in Chapters 2 and 5 of this book.

For a non-cohesive soil, Terzaghi gives the following equation as applying to a surface-loaded strip of infinite length:

$$q_c = bw \tan^4 \left(45° + \frac{\phi}{2}\right)$$

where w = weight of soil per unit volume
b = $\frac{1}{2}$ (breadth of loaded strip)

whereas Prandtl, taking both c and ϕ into account, gives the equation

$$q_c = \frac{c}{\tan \phi}\left[\tan^2 \left(45° + \frac{\phi}{2}\right).e^{\pi \tan \phi} - 1\right]$$

The value for c for purely cohesive soils can best be obtained by means of the unconfined compression test described later. If a stress-strain curve be drawn for this test, a curve as shown in Fig. 16 results;

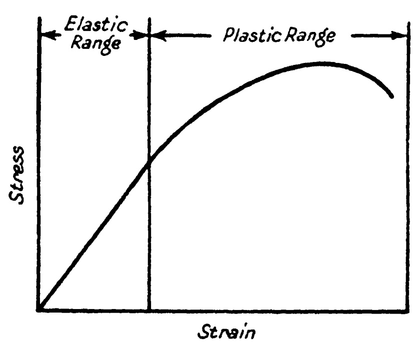


Fig. 16.—Stress-strain curve for normal clay.

for certain brittle clays, a curve similar to that shown in Fig. 17 is

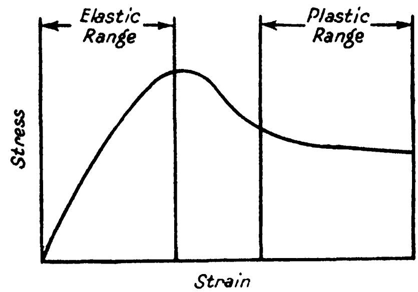


Fig. 17.—Stress-curve for brittle clay.

obtained. In such tests, the well-known relationship $c = \frac{\sigma_x}{2}$ obtains, where σ_x is the stress at failure. If elastic theories such as that of Boussinesq are used, the value of the cohesion c should be calculated from the maximum stress attained below the elastic limit of the soil; but for plastic theories such as those of Prandtl, the value of c should be found from the maximum stress in the plastic range, and this value may be either greater or less than the stress in the elastic range.

In clean sandy soils, cohesion may be produced by the rise of water due to capillary action; thus, if a man walks over a dry sandy beach, his feet sink into the sand, but when he reaches a damp area of sand, which is firm and hard, he can walk on it without sinking in. This difference of behaviour is due to cohesion set up in the sand by the capillary action of water.

Methods of the kind outlined above assume the soil to be an elastic, homogeneous, isotropic, semi-infinite solid, but it is known that it is seldom that all of these assumptions apply in practice. Stresses calculated on these assumptions are, however, as accurate an estimate of the stresses as can be obtained at the present time. Boussinesq's methods in particular were based on elastic theories which, although exceedingly complex, give results which can be easily understood and applied by engineers. Further important results obtained by him are as follows :—

Stress caused by a Surface Point Load.

In Fig. 18,

$$r^2 = x^2 + y^2$$

$$\text{and } R^2 = x^2 + y^2 + z^2$$

The vertical stress at any point A distant R from O, the point of application of the load P at the surface, is given by

$$\sigma_z = \frac{3P}{2\pi} \cdot \frac{z^3}{R^5}$$

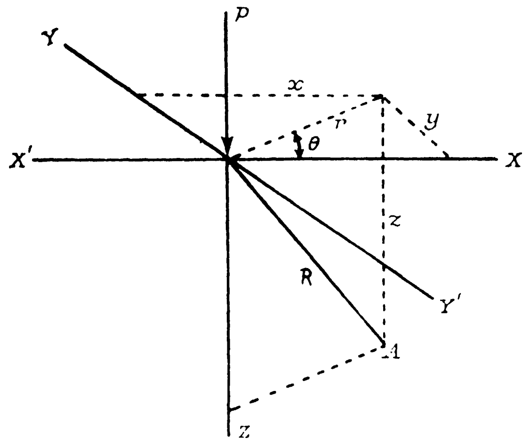


Fig. 18.—Co-ordinates of point A below point of application of point load P at surface.

and the lines along which maximum principal stress exists are radial lines. The maximum principal stress at any point A in Fig. 19 is given by $\dfrac{3P}{2\pi} \cdot \dfrac{\cos^2 \psi}{r^2 + z^2}$

and $r^2 + z^2 = d^2 \cos^2 \psi$

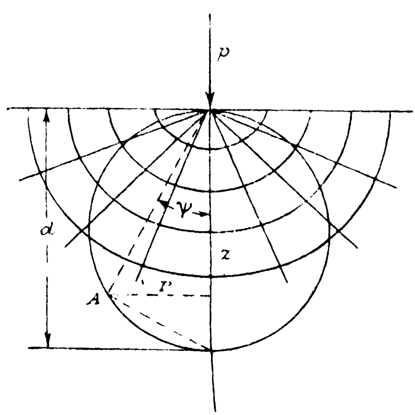


Fig. 19.—Showing surfaces of maximum principal stress in an elastic medium below point load P at surface.

Thus surfaces of equal maximum principal stress are spheres tangential to the surface at the origin.

Stress caused by a Loaded Circular Area on the Surface.

This is represented in Fig. 20. The vertical stress due to a loaded circular area on the surface at a depth z below the centre of the area is given by

$$\sigma_z = p\left[1 - \left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{\frac{3}{2}}\right] = p(1 - \cos^3\alpha)$$

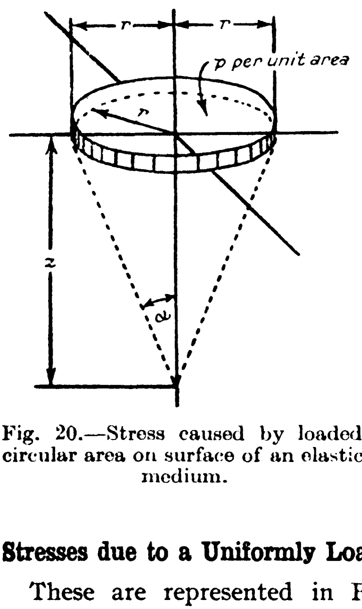


Fig. 20.—Stress caused by loaded circular area on surface of an elastic medium.

where p = load per unit area on the surface, and r = radius of loaded circle, and $\tan\alpha = \frac{r}{z}$.

If $\phi = 0°$, maximum shearing stress $= 0.33p$ and $p = 3c$. If ϕ is present, and if friction $q = \frac{1}{2}p$ acts away from the centre, then $p = 3.6c$. It is clear that the presence of internal friction increases the bearing capacity of the soil.

Stresses due to a Uniformly Loaded Strip.

These are represented in Fig. 21. If σ_1 and σ_2 represent the maximum and minimum principal stresses respectively acting at a point below the surface, then

$$\sigma_1 = \frac{p}{\pi}(\alpha + \sin\alpha), \text{ and } \sigma_2 = \frac{p}{\pi}(\alpha - \sin\alpha)$$

where α = the angle in radians between the lines joining the point and edges of the loaded strip.

The maximum shearing stress $s = \pm\frac{1}{2}(\sigma_1 - \sigma_2) = \pm\frac{p}{\pi}\sin\alpha$.

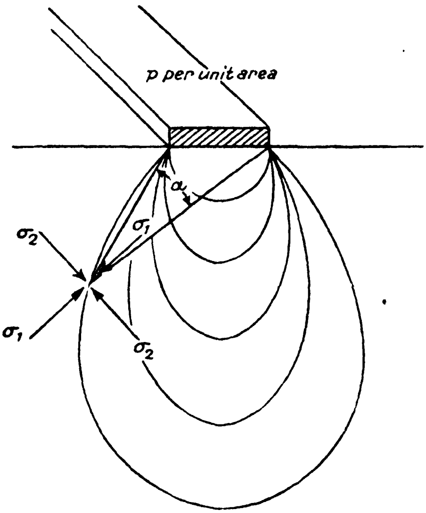


Fig. 21.—Showing distribution of stresses in an elastic medium due to strip load at surface.

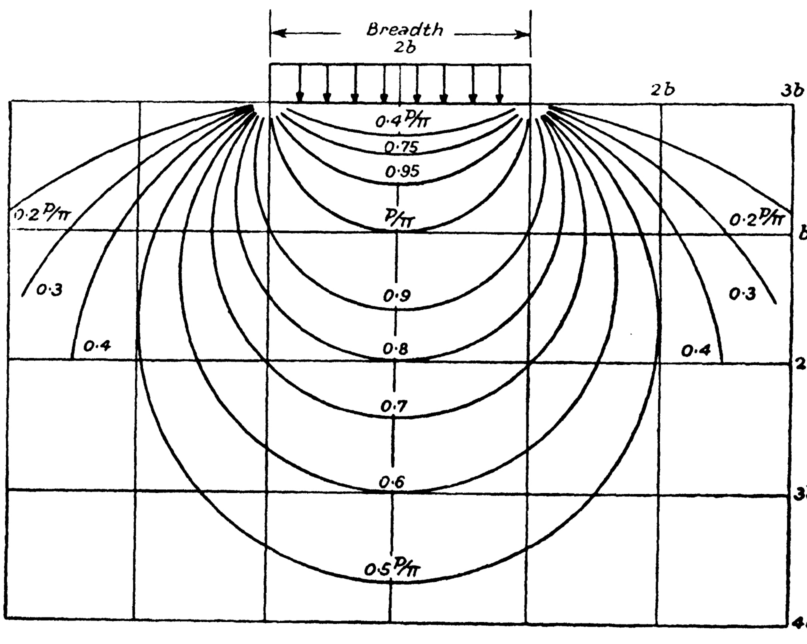


Fig. 22.—Showing lines of maximum shear stress in an elastic medium due to a strip load at the surface.

The points at which maximum shearing stresses occur form the locus of circles which can be constructed by setting up lines forming an angle $= 90° - \alpha$ (see Fig. 23) from each end of the loaded strip; the intersections of the lines so drawn are the centres of these circles. See also Fig. 47, page 61.

The corresponding values of the shearing stress occurring around each circle are shown in Fig. 23, whence it will be seen that the worst shearing stress occurs around a semi-circle whose diameter is the width of the loaded strip, its value being equal to $\frac{p}{\pi}$.

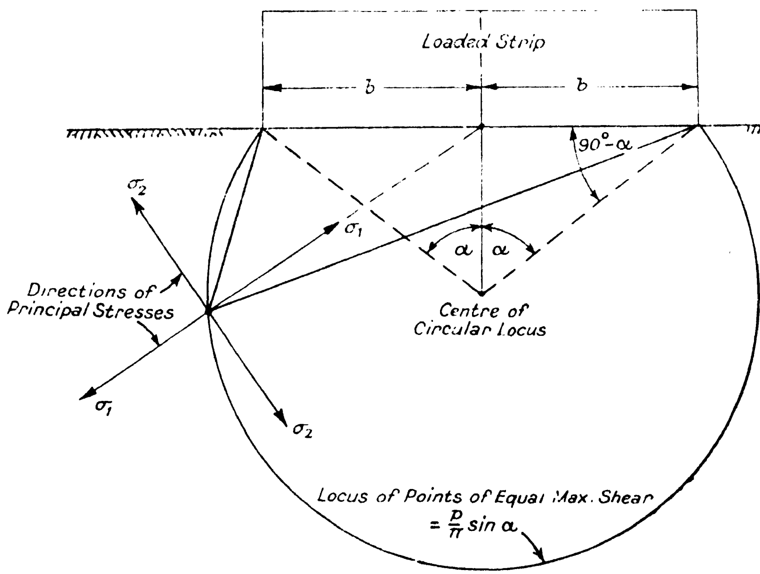


Fig. 23.—Showing construction for finding centre of circular locus of points of equal maximum shear in an elastic medium below a uniformly loaded strip at the surface.

Bearing Capacity of a Loaded Strip on a purely Cohesive Soil.

The following is due to A. H. D. Markwick.[1] Consider the element A (Fig. 24a), subject to a vertical pressure p_1. If deformation does not take place, the surrounding soil exerts a horizontal pressure p_2 on the element. The least value of p_2 necessary to prevent deformation is governed by the fact that in no direction must the shearing stress exceed the cohesive strength c of the soil. Resolving on any plane ML at an angle θ to the vertical (Fig. 24b), the shear stress can be shown to be $(p_1 - p_2) \sin\theta \cos\theta$, the maximum value of which, $\frac{(p_1 - p_2)}{2}$ occurs when $\theta = 45°$. Rupture will not occur on this plane if

$$\frac{(p_1 - p_2)}{2} \leqq c, \text{ or } p_1 \leqq p_2 + 2c \qquad (1)$$

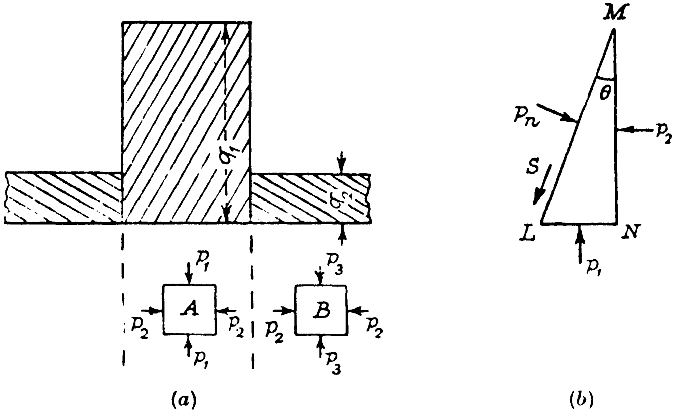


Fig. 24.—(a) Stability of a loaded strip of soil. (b) Stability of element of soil. (Markwick.)

Similarly, for element B (Fig. 24a)

$$p_2 \leqq p_3 + 2c, \text{ and } p_1 \leqq p_3 + 4c \qquad (2)$$

If p_1 is equal to the bearing pressure q_1 on the foundation, plus the pressure wh due to the weight of soil above the element, and p_3 the pressure due to the weight of the soil wh and the superimposed load q_2

$$q_1 + wh \leqq wh + q_2 + 4c, \text{ or } q_1 \leqq 4c + q_2 \qquad (3)$$

This result agrees with the value deduced by C. A. Hogentogler and K. Terzaghi by another method[2]; the value found by Prandtl[3] and H. Hencky[4] by more rigorous analysis is $5.14c$, as given previously on page 19.

General Case of the Bearing Capacity of a Soil under a Circular Loaded Area.

This problem can be considered as the general three-dimensional case of a uniformly loaded circular area carried by a soil in which both frictional and cohesive resistances are present.

Consider first the element of soil in Fig. 24b, subject to a stress p_1 acting normally to LN. A stress p_2 acting normal to MN is just sufficient to prevent deformation and can be calculated on the hypothesis that the shear stress on any plane must not exceed the quantity

$c + p_n \tan \phi$, where p_n is the normal stress on the section and c and ϕ have the same meanings as before. A. L. Bell[5] solved this problem, and it can be shown by resolving parallel and at right angles to LM, that failure will occur on the plane where $\theta = \left(\frac{\pi}{4} - \frac{\phi}{2}\right)$ and that the relation between p_1 and p_2 on this plane is given by

$$\frac{p_1}{p_2} = \frac{1 - \sin \phi}{1 + \sin \phi} - \frac{2c \cos \phi}{1 + \sin \phi} \quad \ldots\ldots(4)$$

The reasoning previously followed cannot easily be applied to the three-dimensional case, since the value of the horizontal stress must diminish with increase in the distance from the centre of the load.

The method by C. A. Hogentogler and K. Terzaghi previously given has therefore been extended to the three-dimensional case. The stability of the load is assumed to depend upon the equilibrium of a ring-shaped section (Fig. 25a) in which the angles of the wedges ODA

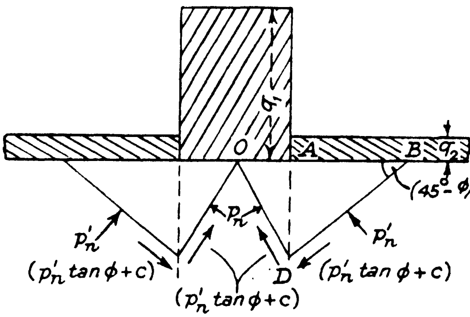


Fig. 25(a).—Showing stability of loaded circular area, and section of wedge rupture.

and ABD are both made equal to $45° - \frac{\phi}{2}$, since the greatest risk of failure occurs at this angle.

The stability of an element of the wedge of rupture (Fig. 25b) can now be considered. Consider first the prism $OADD'A'$ immediately under the load in the case of limiting equilibrium. The forces acting are the vertical stress q_1, the normal and shear stresses on plane ODD', the circumferential stress p_t, and the horizontal stress H'. Resolving vertically and horizontally :

$$q_1 = \frac{p_n}{(1 - \sin \phi)} + \frac{c \cos \phi}{(1 - \sin \phi)} - \frac{2rw \cos \phi}{3(1 - \sin \phi)} \quad \ldots\ldots(5)$$

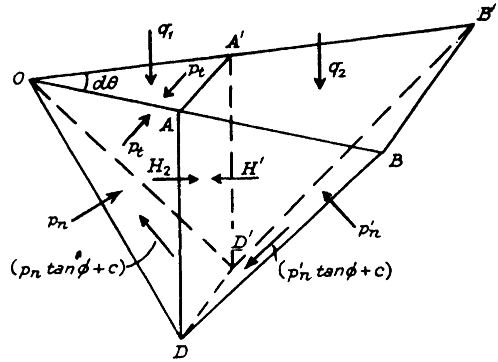


Fig. 25(*b*).—Showing stresses on element of wedge of rupture. (Markwick.)

where w = weight of soil per cubic foot
and r = radius of area of contact

$$\text{and } H' = \frac{p_n}{2(1+\sin\phi)} - \frac{c}{2}\cdot\frac{\cos\phi}{(1+\sin\phi)} + \frac{p_t}{2} \quad \text{.........(6)}$$

Eliminating p_n from (5) and (6)

$$H' = \frac{q(1-\sin\phi)}{2(1+\sin\phi)} - \frac{c\cos\phi}{(1+\sin\phi)} + \frac{rw\cos\phi}{3(1+\sin\phi)} + \frac{p_t}{2} \quad \text{.........(7)}$$

Similarly the limiting equilibrium of sector $ABDD'B'A'$ may be considered. The forces are similar to those acting on the element $OADD'A'$, but no circumferential forces are assumed, since the radial cracking of the ring of soil when it is displaced makes it impossible to rely upon this source of resistance to deformation. By resolving horizontally and vertically as before and then eliminating p'_n, it can be shown that

$$H_2 = \frac{q_2(3-\sin\phi)(1+\sin\phi)}{2(1-\sin\phi)^2} + \frac{c(3-\sin\phi)\cos\phi}{(1-\sin\phi)^2} + \frac{rw(1+\sin\phi)^2(2-\sin\phi)}{3(1-\sin\phi)^2\cos\phi} \quad \text{.........(8)}$$

If q_1 represents the ultimate bearing capacity, $H' = H_2$, whence from (7) and (8)

$$\frac{q_1(1-\sin\phi)}{2(1+\sin\phi)} = \frac{q_2(3-\sin\phi)(1+\sin\phi)}{2(1-\sin\phi)^2} + \frac{4c}{(1-\sin\phi)\cos\phi} + \frac{rw(1+6\sin\phi-3\sin^2\phi)}{3(1-\sin\phi)^2\cos\phi} - \frac{p_t}{2} \quad \text{.........(9)}$$

Consideration of the numerical coefficients of the first and last terms of (9) shows that the value attached to p_t has an important effect upon the bearing capacity. If p_1 in equation (4) represents the vertical stress at a depth x from the surface, and if p_2 in the same equation represents the horizontal stress p_{tx} at a depth x, equation (4) then becomes

$$p_{tx} = (q_1 + wx)\left(\frac{(1 - \sin\phi)}{(1 + \sin\phi)} - \frac{2c\cos\phi}{(1 + \sin\phi)}\right) \qquad (10)$$

where $p_1 = q_1 + wx$.

Now p_t is the average value of p_{tx} taken over OAD, and on substituting in equation (4) it can be shown that

$$q_1 = \frac{q_2(3 - \sin\phi)(1 + \sin\phi)^2}{2(1 - \sin\phi)^3} + \frac{c\cos\phi}{(1 - \sin\phi)}\left[1 + \frac{4}{(1 - \sin\phi)^2}\right] + \frac{rw\cos\phi}{3(1 - \sin\phi)}\left[\frac{(2 - \sin\phi)(1 + \sin\phi)^2}{(1 - \sin\phi)^3} - \frac{3}{2}\right] \qquad (11)$$

Equation (11) above is of the general form $q_1 = Aq + Bc + Caw$ where A, B and C are numerical coefficients depending on ϕ. If the values of these coefficients are plotted against values of ϕ, it is found that they increase rapidly with increase in ϕ, while the resistance due to cohesion is also seen to be greatly increased when internal friction is present.

TABLE 1. Calculated Ultimate Bearing Capacities of Typical Soils under Loaded Circular Area (based on a soil density of 100 lb. per cub. foot).

Type of Soil	Typical mechanical properties of soil: Cohesion c, Lb. per sq. ft.	Cohesion c, Lb. per sq. in.	ϕ in deg.	Calculated bearing capacity; lb. sq. in.: Bare ground ($q_2 = 0$), $r = 6''$	Bare ground, $r = 18''$	With surfacing ($q_2 =$ 100 lb. sq. ft.), $r = 6''$	With surfacing, $r = 18''$	Increase in bearing capacity due to surfacing per cent., $r = 6''$	Increase, $r = 18''$
Clay, almost liquid	100	0.69	0	3.5	3.6	4.6	4.7	31	30
Clay, very soft	200	1.39	2	7.7	7.9	8.9	9.1	16	15
Clay, soft	400	2.78	4	17	17.1	18	19	9	9
Clay, fairly stiff	1000	6.94	6	46	47	48	49	4	4
Clay, stiff	1500	10.42	8	77	78	79	80	3	3
Clay, very stiff	2000	13.89	12	127	128	130	131	2	2
Silts, wet	0	0	10	0.4	1.2	2.8	3.6	600	200
Sand, dry	0	0	34	8.6	26	33	50	320	93
Sand predominating with some clay	400	2.78	30	87	97	102	113	33	16
Sand-gravel mixtures, cemented	1000	6.94	34	290	308	314	332	8	8

Moisture in the Pores of a Soil.

The presence of moisture in the voids of a soil has a profound influence upon the elastic properties of the soil, since pressure caused by loads is taken up both by the water in the voids and by the granular soil skeleton. The former type of pressure is known as the " neutral pressure " and the latter as the " effective pressure " or " intergranular pressure," and the total pressure intensity across any plane within the soil is the sum of these two pressures.

Three types of neutral (i.e., pore) pressure are recognised, i.e. :--

1. That due to hydrostatic pressure, i.e., a static condition.
2. That due to percolating water, i.e., an upward pressure.
3. That due to " hydrostatic excess," i.e., the effect of load.

In addition to these three types of pressure, pressure is caused by capillary moisture.

Neutral Pressure due to Hydrostatic Pressure. This is denoted by u_z, and is dependent on the density w_w of the water, and the depth h of the given point below the water level, so that $u_z = w_w h$.

Neutral Pressure due to Percolating Water. This is denoted by p, and is proportional to the value of the hydraulic gradient at the given point.

Consider a small prism cut out of a stream-line with a cross-section a and length ΔL, as shown in Fig. 26. If unit pressure p acts on the face a, then to produce the pressure p, a hydraulic head given by the expression $\Delta h = \dfrac{p}{w_w}$ is required. Thus the hydraulic gradient at the face a is

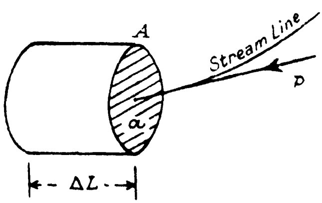


Fig. 26.—Showing imaginary prism of water cut out of a stream-line.

$$i = \frac{\Delta h}{\Delta L} = \frac{1}{w_w} \cdot \frac{p}{\Delta L}$$

If the neutral pressure due to percolating water acts in an upward direction, it has the effect of reducing the effective weight of the soil, and at a certain critical value of i, the whole weight of the soil will be balanced by the pressure of the water. This critical value for the hydraulic gradient may be obtained from the relationship

$$w_w i = (1 - n)(w - w_w)$$

where w_w is the unit weight of water,
w is the unit weight of soil,
n is the porosity of the soil, i.e., $\frac{\text{volume of voids}}{\text{total volume}}$

Hydraulic gradients exceeding 1.0 are undesirable.

Neutral Pressure due to Hydrostatic Excess. This type of pressure takes place during the consolidation of a soil and is caused by squeezing out of water from the moisture-filled voids in the soil. Terzaghi has deduced a formula for the value of this pressure, but it is of a complicated nature and will not be given here, since this excess pressure is unlikely to be of much significance in the case of highway and airport engineering problems. The structural engineer who may be called upon to erect heavy buildings on saturated clay soils should consult more detailed mathematical treatises on soil mechanics for further information concerning neutral pressure due to hydrostatic excess.

Pressure due to Capillary Moisture. The action of capillary moisture in a soil is similar to that of a uniformly distributed load placed on its surface, the value of such load being equal to the head resulting from the effective length of capillary movement h_c. This pressure h_c acts within the mass in all directions, and is additional to the principal stresses, which accordingly become $\sigma_1 + h_c$ and $\sigma_2 + h_c$. The maximum shearing stress is, as has been verified experimentally by Terzaghi, unaffected by the presence of capillary moisture, since

$$\begin{aligned} s_{max} &= \tfrac{1}{2}\left[(\sigma_1 + h_c) - (\sigma_2 + h_c)\right] \\ &= \tfrac{1}{2}\left[\sigma_1 - \sigma_2\right] \end{aligned}$$

Lateral Pressure in an Unloaded Semi-Infinite Mass.

If a cube of side 1 unit, situated at a depth z below the horizontal boundary of an elastic continuum, is compressed by a vertical pressure $\sigma_z = wz$, where w = unit weight of soil, then compressive stresses σ_x and σ_y act upon the lateral sides of the cube. If Poisson's Ratio is denoted by μ, and if no lateral displacement takes place, then since

$$\sigma_x = \sigma_y,\ \frac{\sigma_x}{\sigma_z} = \frac{\mu}{1-\mu},\ \text{and if } \frac{1}{\mu} = m,\ \frac{\sigma_x}{\sigma_z} = \frac{1}{m-1} = K$$

The ratio $\frac{\sigma_x}{\sigma_z}$ is called the hydrostatic pressure ratio and is denoted by K.

The following relation connects K with ϕ, the angle of internal friction :—

$$\frac{\sigma_x}{\sigma_z} = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2\left(45° - \frac{\phi}{2}\right) = K$$

Terzaghi has deduced values of K for dense sands ranging from 0.40 to 0.45, corresponding values for loose sands being 0.45 to 0.50, and for clays 0.70 to 0.75.

The coefficient K can be found in the laboratory by enclosing a soil sample in a rubber jacket and placing it in a container with water. The sample is then subjected to vertical loading, and this causes an increase in pressure in the adjacent liquid. The ratio between the induced water pressure and the applied load gives the hydrostatic pressure ratio. (See also Chapter 5 for an outline of the principle of the tri-axial shear or compression test).

Consolidation of Soils.

Loose unconsolidated soil has no elasticity, but as consolidation proceeds the mass acquires a certain amount of elasticity, though stresses are not usually proportional to strains, and Hooke's Law does not then apply. Soils settle under compression, part of the settlement being permanent, and part reversible, this latter being termed the " elastic rebound."

Consider the behaviour of a loaded plate placed on hitherto uncompressed sand (Figs. 27 and 28). If the sand is subjected to cycles of loading and unloading, using the same maximum load for each cycle, the irreversible part of the settlement gradually decreases after each cycle of loading and unloading. The curves obtained by plotting the position of the loaded plate against the load give " hysteresis loops " (Fig. 28) which become longer and narrower as the cycles proceed, until finally the sand mass behaves as an elastic body and the loading/unloading curve becomes a closed loop, although Hooke's Law may still not apply.

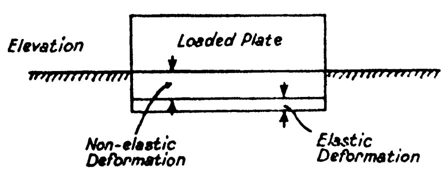


Fig. 27.—Loaded plate on uncompressed sand.

If the maximum load on the plate is now increased, irreversible deformations appear once more, which explains why a gravel road which has stood up to light traffic for a long time without permanent deformation will nevertheless be destroyed if only a single very heavy lorry be driven along it.

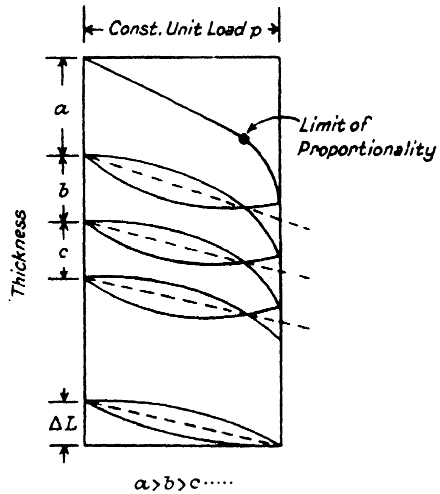


Fig. 28.—Showing elastic and non-elastic deformation of sand mass. (Successive loadings and unloadings of previously uncompressed sand).

Behaviour of Soil Particles during Compaction of a Soil. The consolidation or settlement of a soil mass involves primarily a reduction in the porosity, or the ratio of voids to overall volume, with concomitant rearrangement or " mutual accommodation " of the particles. In addition, the particles themselves may be deformed, especially when they are of laminar shape, as is the case in clay soils.

Mutual Accommodation of Sand Particles. The behaviour of particles of sand depends to some extent on the way in which the load is applied. If the horizontal surface is loaded with a rigid circular plate, the grains of sand are mostly driven downwards towards their ultimate positions in the consolidated mass, while with a non-rigid plate, a small number of grains become confined at the centre of the disc and support it, and the edges of the disc settle down. In both cases, sand tends to escape from the edges of the loaded plate. If, however, the soil mass is an elastic continuum, the settlement is greater at the centre of a non-rigid disc than at the edges, and a concavity in the plate is developed.

Mutual Accommodation of Clay Particles. The behaviour of a mass of clay under load is similar to that of a wet sponge. Compression causes a marked vertical movement in a wet clay, the water being squeezed out of the pores by the downward action of the load. This process may be a prolonged one and may last for years, as is evidenced by the well-known movement of buildings on clay foundations, in contrast to the consolidation of a sandy soil, which takes place in a relatively short time.

According to Terzaghi, when a saturated mass of clay is placed under load, the entire stress is taken up at first by the moisture in the soil pores, the moisture gradually transmitting it to the solid particles, so that the pore moisture is under a continually decreasing excess hydrostatic pressure.

Modulus of Elasticity of a Soil. As mentioned above, a consolidated soil possesses elasticity, but does not obey Hooke's Law. The modulus of elasticity of a soil is not a constant such as Young's Modulus for a steel, but is thought to vary with the depth of the soil. Actual values will vary from one soil to another, and approximate figures can be obtained experimentally from compression tests. These tests may be carried out on either confined or unconfined samples.

Modulus of Elasticity of a Soil from Unconfined Compression Tests. E_1. A sample of cohesive soil of length L is subjected to a compression load as shown in Fig. 29. If the stress-strain curve is plotted, it will

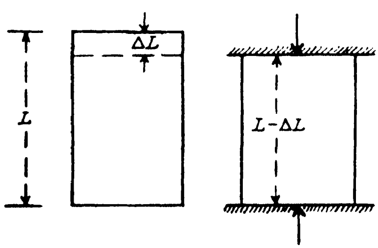


Fig. 29.—Compression of a clay.

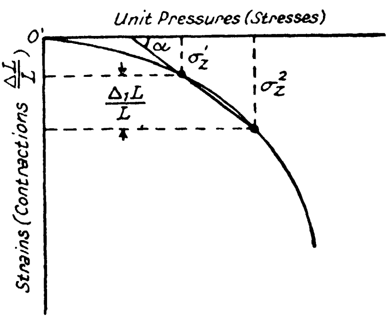


Fig. 30.—Stress-strain curve for a clay.

take the form shown in Fig. 30. The portion of the curve between σ_z^1 and σ_z^2 may be assumed to be a straight line, so that for the corresponding contraction $\Delta_1 L$ we have

$$E_1 = p \div \frac{\Delta_1 L}{L} = \frac{pL}{\Delta_1 L}$$

Modulus of Elasticity of a Soil from Confined Compression Tests. E_2. If a sample of soil is confined in a rigid metal cylinder of height L units, and unit load is applied by means of a piston, with cycles of loading and unloading until a closed hysteresis-loop (Fig. 28) is obtained, then the soil is in an elastic state. If the elastic rebound is measured ($\Delta_2 L$), then the modulus of elasticity is given by $E_2 = \frac{\sigma_z L}{\Delta_2 L}$. The value of E_2 can also be obtained from consolidation tests.

Modulus of Elasticity of a Soil from Consolidation Test. E_2. The voids ratio, i.e., the total pore space divided by the total volume of soil particles, is determined under increasing unit loads and a graph plotted. Assuming that the soil is already compressed and elastic at the beginning of the test, with a voids ratio of e, and that this voids ratio is reduced to e_1 and e_2 by additional pressures of p_1 and p_2, then the final average voids ratio $e_f = \frac{e_1 + e_2}{2}$. By Hooke's Law,

$$\frac{p_1 + p_2}{2} = E_2 . \frac{e - e_f}{1 + e}, \text{ or } E_2 = \frac{1 + e}{a}$$

where a $= \frac{2(e - e_f)}{p_1 + p_2}$ and is therefore a variable depending upon the particular type of soil.

Settlement of a Soil under Constant Load.

Fig. 31 represents a layer of soil saturated with moisture and confined between a pervious overburden above and an impervious

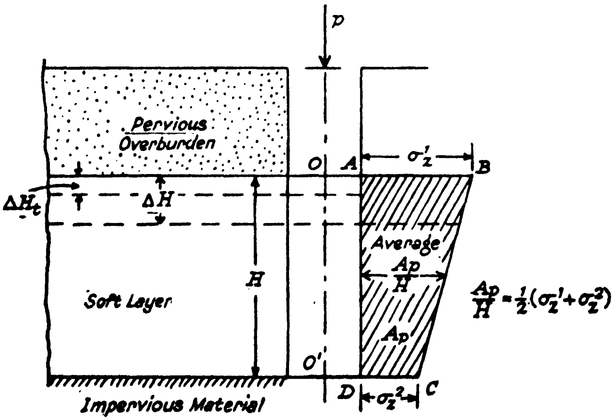


Fig. 31.—Settlement of a soft soil under load.

layer below. Let H be the original thickness of the soft layer and ΔH the total settlement which is to be calculated. The vertical pressure at the top and bottom of the soft layer can be calculated from Boussinesq's formula (page 16). Let these pressures be $\sigma_z{}'$ and $\sigma_z{}''$. In Fig. 31, AB is proportional to $\sigma_z{}'$, and DC to $\sigma_z{}''$. The average vertical pressure in the soft layer is $\frac{1}{2}(\sigma_z{}' + \sigma_z{}'') = \frac{Ap}{H}$.

Fig. 32 represents a foot-cube of the material confined on its vertical faces and base, under a load of 1 ton, in which the stress-strain relationship $\frac{\sigma_z}{\Delta H}$ is constant, where ΔH is the settlement. Since $E_2 = \frac{\sigma_z}{\Delta H}$, therefore $\Delta H = \frac{1}{E_2}$ ft., and the volume of water squeezed out of the soil is $\frac{1}{E_2}$ cub. ft. (="specific loss of moisture.")

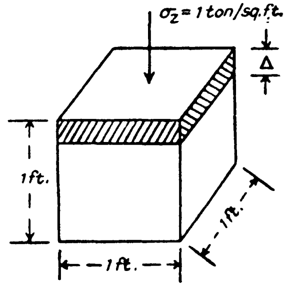


Fig. 32.—Cube of soft soil under load.

Thus for a vertical prism of soil of an area of 1 foot square and height H, for a superimposed surface pressure P, we have

$$\text{total settlement} = \Delta H = \left(\frac{\sigma_z{}' + \sigma_z{}''}{2}\right)\frac{1}{E_2}H = \left(\frac{\sigma_z{}' + \sigma_z{}''}{2E_2}\right)H$$

Thus determination of total settlement resolves itself into a determination of E_2, which is generally done by means of consolidation tests (see above). The variable a is determined by taking the slope MN of the stress-strain graph for a small increment in the pressure (Fig. 33), and the total settlement is given by the formula

$$\Delta H = \frac{(\sigma_z{}' + \sigma_z{}'')Ha}{2(1 + e)}$$

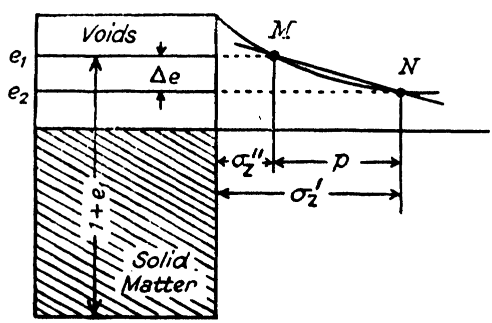


Fig. 33.—Showing consolidation of a soft soil under load.

Rate of Settlement under Constant Load.

In Fig. 31, water will flow upwards into the pervious layer when the consolidation load p is acting. The water squeezed out of a vertical prism of 1 square foot cross-section in a very small time dt is given by $Q = ki.dt$, where k is the co-efficient of permeability, and i is the hydraulic gradient. If k is constant and if Q increases during time dt, then i must also increase during that time.

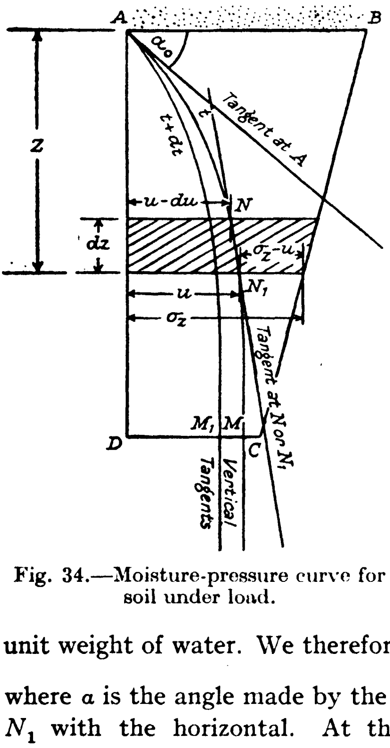


Fig. 34.—Moisture-pressure curve for soil under load.

If the values of the neutral pore pressure u are known at different vertical depths in the soil and are plotted on the horizontal axis as in Fig. 34, a curve AM is obtained which is known as the "moisture-pressure" curve, at time t. As time elapses, all neutral pressure values decrease, and a moisture-pressure curve at time $t + dt$ would be represented by AM_1.

Let us consider an elementary layer of thickness dz at a depth z. In passing through this layer, the drop in neutral pressure will be du, which corresponds to a loss of hydraulic head of $\frac{d_u}{w_w}$, where w_w is the unit weight of water. We therefore have $i = \frac{1}{w_w} \cdot \frac{du}{dz}$, and $\frac{du}{dz} = \cot \alpha$ where α is the angle made by the tangent to the curve at points N and N_1 with the horizontal. At the surface of the layer, where the hydraulic gradient drives the water into the pervious layer, we have $i_0 = \frac{1}{w_w} \cot \alpha_0$, and the discharge during time dt is given by

$$Q_0 = \frac{k \cot \alpha_0 . dt}{\gamma_w}.$$

The decrease in volume of the vertical prism of 1 square foot area during the time dt will be the volume of the water discharged during that time. Hence $d(\Delta H)_t = Q_0$, or

$$\frac{d\,(\sigma_z' + \sigma_z'')tH\alpha}{2(1+e)} = \frac{K \cot \alpha_0\, dt}{w_w}, \text{ i.e., } dt = \frac{d\,(\sigma_z' + \sigma_z'')t\, H\, \alpha\, w_w}{2k(1+e)\cot \alpha_0}$$

Relation between Voids Ratio and Pressure in Soils.

Experimental values can be found for the ratio $\frac{\text{total volume of voids}}{\text{total volume of solid}}$ at varying superimposed loads. If this ratio is plotted against load during both loading and unloading, curves of the type shown in Fig. 35 are obtained. The upper curve shows the behaviour of an undisturbed sample of soil, the lower one the behaviour of a remoulded soil. It will be noticed that the latter curve is a more regular one than the former. According to Terzaghi, remoulded soils give logarithmic curves for the voids ratio during both compression and expansion, i.e., during loading and unloading.

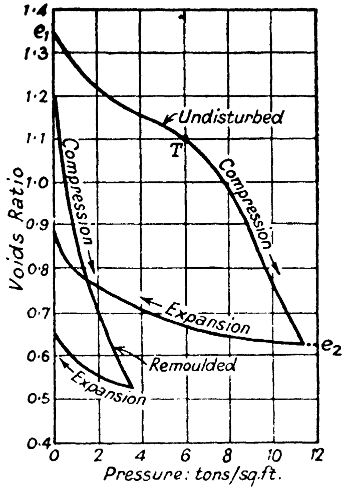


Fig. 35.—Showing relation between voids-ratio and pressure in soils.

Pre-Consolidation Unit Load. From a knowledge of the voids-ratio compression curves of a soil, it is possible to determine what the superimposed load has been at some previous time, and this may be used to throw light on the amount of erosion which has taken place in the course of geological time. This method has in fact been used in the elucidation of facts concerning the covering and uncovering of the London Clay, in recent work by A. W. Skempton.

Curves I and II in Fig. 36 are idealised compression and expansion curves which can be assumed to represent the behaviour of a given soil under the accumulation and subsequent removal of overburden. It cannot be assumed that the depth from which a sample is taken is a true measure of the total superincumbent load which the sample has had to carry in the past, since geological erosion will almost certainly have removed some of the maximum overburden. The dotted portion of curve I represents a further decrease in the voids-ratio which would have occurred had the thickness of overburden above the sample been greater than was actually the case.

When the sample is extracted and in effect the overburden above it removed, expansion of the soil occurs by relief of pressure, so that the voids-ratio increases to a value e_3, which is its initial value in the laboratory. When the sample is now reloaded with a unit load not exceeding a value p_c, the pre-consolidation load, the compression curve is practically a straight line as shown in curve III in Fig. 36. The addition of a further load causes the voids-ratio to decrease at a greater rate, and the curve becomes steeper as in curve IV, and tends to touch curve I. During unloading, expansion again occurs, as shown by curve V.

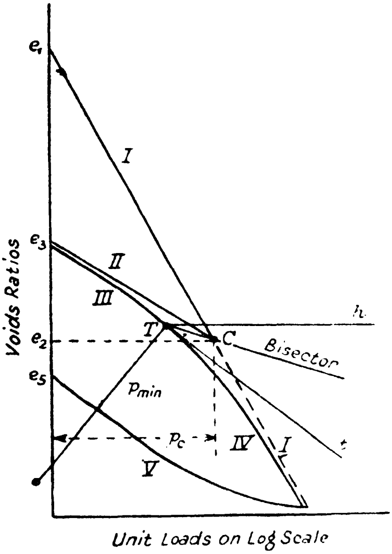


Fig. 36.—Showing effect of overburden on consolidation of clay.
(A. W. Skempton).

It is evident that the pre-consolidation load p_c can be determined from curves III, IV and V, which are experimental.

The position of what is known as the "virgin branch," i.e., the dotted portion of curve I, is determined by the results of various cycles of loading and unloading. Point T is the position on curve III-IV for which the radius of curvature is a minimum. The line Th is drawn parallel to the x-axis, while the line Tt is drawn tangential to the curve III-IV. The bisector of the angle hTt cuts curve I at C. The value of the unit load corresponding to point C gives the pre-consolidation load p_c. The voids-ratio c_2 for this pre-consolidation load is the value which should be used in the determination of E_s (see page 34), or of settlement (see page 35).

The foregoing treatment of the consolidation of soils should be read in conjunction with its treatment in the following chapter of this book.

Friction and Cohesion in Soils.

The assumption that $\dfrac{\sigma_2}{\sigma_1} = \dfrac{1 - \sin\phi}{1 + \sin\phi}$ pre-supposes that ϕ, the angle of internal friction for a given soil, is constant in a soil mass, but this is not usually the case, since it is known that ϕ varies from point to point in the mass. The value of ϕ is in fact affected not only by the type of soil, but also by the normal pressure, the soil density, the grain-size of the soil, the shape of the soil grains, i.e., by the uniformity or otherwise of the soil.

For medium and coarse sands in a dry state, ϕ varies from about 30° to 35°, the value being less if the sand is submerged. For damp sands, the value is much the same as for dry sands, but the angle of repose may be different. Clays have a much lower value for ϕ than sands, ranging from a few degrees up to 15°, or in exceptional cases 20°.

Cohesion in soils may be what is known as " true cohesion " due to the mutual attraction of particles through the action of molecular forces, or " apparent cohesion," which is caused by the presence of moisture films, there being a surface tension between the soil particles and the included moisture in the soil

Cohesion in sands is rare, and when it occurs is small enough to be neglected. The cohesion which is considered, values for which are given in Table 2 below, is that due to the presence of moisture. In practice, cohesion may be dependent on normal stress, since this controls the moisture content, which in turn affects cohesion, but the effect of normal stress is neglected in the average values quoted in the Table.

TABLE 2.

Type of Soil.	*Cohesion in lbs. per sq. ft.*
Almost liquid clay	100
Very soft clay	200
Soft clay	400
Medium clay	1000
Muddy sand	400
Very dense sand and gravel	1000

CHAPTER TWO

THE MECHANICAL AND PHYSICAL PROPERTIES OF SOILS.

Size of Soil Particles.

Soils consist of discrete mineral particles of varying sizes which enclose a network of voids or pores. These voids are filled with air or water or both, while the mineral particles consist of the products of the physical and chemical weathering of rocks (see Chapter 3 for fuller details on this matter) and may vary in size from pebbles down to grains of sub-microscopic dimensions. The coarser particles, i.e., those exceeding 0.002 m.m. in diameter, consist mainly of rock debris (pieces of granite, limestone, sandstone, quartzite, basalt, etc.), together with fragments of the more resistant rock-forming minerals such as quartz, mica, felspar, augite, hornblende, iron-ores, etc. The finer particles constitute what is termed by soil physicists the " clay fraction," and consist principally of the products of the chemical weathering of rocks ; the particles which make up the clay fraction differ from those of the coarser fractions in their physico-chemical properties as well as in their size, and they exhibit characteristics associated with those of micro-crystalline and colloidal substances.

The size ranges adopted for purposes of classification by the International Society of Soil Science are :—

Coarse sand	2.0 to 0.20 m.m. in diameter
Fine sand	0.20 to 0.02 m.m. ,, ,,
Silt	0.02 to 0.002 m.m. ,, ,,
Clay	less than 0.002 m.m. ,, ,,

The classification suggested by the Massachusetts Institute of Technology recognises certain intermediate sizes not included in the above and is thought to be preferable to it, since the ranges given correspond roughly with important differences in the engineering properties of soils. It is as follows :—

Gravel		above	2.00	m.m.	in diameter	
Sand	Coarse		2.00	,,	to 0.6	m.m. diameter
	Medium		0.6	,,	,, 0.2	,, ,,
	Fine		0.2	,,	,, 0.06	,, ,,
Silt	Coarse		0.06	,,	,, 0.02	,, ,,
	Medium		0.02	,,	,, 0.006	,, ,,
	Fine		0.006	,,	,, 0.002	,, ,,
Clay			Below		0.002	,, ,,

It should, however, be noted that the above classification, although simple and practical, is by no means accepted universally at present. A. W. Skempton[6] has suggested definitions of various clays as follows :

Muds. Almost fluid soils which have been consolidated under not more than a few feet of overlying sediments.

Soft Clays and Firm Clays. Plastic soils which can be moulded in the fingers, and which have been consolidated under overburdens of from a few feet to several hundred feet.

Stiff Clays. Soils which can be moulded with the fingers only with difficulty, and which have been consolidated under many hundreds or many thousands of feet of overburden. They often contain fissures with an apparently random distribution, and if a large sample of such a clay is dropped from a height, it breaks into polyhedral fragments along the fissure planes.

Shales. Materials with more or less parallel laminations which are brittle. If ground with water, they can be reduced to a plastic mass. Such soils have been consolidated under many thousands of feet of overburden.

Compressibility of Soils.

All soils can be compressed under load. Sands have relatively little compressibility, while silts and clays have a large compressibility, their complete consolidation being achieved only after a long period of time which may be virtually infinite (see also pages 32-33, 43).

Plasticity of Soils.

The plasticity of a soil is the ease or otherwise with which it can be moulded ; sandy soils are non-plastic, while clay soils are extremely plastic when wet.

Cohesion of Soils.

The cohesion of a soil is the property whereby its constituent particles adhere one to another. Sandy soils, either dry or wet, possess little or no cohesion ; silts possess moderate cohesion, especially when damp, while clays, unless saturated with water, possess a high degree of cohesion.

Cohesion is a function of the moisture content of the soil and of the surface area and adsorptive properties of the particles. The cohesive force c between two spheres caused by a ring of water around the points of contact is given by the expression

$$c = \frac{2\pi r T}{1 + \tan\frac{\theta}{2}}$$

where T = surface tension,
r = radius of each sphere,
θ = angle subtended by the radii passing through the points of contact of the spheres.

Actually, soil particles are not spherical in shape (see Chapter 3), or indeed any other regular shape, so that the above ideal formula does not apply in practice. It is, however, known that at the saturation point the cohesion of a soil becomes zero.

Cohesion caused by moisture films in coarse-grained soils is relatively low, the degree of cohesion increasing with decrease in particle size. As the grain-size of the soil decreases, factors other than moisture are found to come into play ; thus organic material acts as a cementing agent to an extent at present unknown, while the physical forces associated with the colloidal state also operate.

Physical Properties of Clay Soils.

A soil is usually described as a clay when it contains a clay fraction which amounts to more than 30 per cent. of the total weight of the soil. Such a clay fraction confers colloidal properties on the soil in which it occurs, and exerts a predominating influence on the properties of the soil as a whole. Clays are markedly retentive of water, and changes in their water content cause corresponding changes in volume of the soil, the actual amount of increase of volume with increase in water content depending on the mineralogical nature of the clay minerals present in the clay fraction (see also Chapter 3). The cohesive properties of the clay minerals confer cohesion on the clay soil in which they occur.

If a soil contains only a small proportion of clay particles, the coarser fraction of the soil exerts by far the greater influence on the properties of the soil. The structure of a soil such as a sand-clay mix consists of a skeletal framework of sand-silt particles with clay particles partly filling the voids between the larger grains. The clay fraction may be present as a coating to the larger grains, or as independent aggregates.

Shrinkage of Clays. When a wet plastic clay dries slowly, shrinkage occurs in two stages, the first of which is termed " normal shrinkage " and the second " residual shrinkage." During normal shrinkage, the decrease in volume of the clay is equal to the volume of water lost by evaporation, and at the end of this stage the soil becomes lighter in colour. The water content at the end of normal shrinkage and the amount of residual shrinkage (this latter being less than the normal shrinkage) both depend on the type of clay minerals which make up the clay fraction. Thus kaolin clays (see page 73) possess very little residual shrinkage, but montmorillonite clays (see page 74) have a very marked residual shrinkage owing to the moisture which is held between the lattice layers of the crystal structure.

Permeability of Clays. Although the voids-ratio of a clay soil is greater than that of a sandy soil, the permeability is less and the size of the individual pores is very much less. The relative impermeability of clays is thus due to properties conferred on the soil by the clay particles, and not to any inherent low values for the voids-ratio. Most clays have in fact very high voids-ratios.

Compressibility of Clays. The degree of compressibility of a soil depends on its air and water content. Air is more readily squeezed out of a soil by loads than water, so that sandy and silty soils, which normally have both air and water in their voids, are fairly easily compressed. Clay, however, rarely contains air in the voids, so that consolidation under pressure can only be effected in clays by the squeezing out of water. Compression of clay is accordingly a prolonged process owing to the relatively low permeability of such a soil.

Curves can be constructed which show the relation between the decrease in thickness of a soil sample and the time which has elapsed since the application of a load, such curves being based upon experimental results, their axes showing time of consolidation and the percentage of total compression respectively (see also pages 45-6). The shape of such curves depends naturally upon the permeability of the soil, and from them co-efficients of permeability can be calculated.

It is known that for any given soil, the time required to produce a given degree of consolidation is proportional to the square of the thickness of the layer under consolidation. Thus if a sample of clay 0.5 inch thick takes an hour to reach, say, 90 per cent. consolidation under a given load, then a layer of similar clay 40 feet thick would require about 100 years to reach the same degree of consolidation under the same load. It is thus possible to derive time-compression curves for any thickness of material from experimental values obtained from samples of known thickness by multiplying the time-factor by H^2, where H is the thickness of the material.

Terzaghi has carried out experiments on the consolidation of mixtures of sand and mica, which is a flaky mineral, as a result of which it was concluded that the large differences in the compressibility of different kinds of soils could be attributed to the mechanical effect of larger or smaller amounts of scale-like or flaky particles. Microscopic examination of clay minerals reveals the fact that such minerals possess scale-like shapes, so that the large compressibility of clay soils appears to bear out Terzaghi's experimental work. The same author[7] holds that if the state of stress in a clay is constant or is only changing extremely slowly, the effective pressure on the clay is transmitted from one grain to another through the solid part of the adsorbed film of moisture on the grains. So long as this film remains intact, the mass of clay is in a solid state and has a high modulus of elasticity and a low compressibility. If when the clay is in this state the pressure is increased beyond a certain limiting value, the bond between the adsorbed layers breaks down and the particles become separated from one another by a film of adsorbed moisture. In this state the clay is lubricated, so that further application of load increases consolidation until enough water has been squeezed out to allow of the adsorbed solid layers coming into contact once more, after which the clay is again in a solid, elastic state. This process may take decades or even centuries for its final completion.

Terzaghi maintains further that in a consolidated clay the stresses are borne partly by highly viscous films of adsorbed water surrounding the individual clay particles and partly by the solid portions of the adsorbed layers. A constant film bond stress is associated with viscous flow which occurs at a constant rate, but the grain bond stress does not produce movement.

The Oedometer or Consolidometer Test.

In this test, a sample of clay is held in a brass ring 3 inches in diameter and $\frac{3}{4}$ inch in depth. Porous Portland limestone discs, each about $\frac{1}{2}$ inch thick, are placed above and below the sample, and pressure is applied through the upper stone disc by means of a lever system in which a force of 10 lbs. exerted on the lever-arm causes a pressure of 1 ton per square foot on the clay sample. The porous stones are inundated with water during the test in order to prevent drying out of the sample (see Fig. 37).

The consolidation process can be observed by noting the decrease in the thickness of the sample under constant load, using a dial gauge sensitive to 1×10^{-4} inch. It will be found that at first the rate of compression is rapid, but that it decreases with time, the curve for the compression-time relationship being similar to that shown in Fig. 38*a*. It will also be found that after a time, further compression under the original load is negligible, and the total movement recorded by the dial gauge up to this stage may be taken as the final compression $s\omega$ under the original load. If s_t is the compression at any time t, then the degree of consolidation μ at time t is given by $\mu = \frac{s_t}{s_w}$.

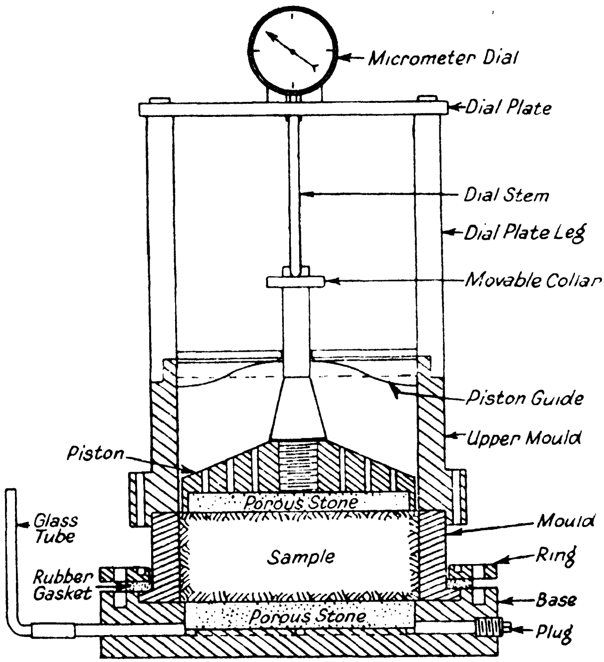


Fig. 37.—Diagram of Consolidometer or Oedometer.

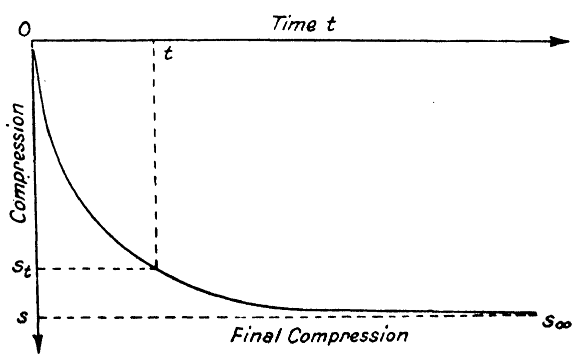


Fig. 38a.—Showing relation between time and compression of soil under constant load.

If the degree of consolidation is plotted against time, a curve similar to that of Fig. 38*b* is obtained, and similar curves can be obtained for different values of the load on the sample.

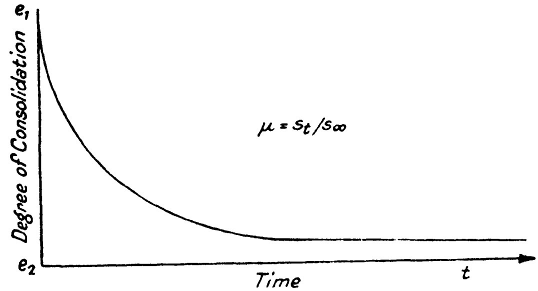


Fig. 38*b*.—Showing relation between time and degree of consolidation of soil.

If the load is increased after the clay has reached equilibrium under the initial load, then a further decrease in the volume of the sample occurs, and the voids-ratio of the clay is progressively reduced. Thus a pressure-voids-ratio curve can be obtained as given in Fig. 39, and this curve is characteristic for any particular soil.

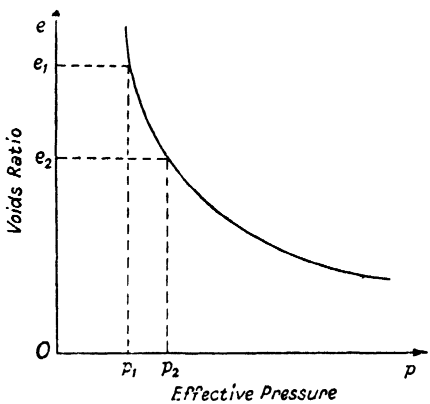


Fig. 39.—Showing relation between pressure on and voids ratio of soil.

The consolidometer test gives a means of determining the compressibility of a soil, which for linear consolidation is defined as the decrease in thickness per unit thickness per unit increase in pressure on the soil.

Determination of Compressibility of a Soil from Consolidometer Tests.

Suppose that a sample of clay has a thickness H_1, with a voids-ratio e_1 under a pressure of p_1. If the pressure is increased by σ to p_2, the thickness will be reduced to H_2 and the voids-ratio to e_2. Then

$$\frac{\text{Original volume of voids}}{\text{Original volume of sample}} = \frac{e_1}{1 + e_1}$$

and therefore

$$\frac{\text{Decrease in volume of voids}}{\text{Original volume of sample}} = \frac{e_1 - e_2}{1 + e_1}$$

$$= \frac{\text{Decrease in volume of sample}}{\text{Original volume of sample}}$$

since the decrease in the volume of the sample is the same as the decrease in the volume of voids.

Since the area of the sample remains constant under the test, the thickness is proportional to the volume, and therefore

$$\frac{\text{Decrease in thickness}}{\text{Original thickness}} = \frac{e_1 - e_2}{1 + e_1}$$

$$\text{or } \frac{H_1 - H_2}{H_1} = \frac{e_1 - e_2}{1 + e_1}$$

and therefore, compressibility $= \dfrac{H_1 - H_2}{\sigma H_1} = \dfrac{e_1 - e_2}{\sigma(1 + e_1)}$

If the average slope of the pressure-voids-ratio curve over the pressure increment $\sigma = a$, $a = \dfrac{e_1 - e_2}{p_1 - p_2}$

$$a = \frac{e_1 - e_2}{\sigma}$$

or in the limit, $a = -\dfrac{de}{dp}$

and the compressibility $= \dfrac{a}{1 + e_1}$

The term a is not a constant, but varies with the voids-ratio, and therefore with the pressure; it can, however, be calculated for any value of e or p from the p/e curve.

The final compression of a clay layer can be calculated from this curve for any increment of pressure, the equation being

$$H_1 - H_2 = \frac{e_1 - e_2}{1 + e_1} H_1$$

Terzaghi's Theory of Consolidation leads to the relationship

$$\text{Degree of consolidation } \mu = \psi\left(\frac{Ct}{d^2}\right)$$

where C is the " consolidation factor," a constant for a given clay,
t is the time after application of the load,
d is the " drainage path," i.e., the greatest distance which the extruded moisture has to travel through the clay, measured in a straight line, before reaching a free drainage surface. In the consolidation test d is one-half the thickness of the sample, since the water can drain from both surfaces, i.e., at the junction of the sample with the porous stones. The foregoing equation can be written in the form $\mu = \psi(\tau)$, where τ is the " time-factor."

The function ψ depends on the distribution of pressure in the layer, and upon whether one or both faces of the layer constitute free drainage surfaces. For the conditions obtaining in the consolidation test, i.e., free drainage from both faces of the sample and uniform distribution of pressure, the solution is

$$\mu = 1 - \frac{8}{\pi^2}\left(e^{\frac{-\pi^2}{4}\tau} + \frac{1}{9}e^{\frac{-9\pi^2}{4}\tau} + \frac{1}{25}e^{\frac{-25\pi^2}{4}\tau} + \ldots\right)$$

where e = voids-ratio as before.

It follows from the expression $\mu = \psi\left(\frac{Ct}{d^2}\right)$ that the degree of consolidation is inversely proportional to the square of the drainage path d. A check on the validity of the theory is obtained by comparing the shape of the experimental time/consolidation curve for any particular clay with that obtained from the formula $\mu = 1 - \frac{8}{\pi^2}\left(e^{\frac{-\pi^2}{4}\tau} + \ldots\right)$ given above. Experimental work by Skempton on remoulded samples of London Clay has led to the following conclusion :

1. The rate of consolidation agrees within reasonable limits with the rate predicted by theory.
2. The rate of consolidation is inversely proportional to the square of the thickness of the sample.
3. The coefficient of consolidation is a constant for London Clay.
4. The pressure/voids-ratio ratio is independent of the thickness of the sample.

Indicator Tests on Soils.

So-called indicator tests are widely used as a rough guide to the engineering properties of soils and were first described by Atterberg in 1911. Details of these tests are given in Chapter 5 of this book, to which reference should be made for the meaning of terms which are not defined in the present chapter.

Liquid Limits vary from zero in the case of purely sandy soils to 100 or more for highly colloidal clays, particularly those with considerable organic content. Typical values quoted by Skempton are :—

Sandy clays and silts	35 to 40
Normal clays	40 to 80.

High liquid limits are commonly, though not always, associated with high natural water contents, a factor which is obviously bound up with climatic and local conditions as well as with the nature of the soil. The term *liquidity index* is sometimes used in this connection, the term meaning $\frac{\text{water content} - \text{plastic limit}}{\text{plasticity index}}$. For soft clays, the liquidity index is between 1.0 and 0.5, while in the case of stiff clays such as the London, Kimmeridge or Gault Clays, it is approximately zero. Firm to stiff clays lie between zero and 0.5, while shales tend to give a negative value.

Compression Index.

If a graph be plotted of voids-ratio against log pressure, it is found to be substantially a straight line for which the equation is :—

$$e = e_1 - k \log \frac{p}{p_1}$$

where e is the voids-ratio at pressure p,
e_1 is the voids-ratio at pressure p_1,
k is the slope of the graph.

The constant k is termed the compression-index of the soil, and this compression curve can evidently be defined by the constants of compressibility e_1 and k.

Clays with high voids-ratios have higher liquid limits and steeper compression curves than those with low voids-ratios, while significant correlations between the constants of compressibility and the amount of clay fraction can also be found ; thus it is sometimes possible to estimate the values of e_1 and k from the mechanical analysis of a soil (see Chapter 5). Exceptions to these correlations occur mainly among the non-sedimentary clays such as boulder-clay, drewite and kaolin clay. The latter type is composed very largely of the mineral kaolinite (see Chapter 3); drewites are lime-muds consisting of small needle-shaped crystals of the mineral aragonite, while boulder-clays, which are the product of former glacial action, are composed rather of finely comminuted material resulting from the grinding action of ice on rock masses than of clay fractions produced by the normal processes of chemical weathering of such masses.

Sedimentation-Compression Curves.

The term " sedimentation-compression curve " was introduced by Terzaghi in 1941 to distinguish the behaviour of a clay layer under an accumulating thickness of overlying sediments from that of a similar clay under laboratory conditions of increasing load. Pressures are calculated from the thickness of the overburden, and allowance has to be made for hydrostatic uplift occurring in that part of the overburden which is below ground-water level.

With full uplift, the pressure p at a depth h in a stratum with an average voids-ratio e is $p = (\rho - 1)(1 + e)h$, where ρ is the density of the solid soil particles, since the density of a material with its voids fully saturated is $\frac{\rho + e}{1 + e}$, and hence the density reduced by hydrostatic uplift is $\left(\frac{\rho + e}{1 + e}\right) - 1 = \frac{\rho - 1}{1 + e}$.

The pressure under which a sample has been consolidated is therefore $p = \frac{\rho + e_1}{1 + e_1} . h_1 + \frac{\rho - 1}{1 + e_2} . h_2$, i.e., the sum of the pressures of the overburden above and below water-level, where e_1 and e_2 are the average voids ratios of the strata above and below ground-water level, h_1 is the depth of ground-water below the surface, and h_2 is the depth of the sample below ground-water level.

Clays which have been consolidated under the weight of overburden found to be existing at the present time are known as " normally consolidated " clays, while those which at present exist under only a few feet of overburden, but which at some time in the past have been deeply buried, are known as " over-consolidated " clays. Superficial clays which have been deposited in shallow basins and have never been buried under anything but the smallest cover might be termed " under-consolidated " clays.

In Fig. 40, voids-ratio is plotted against log pressure for a sample of London Clay taken at a depth of 50 feet below the present surface. This clay was originally deposited as mud (represented by the point *a* on the curve, and then consolidated under 50 feet of overburden to point *b* on the curve). Under further accumulating overburden, the voids-ratio was reduced until the full thickness of Upper Eocene Beds

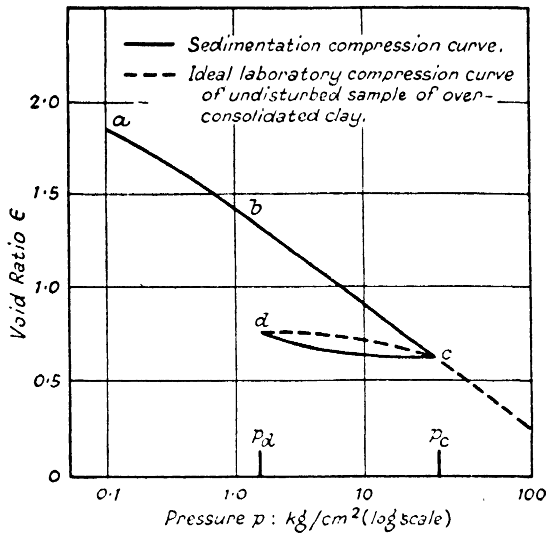


Fig. 40.—Showing physical properties of normally consolidated and over-consolidated clay. (Skempton.)

above the clay produced a pressure (the " pre-consolidation load ") corresponding to the point *c* on the curve. Uplift and consequent erosion of the overlying beds then reduced the thickness of the overburden, and under the decreasing load the voids-ratio increased, as shown by the full line *c*—*d* in the figure. It will be seen from this figure that the difference between a normally consolidated and an over-consolidated clay is represented by the difference in the voids-ratios at the points *b* and *d*.

Comparison between Laboratory and Sedimentation Compression Curves.

Experimental work carried out by A. Casagrande on various clays, using pressures ranging from 0.5 to 10 kg./sq. c.m., has shown that the laboratory compression curves obtained from that range of pressures fall substantially below the sedimentation compression curves for soft and for firm clays. Terzaghi has suggested that an explanation of this discrepancy is that in the laboratory, loading is many thousand times more rapid than in nature, and that this rapid loading leads to partial breakdown of the natural voids structure in the clay. As a result of this work it is now customary to carry out laboratory tests on undisturbed samples, with the object of ensuring that the clay under test retains as far as possible the same voids structure as that which it has in the clay strata under investigation.

The Shear Strength of Soils.

The shear strength of a soil is the resistance offered by it to lateral movement, and this resistance depends upon the combined effects of cohesion and internal friction, as explained in Chapter One. As already mentioned there, internal friction is the more important factor in the case of sands and gravels, while cohesion plays the dominant part in the shear strength of silts and clays. Laboratory studies of the shearing resistance of soils have been carried out by L. F. Cooling and D. B. Smith[8]. The modified Coulomb relation for shear resistance is

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

where s = resistance to lateral displacement along an interface
c = resistance due to cohesion,
p_n = normal component of the pressure on the interface,
ϕ = angle of internal friction.

Compression tests were made on unrestrained samples $\frac{3}{4}$ inch in diameter and $1\frac{3}{4}$ inches long. The ends of the specimens were made cone-shaped to prevent barrelling of the sample during the test, and the duration of the test was from 3 to 5 minutes. The type of failure under load was found to vary with the type of soil and to some extent with the moisture content.

In most cases a well-defined shear plane passes right across the specimen, clear of the end pieces, and in some cases shear planes occur inclined equally to and on opposite sides of the axis of compression. Sometimes Lüder's lines appear in the early stages of the test and subsequently multiply rapidly as the test proceeds. In the case of plastic clays the deformation prior to failure is large, the compressive stress falling away slowly after the maximum has been reached. Sandy clays behave differently, the strain to failure being small and the rupture being of a brittle nature, the stress dropping rapidly after the maximum has been reached. The shear plane is inclined at 45° or less to the axis of compression. These tests were carried out on both disturbed and undisturbed samples. The relation between compressive strength and shear strength varies for different types of soil; thus it has been found that with plastic clays, the compressive strength is twice the shear strength, but with sandy clays, the compressive strength is more than twice the shear strength, the factor tending to increase as the soil becomes drier. It has been suggested that this difference in properties can be correlated with the value of the angle of internal friction.

The following equations have been put forward as likely to represent the relationship between cohesion, angle of internal friction and the compressive and shear strengths of a soil :—

$$\sigma = 2c \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

$$s = c \cos \phi$$

where σ is the compressive strength as found from an unconfined compression test,
s is the shear strength,
c is unit cohesion,
ϕ is the angle of internal friction.

From the above, it follows that

$$\sigma = 2s \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \sec \phi,$$

$$\text{or} \quad \frac{\text{compressive strength}}{\text{shear strength}} = \frac{2}{1 - \sin \phi}.$$

This is the relation between maximum compressive and maximum shear strength. Thus, if σ and s are known, c and ϕ can be found.

Difficulties in carrying out Shear Tests on Soils.

There is always some difficulty in obtaining really reproducible results when soils are subjected to compression and shear stresses, and by varying test conditions almost any values may be obtained for cohesion and internal friction.

The shear strength of a soil depends upon the pressure carried by the soil skeleton (the effective pressure) and not on that carried by the pore water (the neutral pressure).

Clay is a plastic material, so that the speed of the test influences the results obtained. To get comparable results, tests should always be carried out at the same rate. If the rate of test is too high, viscous effects in the pore water will give too high a value for the shear strength, and ideally the test should be carried out at such a rate that the water pressure in the pores is always zero. In practice this results in testing times which are inordinately long, since the volume changes caused by shearing stresses set up stresses in the pore water which can only be relieved by consolidation, which, as has already been shown earlier in this chapter, is a slow process in relatively impervious clay soils. The viscous effects are largely eliminated if the test takes several minutes or longer, but to allow of complete consolidation, several hours may be required. A compromise must usually be made whereby the time of test is limited to 25 or 30 minutes.

Clay soils offer a resistance to shear which increases with the increase in shearing strain until a maximum value is reached which is the shear strength of the soil. After a soil has failed in shear, the shear resistance may continue constant, or it may drop, depending on the type of soil. In the case of plastic clays and loose sands, the shearing resistance remains constant after failure ; in harder, more brittle clays and dense sands, a marked drop often occurs. It should be noted that in practice the value of the shear strength after failure (known as the residual shear strength) is often important, so that such value should be measured.

The following conditions have been suggested as being necessary if the same soil is to give the same shear strength in two different tests :—

(a) The moisture content, and hence the voids-ratio should be the same for all samples of the same material.

(b) The same initial condition must be reached by using the same loading procedure. This could be done by arranging :—

1. To increase the consolidation pressure from zero to the maximum obtaining in the natural conditions, and then to carry out shear tests.
2. To induce a state of over-consolidation by pre-consolidating the specimen to a certain maximum load, and then unloading to a smaller pressure, under which the sample is sheared.
3. To compact the material to a standard compaction by a standard method and then to follow either of the above methods.

It cannot be over-emphasised that shear tests must be run slowly, so that the total applied stresses are effective stresses which can be fully developed and sufficient time is allowed for volume-change adjustments to take place. If this is done, excess hydrostatic pressure due to pore moisture is kept to a very low value owing to the slow, continuous loading method or by means of very small increments of shear loading.

Cohesionless Soils. These soils are highly permeable ; water can escape freely under test conditions, so that externally applied stresses are therefore the effective stresses.

Remoulded Cohesive Soils. The permeability of these soils is very low, so that ordinary rates of loading result in much of the external stress being taken up by the neutral pressures of the pore moisture rather than by the soil skeleton. It is, however, thought that direct shear tests carried out on remoulded clay samples give some idea of the properties of the undisturbed material if the test conditions can be well controlled, and if a slow shear force be applied over a period of 24 hours.

The cohesion obtaining depends on the maximum pre-consolidation load and on the moisture at the particular stage of unloading. The figure obtained is therefore not strictly a soil constant, but is largely a hysteresis effect of loading and unloading varying widely with the method of test adopted and the consolidation characteristics of the soil. This is a controversial matter which is by no means fully understood at the present time.

Undisturbed Cohesive Soils in their Natural State of Consolidation. These soils, when subjected to a quick shear test under zero normal pressure, or to a simple unconfined compression test, give a shear strength which is somewhat lower in value than the shearing strength at pre-consolidation load. The results obtained in such circumstances have therefore an inherent factor of safety, so that the rapid shear test and the unconfined compression test can be used on such materials as routine tests.

Some Conclusions from The Unconfined Compression Test.

Since the angle of rupture α for an idealised fragmental material is given by the expression $45^\circ - \frac{\phi}{2}$, and ϕ is the angle of internal friction, a simple compression-to-failure test may be expected to give rise to cracks in the material making an angle of 2α with the vertical.

Types of Compression Failure: Method of Occurrence of Cracks. There are two principal types of failure in an unconfined compression test:

(a) A brittle material fails by sliding along one or two planes as shown in Fig. 41(*a*).

(b) A plastic material bulges and fails by sliding along many planes as shown in Fig. 41(*b*).

These types of failure correspond approximately with sands on the one hand and with clays on the other; in practice, however, not all sands are brittle, while all clays do not bulge under the test. Thus a clay which is dry enough to possess a relatively rigid soil skeleton behaves as a brittle material; if, however, it is remoulded, it behaves as a plastic material. In the tri-axial compression or shear test (see page 58) a dense sand tends to fail as a brittle material, while one with a high voids-ratio may bulge like a plastic one.

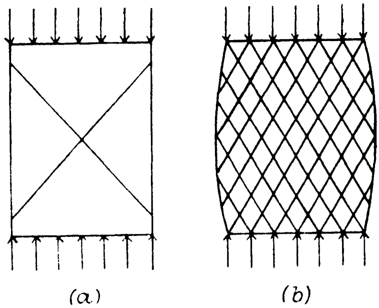


Fig. 41.—Showing mode of failure of (a) Brittle Clay, (b) Plastic Clay.

Direct Shearing Test.

Details of the apparatus used for this test are given on pages 138-9. The angle of friction of a soil can be determined from the results of such a test using the relationship $s = \sigma \tan \phi$, where s is the value of the pulling force at the instant preceding slip, σ is the normal force and ϕ is the angle of friction of the soil.

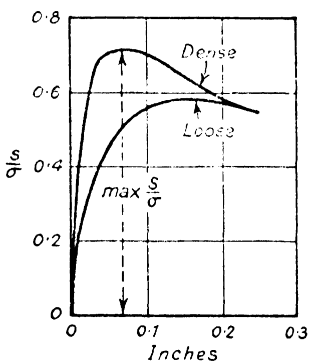


Fig. 42.—Stress-displacement curves for undisturbed and for re-moulded sands.

Fig. 42 shows typical curves for displacement plotted against stress for direct shearing tests on a sand in a loose and a compacted state. The dense sand expanded during the test, with a consequent increase in the voids-ratio, but the loose sand became consolidated. It will be noted that the curve for the dense sand shows a peak value for $\frac{s}{\sigma}$, while that for the loose sand does not; both curves, however, reach the same ultimate.

Practical difficulties which may be encountered during the carrying out of a direct shear test include trouble with sand getting between the contact surfaces of the shear boxes, frictional losses which affect the normal pressure, and the formation of a bulged shearing surface instead of a plane one.

Behaviour of Sand Under Shear. Observations by A. Casagrande show that when a dense sand is subjected to shear, failure occurs by a sudden expansion of the material. The internal strains developed just prior to failure are relieved by a sudden breakdown of the whole soil structure ; crushing of the sand grains is not found.

Behaviour of Clay Under Shear. In contradistinction to the behaviour of sands under shear, the failure of clays is progressive, different parts of the sample being under different strains at the same time, and the shear stress is not uniform throughout the mass.

Fig. 43 shows that the stress-displacement curves for undisturbed and remoulded clays respectively correspond to those for dense sand and loose sands respectively.

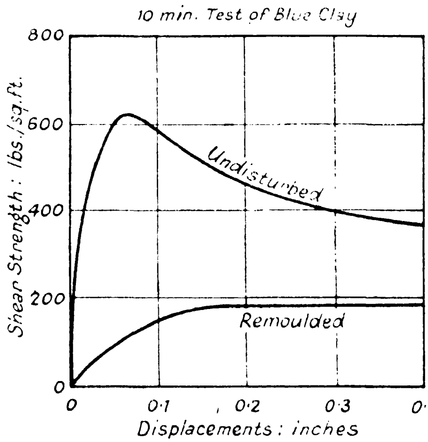


Fig. 43.—Stress-displacement curves for undisturbed and for re-moulded clays.

The Tri-Axial Shear or Compression Test.

The tri-axial compression test was designed to obviate the difficulty experienced in a direct shear test whereby in the latter test the shear is not distributed evenly throughout the sample. The apparatus and the theory used is illustrated in Figs. 44 and 45. A sample *B* enclosed in a rubber jacket (see Fig. 44 and also Fig. 81) between two porous stones is first subjected to a uniform pressure in all directions. This may be done by applying external pneumatic or hydrostatic pressure or by evacuating air from inside the sample.

Mohr's Circle for the state of stress induced in the sample is represented by point *A* in Fig. 45, *OA* being $\sigma_1 = \sigma_2$. The axial pressure σ_1 is gradually increased, as shown by the increasing diameters of the circles. At the instant of failure, Mohr's Circle touches Mohr's

Envelope, which in the case of cohesionless soil passes through the origin O and makes the angle of internal friction ϕ with the horizontal; hence ϕ is determined.

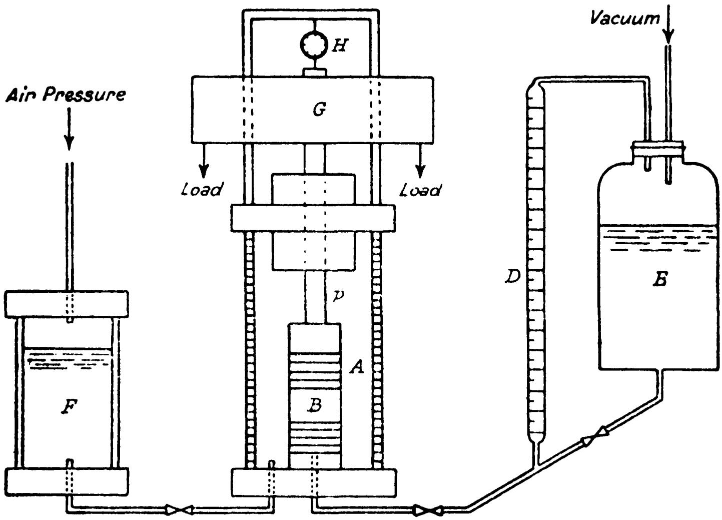


Fig. 44.—Diagram showing apparatus used for the Tri-axial shear or compression test for soils. See also Fig. 81.

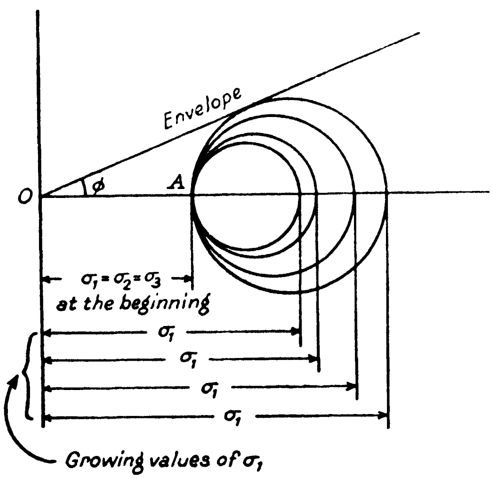


Fig. 45.—Mohr's envelope of stress for gradually increasing loads.

Fig. 44 shows the arrangement of a tri-axial compression machine. A compression chamber A contains the prepared sample B between two porous stones and within an enclosing vertical rubber jacket. A vacuum applied through the aspirator E and observed in the pipette D allows the removal of air and moisture from the sample. Glycerine is forced under pressure from the reservoir F into A and kept at constant pressure throughout the test. A load is applied through G, and deformation of the sample is measured by means of the dial gauge H. The sample is kept fully saturated with water throughout the test, so that changes in its volume under load and its density at failure can be determined by observations on the water level in the pipette D. The effect of the pressure of the glycerine in the chamber A is to eliminate almost entirely the friction in the piston rod P.

Plastic Flow at Failure.

When an elastic continuum is overloaded, its elasticity is destroyed, and plastic flow starts in the form of a very slow movement of the whole mass from underneath the load. It is assumed that in these circumstances the shearing stress in the zone of flow is constant and equal to the ultimate cohesive value of the material. This may be expressed as

$$s = s_c = c$$

where s is the applied shearing stress,
s_c is the constant shear stress in the zone of flow,
c is the unit cohesion of the material.

It has often been observed that plastic flow does not continue indefinitely, although on the assumption given above there would appear to be no reason why it should not do so. It is thought therefore that at a certain stage of plastic flow, either s decreases or s_c increases or that both changes operate simultaneously. A decrease in s would mean that deformation relieves stress, while an increase in s_c would mean that the physical properties of the soil in the zone of flow had changed.

In 1937, M. J. Hvorslev carried out experimental work on two Austrian clays, both in the remoulded state. One of these clays showed a continuous plastic flow before the sample was destroyed in the shearing box. The stress at which plastic flow began was about one-third of the shearing value of the soil, and the velocity of this plastic flow increased until failure of the sample occurred. This process was, however, slow, sometimes over 20 hours elapsing between the last load application and the actual failure of the soil.

Coulomb's well-known equation of failure is

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

Hvorslev has, however, developed another equation for the value of the shear stress of a soil which also takes both internal friction and cohesion into account and which is of the same general form as that of Coulomb. Using the loading-unloading technique described on page 31, Hvorslev determined the "equivalent pressure" p_e, i.e. the pressure which operated during the consolidation of the sample which must have produced the natural voids-ratio e. In Fig. 46, curve I–I represents the natural consolidation-pressure curve of the sample, curve II–II is the rebound branch of a simple over-consolidation curve, while curve III–III is a recompression curve representing the conditions which existed before the clay had reached an elastic state. Hvorslev's equation for the shear stress is $s = f\,\sigma + f_c p_e$.

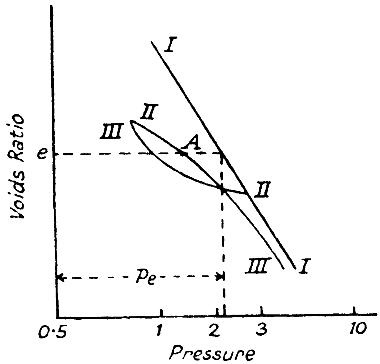


Fig. 46.—Showing relation between voids ratio and variations of load on soils.

where f is a frictional co-efficient,
f_c is a cohesional co-efficient,
p_e is the equivalent pressure,
s is the shear stress,
σ is the normal stress.

If this equation is valid, then the shearing resistance of a soil is a function of the effective normal stress and of the voids-ratio e in the plane of and at the instant of failure. It is, however, independent of the stress-history of the soil, since for a given value of e the equivalent pressure is involved.

A comparison of the results of fast and slow shearing tests led Hvorslev to the conclusion that soil viscosity may cause an apparent increase in the shearing resistance of from 10 to 20 per cent. in the rapid tests, and that if the test is stopped at a point at which plastic flow commences but at which the ultimate shearing resistance has not been reached, there may be a recovery of shearing resistance caused by thixotropic phenomena.

Zones of Discontinuity and Failure Lines.

The relationships outlined above apply only to earth masses in which continuity of strain can be assumed. This condition does not apply where earth masses contain fissures or cracks, or slip lines and shearing surfaces such as slickensides, i.e., zones of discontinuity which have resulted from the action of shearing stresses. In theoretical deductions, it is usually assumed that the shearing stress is distributed uniformly along a line of failure, and that failure occurs simultaneously along the whole length of the line. This assumption is unwarranted, and is used as one of many "design conventionalities" which, although scientifically unjustified, may sometimes give satisfactory results in practice. In reality, since neither the shearing stress nor the shear strength of an earth mass are constant along a failure line, equilibrium can only break down at one or more points in that line, after which the portion of the mass which has failed will be inactive, and different stress conditions will prevail in the remainder of the mass. It is accordingly extremely difficult and sometimes impossible to establish analytically the equation of a line of failure, although for some cases such lines can be deduced theoretically.

Failure Lines in an Elastic Continuum.

The maximum shearing stress at any point in an elastic continuum acts along the planes which bisect the angles between the principal stresses. In the case of a strip-loaded mass, the principal stresses act along lines of confocal ellipses and hyperbolæ, as shown in Fig. 47. In order to find the direction of the maximum shearing stress at any point A, a tangent T is drawn tangential to the ellipse or hyperbola of stress passing through the point A. A line T_1A, inclined at 45° to this tangent, is the direction of maximum shearing stress at point A (see also Figs. 22 and 23, pages 23 and 24). This theoretical treatment has been verified experimentally by W. S. Housel.

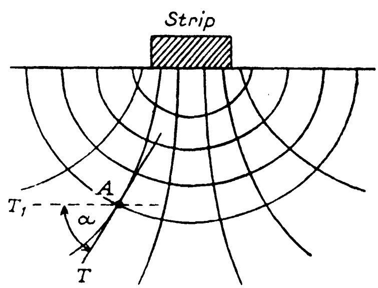


Fig. 47.—Showing location of a point of maximum shear in an elastic medium under a strip load.

Young's Modulus for a Silty Clay Soil.

The Road Research Laboratory at Harmondsworth, England, has made an investigation of the variation of Young's Modulus for a silty clay soil with moisture content and dry soil density, the dynamic method being applied, this method being, however, inapplicable to wet or dry clay soils. The conclusions reached were (see Figs. 48-50) :

1. Young's Modulus decreased rapidly with increasing water content from 600,000 or 800,000 lb. per sq. in. at 0 per cent. water content to between 20,000 and 40,000 lb. per sq. in. at 13 per cent. water content. This value for moisture .content corresponded with the shrinkage limit of the soil.
2. A linear relationship between Young's Modulus and the dry soil density was obtained for soils at moisture contents ranging from 0 per cent. to 13 per cent.

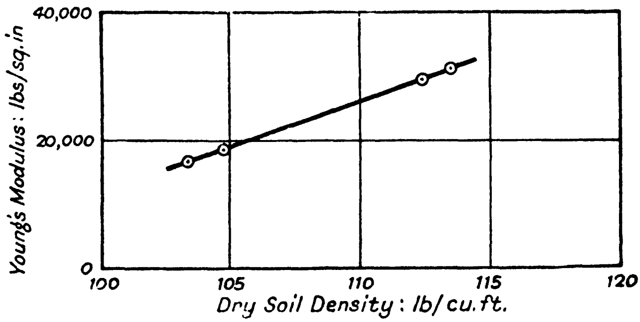


Fig. 48.—Showing relationship between modulus of elasticity and dry soil density at zero moisture content. (Road Research Laboratory).

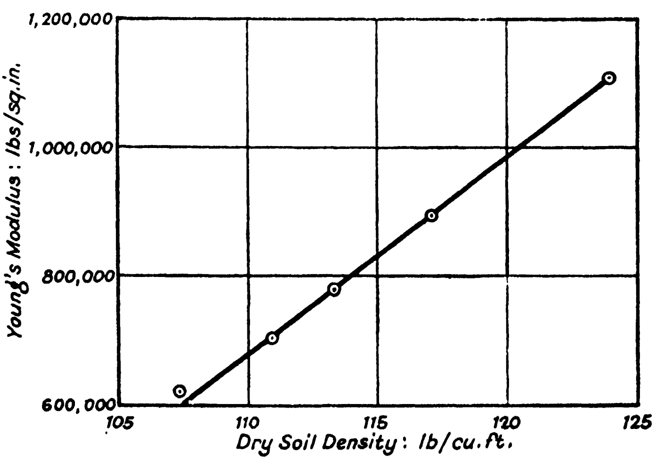


Fig. 49.—Showing relationship between modulus of elasticity and dry soil density at 13 per cent. moisture content. (Road Research Laboratory.)

It is evident that a great deal more research is needed before the exact relationship between the elastic properties of a soil and its physical constants can be determined with any degree of certainty.

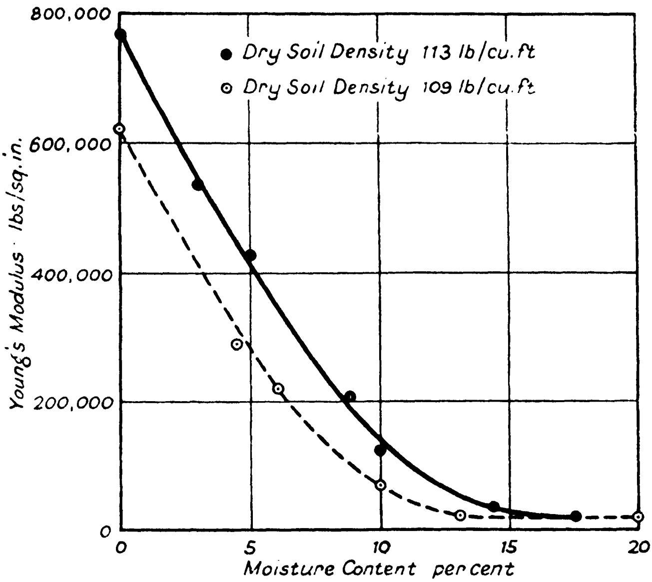


Fig. 50.—Showing variation in modulus of elasticity of a silty clay soil with changes in moisture content—based on dry soil density. (Road Research Laboratory.)

CHAPTER THREE

THE MINERALOGICAL AND OTHER PROPERTIES OF CLAY MINERALS.

Clays are usually defined as hydrous aluminium silicates, and in general this is a correct definition, but detailed studies in recent years have shown that clays exist as a series of isomorphous groups in which aluminous and non-aluminous members form a continuous series.

The Clay Minerals fall into three main groups, i.e., the *kaolin* minerals, the *montmorillonite* group, and a third and less fully understood group which includes the hydrous micas, the American *illites* and the " glimmerton " of the Germans. All these groups include minerals possessing a sheet-like structure and are therefore related mineralogically or crystallographically to the micas. The only other well-known clay minerals are allophane, an amorphous mineral without crystal structure, and a mineral described from America known as attapulgite, possessing a chain structure similar to that of the hornblende group of minerals ; neither allophane nor attapulgite is at all common.

Geological Relationships of the Clays.

The mineralogical identity and physical properties of the various clay minerals are a direct result of their geological history, i.e., their role as soil-making minerals is the result of the physical and chemical agents which have caused their formation. Thus the felspars are thought to give rise to kaolinite clay minerals on weathering, while the ferro-magnesian minerals, augite and hornblende, give rise to montmorillonite clays. It has long been known that kaolinite tends to form in an acid environment resulting either from humic acids or sulphides; kaolinitic clays are also the result of strong leaching action under neutral conditions. Both processes tend to remove bases from the system, and it appears that this factor rather than acidity as such controls their formation, i.e., kaolin minerals form most readily in systems characterised by a minimum of available elements other than silica, alumina and water.

Minerals of the montmorillonite group have been produced artificially in the presence of alkalies and alkali earths, but are known to have formed under the widest range of conditions in the presence of magnesium, and it is thought that the existence of suitable bases rather than alkalinity controls the formation of montmorillonite. Ferrous iron assists the formation of silicate minerals in a manner similar to that of magnesium, while the prevalence of reducing (or at any rate not actively oxidising) conditions tends also to the formation of minerals of this group. But kaolinite is commonly associated with iron oxides, since deep red-soil materials are generally kaolinitic, and this shows that kaolinite rather than montmorillonite is formed in the presence of iron when that element is completely oxidised to hematite, the reason being that the formation of hematite removes iron from the reacting system almost as effectively as a leaching process.

The weathering of rocks containing alkali felspars, especially such rocks as granites and pegmatites, tends to yield kaolinite, while rocks rich in lime felspars and pyroxenes may weather to montmorillonite. The hydrous micas and the clay minerals of marine shales and limestones also tend to weather to kaolinite. Rocks with abundant calcium, magnesium and iron, i.e., the basic rocks, tend to weather to a mineral of the montmorillonite type, although this tendency is affected by physical conditions to such an extent that when subject to active oxidation and leaching, basalts may give kaolinitic soils on weathering.

Members of the hydrous mica (illite or bravaisite group) are rare, occurring most widely in marine shales ; it is known that clay minerals tend to fix potassium at the expense of sodium, so that montmorillonites and kaolinites may in the course of time become changed to hydrous micas.

Kaolinite is in general the most stable of all the clay minerals and forms under the widest range of conditions. Most kaolinite forms at low temperatures, hydrothermal actions producing the variety dickite. Montmorillonite and its associated minerals also form at low temperatures, but this type of clay mineral has been recorded from the hot spring area of the Yellowstone Park of U.S.A. as forming at a temperature of 205 °C., and at pressures of 277 lb. per square inch.

The Properties of Clay Minerals.

(*i*) *Particle Shape.* Kaolinite, montmorillonite and illite all possess pronounced basal cleavages, since the forces binding the units together in the direction of the vertical axis are relatively weak. Fragments of these clay minerals occur as aggregates of flake-shaped crystals, and breakage along cleavage planes takes place more readily in the montmorillonite than in the kaolinite minerals ; the illite group are variable in this respect.

(*ii*) *Base Exchange Properties.* The base exchange properties of kaolinite are small as compared with those of the other groups ; thus illite is thought to possess base exchange properties two to three times and montmorillonite six to eight times as great as those of kaolinite. In all the clay minerals, base exchange capacity appears to increase with decrease in particle size.

(*iii*) *Layer lattice structure.* The general structural features of crystals of clay minerals are now well understood, and resemble Fig. 51(*a*) in general plan. In kaolinite and illite, the lattice structure does

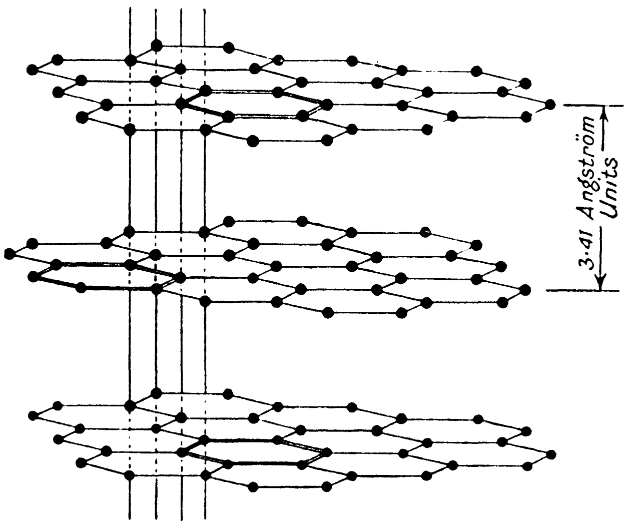


Fig. 51(*a*).—Crystal structure of graphite.

not expand with varying water content, but in the montmorillonite group, the structural units are loosely held together with water held between them ; their vertical dimension varies with varying water content, and the group is said to have an expanding lattice, i.e., its members are what are known as " swell " clays. The importance of this feature in foundation engineering can hardly be over-estimated.

(*iv*) *Relation of Structure to Base Exchange and Particle Size.* As mentioned above, montmorillonite has a high base exchange capacity, higher than can be accounted for merely by broken bonds occurring at the edges of fragments. The units are held together lightly, causing the lattice structure to expand when water enters the crystal between the flat faces of the units, and it is thought that in this way hydration of the exchangeable cation pushes the units apart. In illite, the unit layers cannot expand in this way, and so cannot expose a huge area between the units for base exchange ; as a consequence, base exchange is much lower for illite than for montmorillonite. In addition, illite does not break down under mechanical agitation like montmorillonite, because its unit layers are held tightly together.

In the case of kaolinite, replacements do not generally take place, and the small base-exchange capacity possessed by kaolinite must therefore be attributed to broken bonds on the edges of the fragments. The kaolinite structure is not of the expanding type, and the mineral does not break down into flakes of very small size.

(*v*) *Suggested Structure of Clays.* Clays are essentially aggregates of very minute crystals in flake form, possessing forces of varying intensity which tend to attract individuals to each other and to hold them together, i.e., cohesive forces. These forces may be considered as acting mainly from the flat surfaces of the flakes, since these surfaces make up the greater percentage of the total surface area of the flakes.

When moisture is added to clay minerals, a film of water develops on the flat surfaces of the flakes, probably because of attracting forces, and because of the presence of adsorbed cations which tend to become hydrated. This water separates individual flakes, and also acts as a lubricant between them. Water is a di-polar liquid, and the film between individual flakes is probably made up of oriented di-polar molecules, movement being possible along planes of di-pole ends (see Fig. 51*b*).

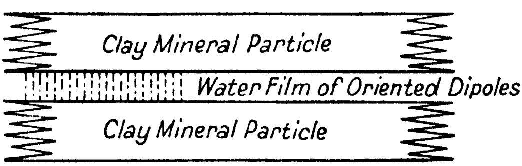


Fig. 51(*b*).—Showing role of water film in a clay particle.

Such physical properties as plasticity, bond strength and shrinkage may be ascribed to (*a*) the structure and composition of constituent clay minerals, these determining particle size when the mineral is worked with water, (*b*) the attractional force between particles, (*c*) the character of the exchangeable bases.

The Mineralogy of Clays.

Clay minerals were for a long time neglected by mineralogists, and even now do not receive the study which their importance warrants. This has been to a large extent unavoidable, since early methods for establishing the purity of such minerals were inadequate, and since both impure and pure types were studied without proper differentiation between them. Among early workers, E. S. Dana was a pioneer in assigning species to the many minerals described, and his early nomenclature still holds good. In this way, a partial knowledge of pure substances representing most of the clay minerals was obtained, but the mineralogical methods available for the study of fine-grained materials were so primitive that the detailed mineralogy of soils, clays and shales involved a great deal of guesswork. Thus in spite of the fact that montmorillonite had long since been recognised as a definite clay mineral, kaolinite was looked upon as the typical mineral in clays, and all kinds of supposed mixtures were postulated in order to try and make the results of analysis fit the various assumptions which were made. Yet another fallacy was upheld by workers who tried to maintain that soils were made up of amorphous oxides and hydroxides of silica, alumina and iron, although crystallinity was easily visible under the microscope even in the smallest particles. There was even a reluctance to accept the fact that clays are made up of definite mineral substances. But improved methods of mineralogical research have fostered the study of the clay minerals, and have gradually led to the classification of groups of related minerals ; thus clays began to be worked out systematically some 20 years ago, and since then steady progress in our knowledge of these somewhat difficult minerals has been achieved.

Chemical analysis of pure species was the first step in this progress, coupled with crystallographic and petrological methods as perfected by A. B. Dick over 30 years ago. The author uses a Swift-Dick petrological microscope fitted with special Zeiss objectives for determinations of clay species, and finds such apparatus very satisfactory for the purpose. Samples passing a 100 mesh sieve are mounted in Canada balsam and compared with a series of slides of typical clay minerals ; in the course of work done with W. H. Cohen, clays down to $0{\cdot}2\mu$ size were dealt with by this means, this necessitating the use of a high-magnification ocular together with a 1/12 inch oil-immersion objective and standardised lighting conditions.

The role of water in clay minerals has always been a difficult question, since such water is commonly present in two or more forms, i.e., adsorbed water and interlayer water, while so-called high-temperature water or more correctly OH, as commonly shown in chemical analysis, is simply one of the several oxide groups forming a definite part of the crystal lattice. Both adsorbed and interlayer water affect the behaviour of clays used in engineering works, the former tending to push the soil grains apart, and the latter causing swelling of the individual clay grains ; adsorption only is characteristic of the kaolin group of clays, and adsorption, together with swell, of the montmorillonite group.

This chapter would be incomplete without some reference to X-ray studies of clay minerals. The lack of well-defined external form, the wide variations in chemical composition, the exceeding fineness of grain, the small differences in refractive index as compared with that of Canada balsam, and the minute differences in birefringence found in most clay minerals make the more exact data furnished by X-rays applied to pure samples of the utmost value in clay determinations, so much so that it is not surprising to find that the relationships of clay minerals one to another and to other plate-like minerals, the relations of the different ions in the crystal structure, and the substitution replacement of one ion by another have been finally determined by X-ray technique. Even later than the latter technique comes the use of the electron microscope, which, in spite of its inherent limitations as an instrument of mineralogical investigation, has utilised the very fine grain and ready dispersibility of the clays for the elucidation of data concerning shapes of clay crystals. Thus halloysite has been shown to possess elongated, blade-like crystals inherited from the endellite whence it is derived, while the pseudo-hexagonal habit of kaolinite persists down to the finest fractions. Montmorillonite, on the other hand, shows little or no characteristic form.

Colour reactions produced by the use of various staining media have proved useful in clay identifications, since a number of organic compounds, especially the aromatic amines and phenols, produce reactions giving a colour change in the presence of certain clay minerals. S. B. Hendricks and L. T. Alexander,[9] following A. Eisenach[10], proposed the use of benzidine for this purpose, while E. A. Hauser and M. B. Leggett[11] studied reactions between amines and clays. An extension of this method, using malachite green solutions, has been developed by W. H. Cohen and the author[12] for the determination of the surface area of clay fractions on an adsorption basis, and has reached a stage at which the presence of clay fractions which are likely to lead to instability in road and aerodrome foundations can be determined with a considerable degree of accuracy. The method is as follows :—

Dye Adsorption Test for Clay Fractions.

0·2 gram of dry soil passing a 40 mesh sieve is placed in a clean conical flask. (All apparatus used in the method must be washed only with distilled water). 0·1 gram solid malachite green dye in 1 ml. water at room temperature is the strength of the dye solution used ; 80 to 100 c.c. of the solution is added to the soil sample by means of a burette, the amount of dye added being noted. The flask is corked and the mixture periodically stirred and shaken ; the flask and its contents are left to stand for 18 to 24 hours, the contents filtered through a dry No. 5 Whatman's 12·5 cm. filter paper, and the filtrate placed in a test-tube. The depth of colour of the filtrate is measured in a Luximeter photo-cell apparatus (see Fig. 52), and the galvanometer reading

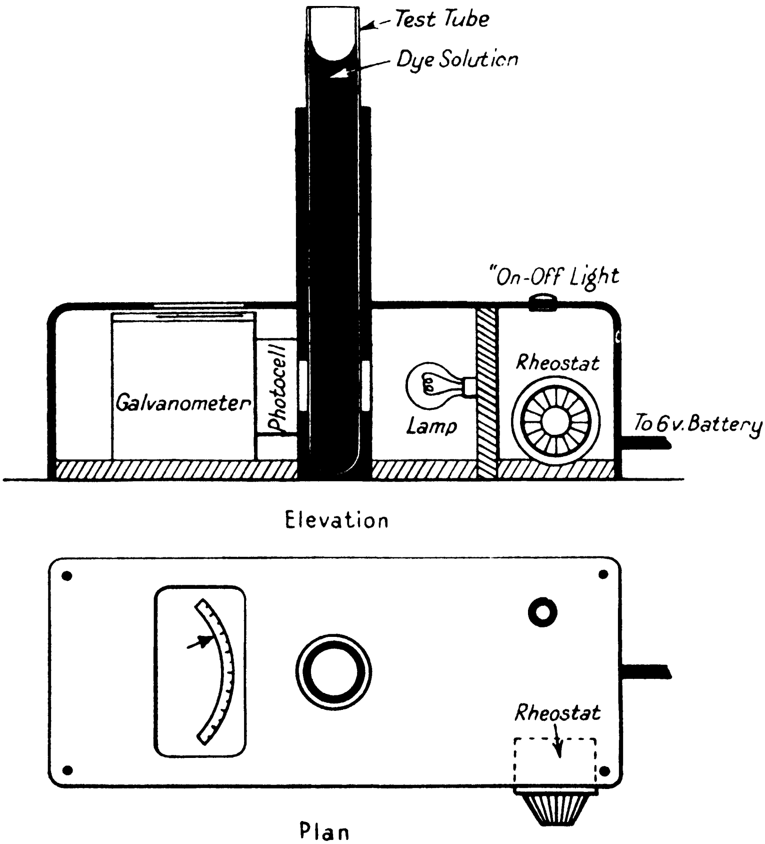


Fig. 52.—Diagram of arrangement of photo-electric cell apparatus used for dye adsorption tests of clay soils.

recorded. The test should be carried out in darkness as far as possible, since the dye loses its colour slowly in the presence of daylight. Depending on the amount of dye solution used, the appropriate correction-calibration curve (see Fig. 53) is used to correct the galvanometer

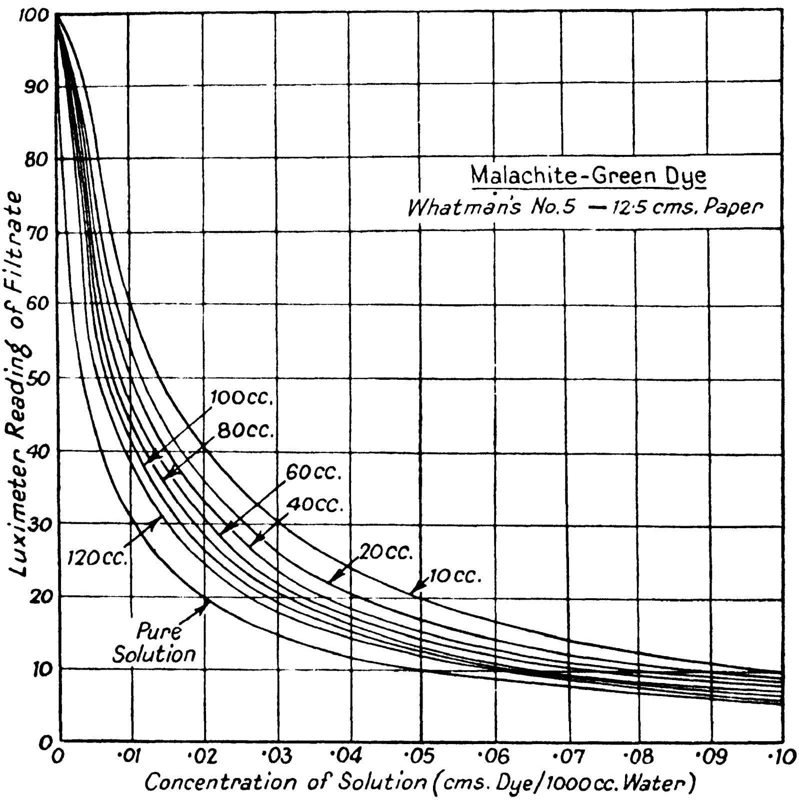


Fig. 53.—Colour calibration curves used to determine colour of dye extracted during filtration in dye-adsorption tests on clays.

reading for the amount of colour absorbed by the filter paper ; the reading is thus converted to the concentration of dye still remaining unadsorbed in the filtrate.

Since the initial and final concentrations of the dye, and also the original weight of the soil sample are known, the grams of dye adsorbed per gram of soil sample can be found from the formula

$$\frac{(C_1 - C_2)Q}{1{,}000W}$$

where C_1 = initial concentration of dye in grams per litre,
C_2 = corrected final concentration of dye in grams per litre,
W = weight of soil sample in grams,
Q = Quantity of dye solution added in mls.

The higher the figure thus obtained by this test, the greater the adsorptive capacity of the soil and the more unsuitable for road foundation purposes the clay fraction tested. Thus a good roadmaking soil gives a figure of about 0·0105 grams of dye adsorbed per gram of soil, whereas a bad roadmaking soil gives a figure of 0·07724 grams of dye adsorbed per gram of soil. A high adsorption figure indicates the presence of a large amount of the swell clays of the montmorillonite group, while a low adsorption figure shows the presence of non-swell clays of the kaolinite group. Details of the theory on which this test is based are available[8].

Impurities in Clays.

Clays are minerals peculiarly subject to contamination from impurities. All soils are mixtures of minerals, so that samples of even approximate purity suitable for detailed mineralogical investigation are extremely rare.

The identity of many of the impurities in clays can be determined fairly accurately, but when appreciable amounts of such impurities occur, or where the grain-size is excessively fine, or where two or more clay minerals are present, the approximate estimation of proportions involves an appreciable margin of error, even by the combined use of chemical analysis, X-rays, petrographical examination and thermal analysis, i.e., heating at a constantly increasing temperature. The likely impurities which make clay determinations difficult are very variable; thus soils in general may contain finely divided detrital minerals such as quartz, felspars, micas, carbonates, iron and titanium oxides and other less common minerals, in addition to organic matter. These impurities are so varied that more than one method may have to be used for their detection. Of the three chief clay groups, only the kaolinite group possesses chemical compositions definite enough to ensure that variations from them are an indication of the presence of impurities; all other clays may have marked variations in the alumina-silica ratio and in accessory bases, while excess silica or alumina may be present and yet may escape detection by chemical analysis. Another difficulty is that different clay mineral species may possess similar X-ray patterns, while the poorly defined patterns which they often give are not always conclusive when more than one species is present, although well-defined impurities are easily identifiable by X-ray analysis. The thermal analysis method referred to by F. H. Norton[13] will detect kaolinite, gibbsite or montmorillonite, and will also detect goethite, gypsum, diaspore, boehmite or the various carbonates. The removal of the iron oxides and hydrous oxides from clays without at the same time materially affecting the iron which is an essential part of the crystal lattice has been a real difficulty for which a method has been perfected by H. G. Dion[14]; the opacity caused by such coatings hinders petrographic determinations, although the author has found that the use of the edges of the finer fractions facilitates such determinations, since such edges are usually unobscured.

The Kaolin Group.

The clay minerals of the kaolin group crystallise more completely and possess simpler chemical relationships than any other group of clay minerals, so that their chief mineralogical relationships have been understood for quite a long time. The minerals of the group are kaolinite, halloysite, dickite, nacrite, endellite and allophane. Of these, only kaolinite and halloysite occur with sufficient frequency to be of much importance to engineers concerned with soils. Kaolinite, halloysite, dickite and nacrite all have the same chemical composition and the same basic space lattice, and differ only in the manner in which the lattice sheets are stacked one upon another.

Halloysite and some kaolinites are so fine-grained that they appear to be isotropic ; allophane is an amorphous material without an organised space lattice, and its composition can vary within wide limits. The space lattice of the kaolin minerals is such that there is little substitution of ions in its structure. Thus an iron-bearing kaolinite (faratsihite) was described by A. Lacroix in 1914, but later work by S. B. Hendricks has shown that the material is a mixture of kaolinite and nontronite (see under montmorillonite group for this latter mineral), and this worker concluded that there was in fact no replacement of alumina by iron in kaolinite. The constancy with which the chemical analyses of kaolinite correspond with the theoretical formula [$Al_2Si_2O_5(OH)_4$] indicates that Al does not replace Si as it does in other clay minerals. The space lattice of the kaolin minerals is such that there are no unsatisfied valences on the cleavage surfaces, and hence there is no adsorption of exchangeable bases on these surfaces. The edge of the sheet will have unsatisfied valences, and there will be a tendency for exchangeable bases to be held on these edges, but the area of the edges of the sheets will be so small compared with the total area of the crystal that exchangeable bases will be insignificant except where the material has been subjected to very fine grinding.

The Montmorillonite Group.

This important group of clay minerals includes swell clays, the minerals of which are composed of mica-like sheets one molecule thick without external crystal form, the members of the group being very variable in chemical composition. Water plays several roles in their constitution, and they possess marked base exchange properties.

Chemical analyses alone do not enable us to understand the structure and physical behaviour of this group of clays ; it is necessary also to consider the relationship of the ions within the crystal structure. The unvarying constants in the lattice structure of these minerals are the number of ions occupying tetrahedral positions, together with the number of oxygen ions, and consideration of these factors enables us to prove the validity of all formulæ relating to the group, basing them on the number of ions in the unit cell or the more convenient half-cell. Thus the Si and the tetrahedral Al must always equal 4, while O + OH must equal 12, and the maintenance of these constants in all formulæ permits of ready comparison between various members of the group, taking first the micas as a basis of comparison.

In formula (1) below, the inter-layer ions are written first, followed by the group occupying tetrahedral positions in the layer lattice, then by the tetrahedral ones, and finally by the O, OH and F. In montmorillonite (see formula (6) later) the order is the same, except that for convenience (Na,Ca) follows $(OH)_2$, and indefinite inter-layer water, represented as (Aq), is last.

Suggested ideal formulæ for the micas, talc and pyrophyllite, all of which possess sheet-like structures and to which the montmorillonite group is closely related, are as follows :—

Muscovite $(K_{0\cdot87}Na_{0\cdot14})(Al_{1\cdot76}Fe''_{0\cdot08}Fe''_{0\cdot05}Mg_{0\cdot12})(Al_{0\cdot86}Si_{3\cdot14})O_{10}(OH.F)_2$ ......(1)

Talc $Mg_3Si_4O_{10}(OH)_2$ ......(2)

Pyrophyllite $Al_2Si_4O_{10}(OH)_2$ ......(3)

Muscovite $K.Al_2(AlSi_3)O_{10}(OH)_2$ ......(4)

Biotite $K(Mg,Fe)_3(AlSi_3)O_{10}(OH)_2$ ......(5)

Formulæ (4) and (5) above represent mica formulæ as commonly written.

Three octahedral positions are possible within the crystal lattice, although a stable lattice structuie exists when only two of these are occupied, as in the case of pyrophyllite. When Mg replaces Al, the three bivalent ions find a place in the crystal structure, valence balance being maintained. In pyrophyllite and talc, no essential substitutions of Al for Si exist, and there is complete balance of valences within the lattice structure, this accounting for the well-known stability of these minerals.

In the micas, the same relations exist between the two Al ions in muscovite and the three Mg ferrous Fe ions in biotite as in pyrophyllite and talc, but one of the four tetrahedral positions in the crystal structure is occupied by Al, i.e., Al replaces Si in one of these positions. This substitution of a trivalent ion Al for a tetravalent Si leaves a valence deficiency of 1, which is balanced by the K ion which occupies a position between the lattice sheets, and ties sheet to sheet.

In members of the montmorillonite group, the unsatisfied valence caused by substitution of ions within the crystal lattice is usually only about one-third as great as in the micas. This valence deficiency may result from the substitution of ions of lower valence in either octahedral or tetrahedral positions, bivalent Mg commonly replacing trivalent Al, and trivalent Al replacing tetravalent Si. The montmorillonite end member of the group possesses a minimum substitution of Al for Si, the essential replacement being that of Mg for Al.

Formula (6) below can be taken as typical for montmorillonite ; it should be noted that in the highly aluminous members of the group, Al replaces Si in tetrahedral positions :—

Montmorillonite $(Al_{1\cdot 66}Mg_{0\cdot 33})Si_4O_{10}(OH_2)Na_{0\cdot 33}Aq.$ ....(6)

Beidellite $(Al_2(Al_{0\cdot 33}Si_{3\cdot 66})O_{10}(OH_2)Na_{0\cdot 33}Aq.$ ....(7)

It should also be noted that a beidellite of a composition approximately that of formula (8) below is known ; since the Al ions in octahedral positions may exceed two, three such positions being available, a more aluminous member of the group is possible.

Ferric iron can replace Al in octahedral positions in any proportions, but cannot occupy tetrahedral ones, so that the ferric iron member of the group, nontronite, differs from montmorillonite and beidellite only in being characterised by the presence of iron. Formula (9) below, represents nontronite :

Beidellite $Al_{2\cdot 18}(Al_{0\cdot 86}Si_{3\cdot 14})O_{10}(OH_2)Na_{0\cdot 33}Aq.$ ....**(8)**

Nontronite $Fe_2(Al_{0\cdot 33}Si_{3\cdot 66})O_{10}(OH_2)Na_{0\cdot 33}Aq.$ ....**(9)**

Complete isomorphism is thought to exist within the montmorillonite group as between montmorillonite, beidellite and nontronite. The commonest members of the group are the aluminous species in which Al is the essential ion occupying tetrahedral positions and in which varying amounts of Al replace Si in those positions. There are usually minor but none the less essential substitutions of Mg for Al, and of Fe for Al. The replacement of Al or Si ions by others of lower valence causes the valence of the lattice structure to be out of balance, and this is compensated for by inter-layer exchangeable ions, which may be either Ca or Na, Ca generally predominating; Ca being bivalent and Na monovalent, only half as many Ca atoms is required to balance the valences.

As already pointed out, in the mica group potassium (K) links one molecular sheet with its neighbour; in the montmorillonite group, the exchangeable bases are linked only to a single surface. This permits the latter group to hydrolyse and so to hold water layers between the individual lattice sheets; thus the physical properties of the group are due to the exchangeable bases which provide a water film between each pair of layers. The thickness of this water film varies with the humidity; under normal humidity, the basal spacing is about 15 Angström units. It is this variation of the basal spacing with changes in humidity that causes the troublesome expansion and contraction experienced with clays of the montmorillonite group.

The two chief exchangeable bases Ca and Na yield rather different physical effects on wetting; thus calcium clays do not disperse completely like sodium clays; they break down to finely granular aggregates and give a minimum of material in permanent suspension. Sodium clays, on the other hand, may disperse so completely that they can give permanent suspensions in the form of gel-like masses. Chemical analyses combined with X-ray studies have shown that sodium clays commonly carry an excess of silica which may be present in the colloidal form or as cristobalite.

The Hydrous Micas (Illite or Bravaisite Group).

The hydrous micas forming the third and last group of clay minerals have been called " illite " by R. E. Grim and " glimmerton " (= shiny clay) by German mineralogists. As long ago as 1878, R. E. Mallard gave the name " bravaisite " to the specific mineral similar to muscovite, but with approximately half the K_2O which characterises muscovite. C. S. Ross suggests the use of the term " bravaisite " for this group of minerals, but at the time of writing there is no uniformly accepted name for it, chiefly because the specific characters of the mica-like clays have not yet been fully worked out.

Mica-like minerals in soils were first recognised by S. B. Hendricks and W. H. Fry in 1930, who also found them in the potash-rich bentonites covering large areas of Ordovician rocks in Eastern U.S.A., and minerals of this type resemble micas and minerals of the montmorillonite group in that Al partly replaces Si in tetrahedral positions, and that four of these positions must be occupied for each half unit cell. The ions in octahedral positions are dominantly Al, with lesser proportions of Mg, ferrous and ferric iron. The role of Ca and Na in minerals of the illite group is not fully understood, but they are thought to occupy positions between the lattice sheets, at any rate in part. It is, however, known that the base exchange capacities of these clays is much lower than in members of the montmorillonite group, but higher than those of the kaolinite group. As regards stability, they therefore probably occupy a position intermediate between these groups.

Clays of the hydrous mica type characterise shales formed under marine conditions and are present in soils derived from such shales. The author has examined many samples of the Karroo Shales of the Permo-Carboniferous System of South Africa, but has not observed any clay minerals of the illite group therein ; the shales referred to are thought to have been lacustrine in origin.

The diagnostic properties of the commoner types of clay minerals have been tabulated by C. E. Marshall[15] and C. S. Ross[16] as shown on pages 78 and 79.

Thermal methods have been developed in recent years as a means of identification of clay minerals and mixtures of such minerals. Thus G. M. Schafer and M. B. Russel[17] have studied thermal curves for various clay minerals, and by the use of the endothermic peaks of such curves claim to be able to determine the amounts of different clay minerals present in the soil. This method does not appear to have come into very extended use up to the present.

Rehydration methods are described by L. H. Berkelhammer[18] in which the rehydration characteristics of a group of twenty clays were obtained by dehydrating at 300° C., and then rehydrating by exposure to a water-saturated atmosphere for periods of from 28 to 80 days. A sharp differentiation was obtained between the montmorillonite and the kaolinite-illite types of clays by this means.

TABLE 3.—OPTICAL PROPERTIES OF THE COMMONER CLAY MINERALS.

Mineral	Habit	Indices of refraction		Sign and birefringence	Diagnostic Tests
		α	γ		
Kaolinite	Plates, forming curved groups; straight extinction; worm-like.	1·560	1·566	0·006 Negative	Plates lying parallel to the slide show $n\alpha$ and $n\beta$. $n\beta = 1.564$. Aggregates of lath-shapes. High R.I. and birefringence.
Dickite	Monoclinic; platy; equi-dimensional grains.	1·560	1·566	0·006 Positive	Plates standing on edge usually show oblique extinction 15° to 20°.
Halloysite	Isotropic; rod-like; conchoidal fracture	Variable, but usually below 1·54		None	Useful index of presence of kaolinite with which it is commonly associated; low R.I., no birefringence.
Illite-bravaisite type	Mica-like; straight extinction	1·525	1·567	0·042	Variable R.I., and high birefringence
Montmorillonite	Very fine-grained and shred-like, but clearly crystalline; areas of elongated plates.	1·478 to 1·512	1·535 to 1·500	0·023 to 0·057	Low R.I. and high birefringence, but both variable.
Beidellite	Plates or short blades	1·488 to 1·523	1·527 to 1·572	0·004 to 0·084	Usually low R.I. but variable; birefringence very variable both depending on amount of Fe_2O_3.
Nontronite	Sometimes very fine-grained; shred-like, but clearly crystalline.	1·56 to 1·63	1·585 to 1·66	0·03 to 0·10	Usually low R.I. but variable; birefringence very variable, but usually high. Pleochroic brown to green.

(*Note*: R.I. = Refractive Index).

TABLE 4.—OPTICAL PROPERTIES OF CLAY MINERALS IN SIMPLIFIED FORM.

Type of Clay	Optical Properties				
	Habit and Appearance	Refractive Index	Colour	Birefringence	Reflected Light
Kaolinite	Typically flaky, sharp edge, pointed, sometimes with re-entrant angle. Sometimes almost isotropic.	Moderate, above Canada balsam	Colourless	Weak	No reaction.
Halloysite	Sometimes long rod-like crystals with saw-tooth end, some round, isotropic, has a rough surface; sometimes some stumpy crystals obtained.	Low	Colourless	None	Slightly pearly lustre.
Montmorillonite	Edges of crystals extremely irregular, characteristic flaky appearance and except in the very small sizes it has a brownish colour. Crystals often worm-shaped.	Just above or just below that of Canada balsam	Brownish (except in the very small sizes)	Moderate. Aggregate polarisation.	No reaction
Bentonite	Varies from subangular to rounded more or less shapeless grains. Aggregate crystals (very fine) common, showing undulose extinction.	Low. Below Canada balsam.	Colourless. Some almost transparent and slightly mottly. Majority are translucent of almost a dirty brown colour.	Strong.	Pearly.
Nontronite	Grains subangular but much less ragged than most clays at the edges.	High. Shadow polarisation. (Compass Needle).	Black, red to brown, may be pleochroic (green to brown).	Moderate	Bright speckly lustre.
Illite (Pyrophyllite)	Typically ragged and flaky. Shows aggregate polarisation. *cf.* kaolinite. Shows perfect cleavage. Crystals commonly needle-shaped.	High. Above that of Canada balsam.	Colourless.	Strong.	Pearly. White to transparent.

The electron microscope has been used to an increasing extent for clay mineral identification and has been the means of establishing the crystal habit of clay minerals such as kaolin with a high degree of precision, besides showing the marked difference in such habit as compared with that of montmorillonite.

Moisture Conditions of Clay Minerals.

It will be clear from the foregoing that there is an intimate connection between the moisture conditions of clay minerals and the physical properties of the cohesive soils formed by them. Thus the coefficients of internal friction and permeability decrease with decreasing quartz content, while a high quartz content results in reduced compressibility and settlement. The properties of powdered quartz and kaolinite are close to one another in this respect, but montmorillonite has the highest compaction value of any common clay mineral, this property being associated with potential danger due to slipping, ability to take up moisture, low permeability and marked swell tendencies. It is also known that sands bonded with kaolinite or halloysite clays develop an " air-set " after moistening, compaction and subsequent slow drying; this bonding action does not occur with montmorillonite or illite clays. All the foregoing are of importance to the engineer who is concerned with the use of soils.

CHAPTER FOUR.

SOIL SAMPLING AND SOIL SURVEYS.

The data in this chapter is taken largely from Road Research Bulletin No. 4 (Department of Scientific and Industrial Research, Great Britain, 1946), "Soil Survey Procedure and Its Application," to which the reader who desires fuller information is referred.

Introduction.

The purpose of a soil survey is to give the engineer relevant information as to soil and ground water conditions existing on road and airfield sites. Such a survey should form an essential part of a preliminary engineering survey ; casual inspection or even occasional trial-holes are not enough for this purpose. The cost of such a survey should only be about the same as that of taking levels and contours over a site, and should therefore form only a very small fraction of construction costs, while mistakes and failures caused by insufficient knowledge of site conditions may prove very costly indeed.

A soil survey provides vertical sections of the ground, showing the nature of the soil in different strata, and is obtained by hand-boring methods. Such sections are known as soil profiles ; from such profiles and from laboratory investigations, deductions can be made concerning:

1. Suitability of proposed location, both horizontally and vertically.
2. Selection of suitable materials for embankments.
3. Determination of suitable cross-sections for cuts and fills.
4. Requirements for subsoil and surface drainage.
5. Treatment of sub-grades.
6. Design of base courses and of surfacings for road and airfield works.
7. Possible use of local materials for soil stabilisation.

Personnel. One engineer and two or three intelligent labourers should be able to carry out about 200 lineal feet of boring per mile of road at a time or 300 lineal feet of boring for airfield sites.

Authorities. Large-scale survey maps will be required, and if these are not available, a survey of the proposed site will have to be made, and the lay-out of the subsequent borings shown on such plan.

In the case of a road, a single line of borings along the centre line or a double line offset to a distance of 50 or 100 feet on either side of the centre line will suffice ; in the case of airfields, a grid with 300 feet squares is adequate.

Site Reconnaissance. A preliminary reconnaissance is made by walking over the ground before any boring is commenced. Quarries, pits, cuttings and scarps can then be examined, and the general topography and vegetation studied, such features as geological faults or unconformities looked for, and from the reconnaissance the intervals between the borings decided on. These may be as much as 1,000 feet in very uniform ground, or as little as 50 feet in rapidly changing ground such as is found in glacial deposits.

Determination of the Soil Profile.

Setting Out. The positions for the boreholes are set out and marked with white-painted pegs, poles being used in long grass ; each peg should be numbered. In the case of developed country, chain and/or tape only will be required ; in undeveloped country, tacheometer work will be required.

Boring. Normally, the depth of boring may be 4-5 feet below existing ground level or proposed formation level, whichever is the lower. The depth bored can be less in valleys where filling is likely, and greater on hills where cuts may be required. If a bed of peat or soft clay is encountered, it should be penetrated completely and its boundaries located as exactly as possible. When rock is encountered in any part of the boring operations, it is obviously unnecessary to proceed further with the boring, except in the case of soil such as boulder clay. After boring is complete and the necessary samples taken, all holes should be filled in and the pegs collected for re-use.

Records. Complete boring records should be kept on sheets such as are shown in Table 5.

Location of Water-Table. If no trace of water be found in a boring, the hole can be filled immediately after sampling, but if water be found, the hole should be left for 12 to 24 hours to enable the water to reach its final level, after which its depth below the surface can be measured and recorded.

TABLE 5.—TYPICAL BORING RECORD SHEET.

Site : Puddletown.

Date : 15th November, 1947.

Boring No. : 63

Chainage : 5450.

Natural Ground Level : 643·6.

Depth below proposed Road Formation : 5·5 feet.

Level of Ground Water above datum : 638·5.

Depth of Ground Water below surface : 5·1 feet.

Remarks : In copse land, near bottom of steep bank ; 2 inches of dead leaves were removed before boring was commenced.

Depth below surface		Sample Tin No.	Behaviour of ground water	Natural Moisture Content %	Description of Sample and Remarks
Feet	Inches				
0	0	55	—	8·8	Light silty top soil.
1	3	56	—	12·7	Green silty clay
2	3	57	—	16·9	Brown sandy clay.
3	6	58	—	17·7	Plastic blue clay.
5	3	59	—	16·9	Mottled sandy clay.
6	9	60	Seeped into hole	24·7	Green sandy clay.
7	9	61	Standing in hole	Water-logged	Green sand.

Simple Identification Tests for Soil.

Before describing boring methods, it will be as well to describe some simple identification tests for soil so that the engineer in charge of boring operations will have some idea what to look for in the course of the work. These tests do not need apparatus and can be carried out easily in the field.

Sand. Coarse and medium sands can be identified readily by visual inspection. It is, however, easy to mistake a very fine sand for a silt, but this mistake can be avoided if samples are examined visually and manually in the dry condition. Dried samples of fine sand exhibit no cohesion, but appear gritty, while dried samples of silt show marked cohesion together with absence of grittiness.

Silt. The particles of a silt are mostly invisible to the eye, and some difficulty may be experienced in distinguishing silts from clays. Two simple tests may be applied for this purpose: (a) *The Shaking Test,* in which a small portion of the wet soil is shaken horizontally in the palm of the hand. If the soil is a silt, water will come to the surface of the soil, which will then appear glossy and rather soft. If the sample is then squeezed between the fingers, the water leaves the surface of the soil, which seems to dry up. The soil finally stiffens, cracks and crumbles. (b) *The Breaking Test.* The sample is allowed to dry, when its cohesion and feel is tested by breaking it with the fingers ; a typical silt feels smooth to the touch, but does not show very marked cohesion, while a clay feels very smooth to the touch and shows marked cohesion.

Soil Types.

H. A. Mohr[21] gives much useful information relevant to the subject of this chapter, and the following notes on soil types :—

Sand is the coarsest of the soils, and ranges from coarse sand passing a ¼ inch mesh sieve and retained on a 28 mesh sieve down to fine sand passing a 65 mesh sieve and retained on a 100 mesh sieve. It can be almost any colour according to its cleanliness, and it has no cohesive properties.

Silt is an inorganic sedimentary deposit of a fine-grained nature intermediate in grain-size between sand and clay ; when dry, it lacks plasticity and possesses little or no cohesion. In some countries it is known as " rock flour." Its colour can be almost any of the colours normally met with in sedimentary deposits, more especially the clays, and as a soil it is extremely treacherous.

Clay is an inorganic sedimentary deposit consisting largely of very fine mineral grains ranging well down into quite colloidal sizes, and displaying, between certain water contents, the property known as plasticity. Depending on the degree of plasticity, it is possible to distinguish sandy or silty clays of low plasticity ranging down to fine-grained highly plastic clays. When wet, a clay can be kneaded like dough, and when dry, it forms a hard mass which cannot be separated into individual grains by finger pressure. *Hard clay* cannot be remoulded by the fingers, and requires the use of a pick for its excavation. *Medium clay* can be remoulded by using considerable finger pressure, and can be excavated with a spade. *Soft clay* can be easily remoulded by the fingers and can be readily excavated with a shovel. Clays have colours ranging from grey to almost white, greenish, brown, reddish and bluish. Apart from convenience of description, the colour classification has no significance.

Peat is partially carbonised vegetable matter ; it is composed of annual deposits of dead vegetation protected from decomposition by almost continual submergence under water. It contains leaves, grass and branches of trees and bushes and is fibrous in texture ; it is from light-brown to black in colour, is odoriferous and a most treacherous soil, which will not support any additional constantly applied load without a considerable reduction in volume.

Size of Soil Sample.

Usual sizes of soil sample are given in Table 6 :—

TABLE 6.—SIZES OF SOIL SAMPLE.

Purpose of Sample	Soil Type	Minimum weight of sample (lbs.)
Soil identification and natural moisture content tests	Cohesive soils and sands.	1½
	Gravels.	7
Compaction tests	Cohesive soils and sands.	10
	Gravelly soils.	20
Soil stabilisation (sand-mix)	Cohesive soils, sands and gravelly soils	10
Soil stabilisation (other processes)	Cohesive soils, sands and gravelly soils	50 to 100

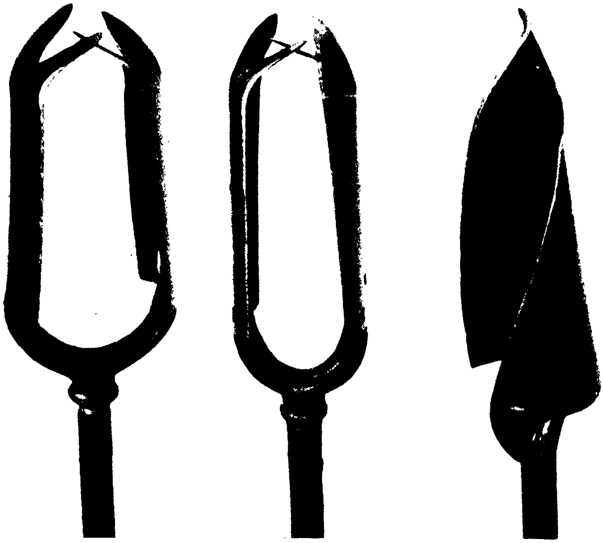
Equipment for Soil Surveys.

A list of equipment for soil surveys is given in Table 7. It will, of course, be understood that the actual items required vary very largely according to the nature and extent of the particular survey. Of this equipment, it is only necessary to describe in any detail that used for boring purposes.

Boring Equipment. Fig. 54 shows details of the 4 inch to 5 inch diameter post-hole auger commonly used for sampling ; in the case of stony ground, a gravel auger is used. A more accurate method than that of the ordinary post-hole auger is that used by the Road Research Laboratory, which permits small samples to be obtained correct to depths of ½ inch, and also accurate study of seasonal variations in moisture content. The sampling tool is shown in Fig. 55. The tool consists of a helical cutter designed to collect and retain any type of soil in its enclosed portion, a removable brass sleeve permitting extraction of the sample, while a pointed rod facilitates accurate location of the position whence soil is removed. A hole is bored with a 4 inch diameter post-hole auger to the approximate depth at which the first sample is required. All loose soil is removed from the bottom of the hole with the auger, and the point of the sampling tool is pushed into the base of the hole. The tool is then rotated about six times in order to collect a sample of about 20 grams of soil. A mark is made on the extension rod, level with the surface of the ground, and the distance of the cutting edge from it measured accurately with a tape.

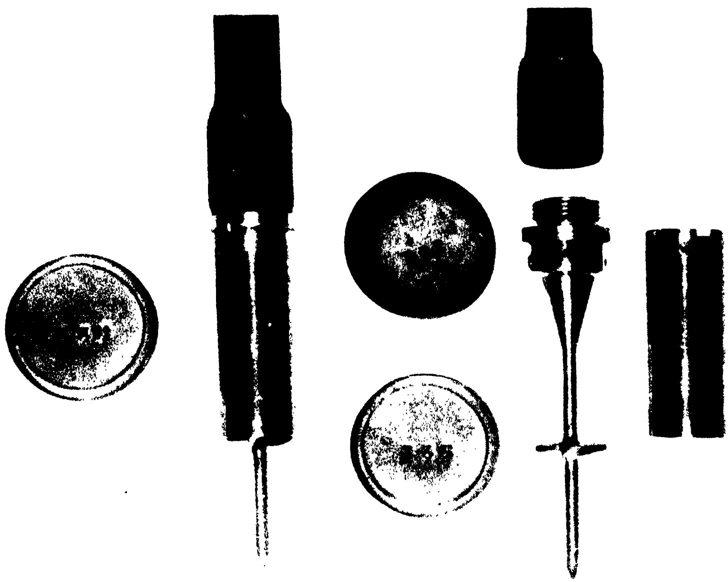
The method used by the writer for obtaining undisturbed samples of soil from shallow depths up to, say, 8 to 10 feet, is to excavate a trial pit about 6 feet by 6 feet in plan, leaving a central portion of undisturbed soil about a foot square in plan in the centre of the hole. This central portion of soil is trimmed off to a cylindrical shape, usually some 6 inches in diameter, and, say, a foot high, and is brushed over with repeated applications of hot paraffin wax around its top and sides. The sample is sawn off gently at its base, which is then treated with paraffin wax, care being taken to apply all the wax as quickly as possible in order to avoid loss of moisture from the soil sample. The latter is packed in a stout wooden box filled with shavings or cotton wool and sent to the laboratory. If necessary, further trimming of the sample can then be carried out. This method is, of course, only suited to quite shallow depths.

A. E. C. Longsdon[19] gives details of an improved form of sampler designed to obtain undisturbed samples of clay soils. It consists of an 18 inch length of B.S. 4 inch diameter water barrel, fitted with a detachable cutting shoe and driving head, the latter having a non-return valve to permit the egress of air and water from the boring during driving of the tool. Provision is made for the operation of a sample extruder. A. H. D. Markwick[20] also gives details of a core-cutter for obtaining an undisturbed soil sample, this sampler being shown in Figs. 56*a* and *b*. It should be noted that special care is necessary in the case of so-called " undisturbed " samples in order that no loss of moisture occurs between the time of taking the sample and its testing in the laboratory. In most cases such samples are enclosed in paraffin wax immediately after extraction and kept sealed until they are tested.



(a) (b) (c)

Fig. 54. Typical Soil Augers.
(a) 5 in. Post Hole Auger. (b) 4 in. Post Hole Auger. (c) Gravel Auger.
(*Reproduced by permission of the Controller of H.M. Stationery Office.*)



(a) (b)

Fig. 56.—(a) Sampling tool attached to extension rod and ointment tin closed.
(b) Sampling tool dismantled to show construction and ointment tin open.
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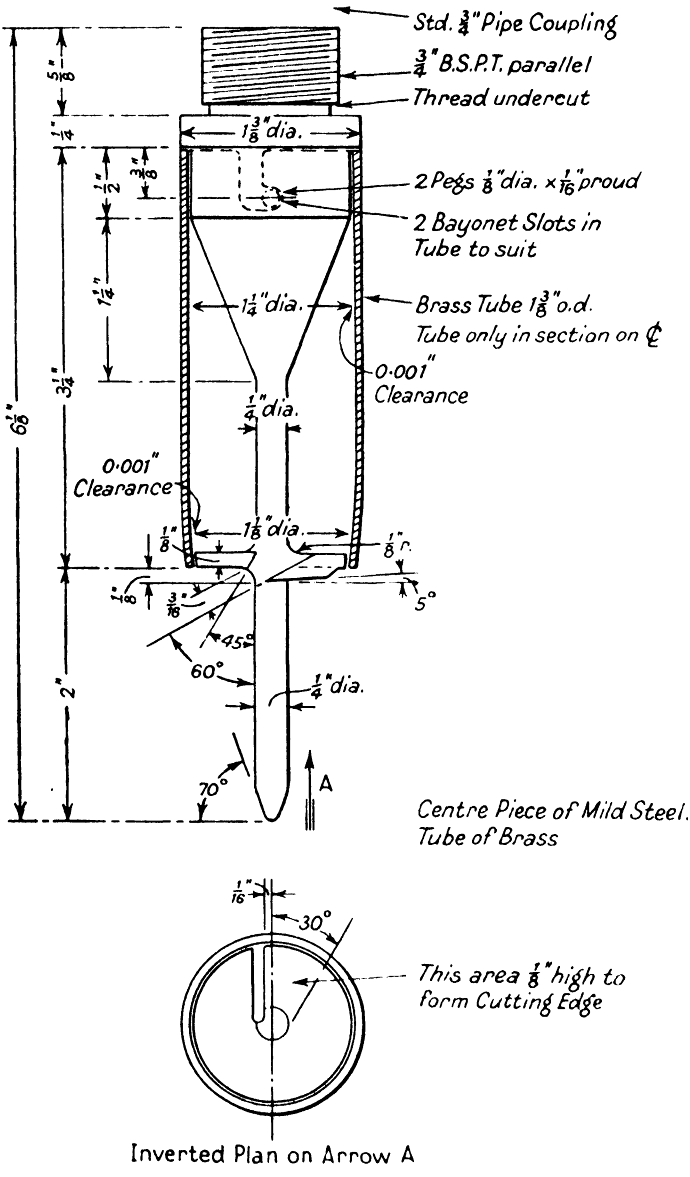


Fig. 55.—Details of 1 in. sampling tool used by Road Research Laboratory.

TABLE 7.—LIST OF EQUIPMENT REQUIRED FOR SOIL SAMPLING AND SOIL SURVEYS.

For Setting Out Positions of Bore-holes.

1 theodolite or tacheometer
1 quick-setting level
1 levelling staff
1 level book
1 100 feet steel tape or 1 100 feet chain
1 100 feet cloth tape
36 surveying arrows with red cloth attached as markers
Supply of 2 inch square wooden pegs, painted white at the tops
1 sledge hammer (10 pounds weight)
6 ranging rods
1 axe.

For Making Bore-holes.

Supply of boring records and pads (see Table 5)
Two or three 4 inch or 5 inch post-hole augers (see Fig. 54*b*)
1 gravel auger
1 twist auger (A.A.S.H.O. type)
1 crowbar capable of being coupled up with the auger extensions
6 tommy bars (for fitting extensions to augers)
Supply of sample tins numbered and greased
3 spatulas
2 spades for clayey ground
2 shovels for sandy ground
2 picks.

General.

1 10 cwt. light motor van
Supply of stout wooden boxes about 15 inches cube to carry tins
Sacks
Pegs
String and twine
Grease for tins
Bottle of hydrochloric acid (dilute) for identifying limestones
Packing material
Pencils
Notebooks
Squared paper for plotting soil profiles in the field.
Set squares
Scales.

Sample Tins. In the case of ordinary post-hole auger borings, air-tight tins $2\frac{1}{2}$ inches high by $3\frac{1}{2}$ inches in diameter are suitable for sampling purposes ; if undisturbed samples are required, large containers varying in size with the diameter of the drilling equipment are used. In the case of the small sampling tool figured in Fig. 56, cadmium-plated tins $2\frac{1}{4}$ inch diameter and 1 inch high are used ; such a tin does not rust. In most cases three to five samples are taken at the base of small borings so that an average of several determinations can be made, usually of moisture content determinations. For other soil tests, larger samples are required.

Sample Bags. In many countries, sample bags have been used for conveying material for test from the field to the laboratory ; the bags used should be capable of holding between 50 and 100 lbs. of loose material ; they should be of stout, closely woven canvas, and numbered clearly on the outside. They are particularly suited for use in tropical or semi-arid countries such as Central and Southern Africa ; they are easily transported and not easily damaged as are sample tins. They suffer, however, from the disadvantage that unless made from very tightly woven material, loss of fines may occur between the field and the laboratory; it is also not possible to determine original moisture contents of the materials which they contain. Lastly, disintegration of the soil by internal friction of the particles one against another during transit is very likely to occur, especially in the types of soil found in the regions mentioned above.

The Selection of Samples for Testing.

The samples are set out in rows in the engineer's office, after which they are placed in a number of groups by visual inspection. From each of these groups a few samples representing the extremes and means of grading, organic content and consistence are selected for test, with perhaps a rapid preliminary sedimentation test carried out with water in a test-tube. The following tests are carried out on the selected samples, if of the loose type :—

1. Moisture content.
2. Mechanical analysis.
3. Liquid limit.
4. Plastic limit.
5. Volume change.
6. Soil density.

All these tests are described in Chapter 5. In the case of " undisturbed " samples, shear and consolidation tests are carried out.

The A.A.H.S.O. T 86-42 Standard Method of Surveying and Sampling Soils for Highway Subgrades is as follows :—

EQUIPMENT.

(*a*) One 3 ft. soil auger with 1½ inch diameter drill end and three 3 ft. extensions.
(*b*) Two small pipe wrenches.
(*c*) One light pick.
(*d*) One shovel.
(*e*) Supply of sample bags and twine.
(*f*) Supply of tie-on tags.
(*g*) Engineer's level.
(*h*) Hand level.
(*i*) One 12 ft. levelling rod, three-section.
(*j*) One 100 ft. steel tape.
(*k*) One 12 inch by 15 inch strip of stiff cardboard.
(*l*) One roll of 20 inch cross-section paper, 10 divisions to the inch each way.
(*m*) Notebooks.
(*n*) Supply of survey stakes.
(*o*) Camera and films.
(*p*) Marking crayons.

The soil should be examined at intervals close enough to determine the soil type and by borings deep enough to penetrate the more or less non-uniform layers of soil.

Examination of Soil Section.

The following points should be studied :—

Texture. The fineness or coarseness and thickness of each layer to a depth of at least 5 or 6 feet should be described.

Colour. The successive layers differing in colour and the thickness of each layer should be described.

Structure. Structure is defined as the kind and size of soil aggregation. Special note should be made of horizons with the following structures :—

(a) Fine granular (granules about the size of bird-shot or smaller).

(b) Coarse granular (granules ranging up to ½ inch or more in diameter and usually more irregular in shape than the granules making up the fine granular structure).

(c) Layered or platy, in which the material splits into thin plates (not to be confused with stratification).

(d) Buckshot structure, in which the soil on drying breaks up into angular fragments (found to characterise heavy clays usually having a high proportion of lime).

(e) Single grain structure, in which the material is like flour or sand with no aggregation of particles.

Consistence. Successive layers or horizons differing in consistence (stickiness, friability, plasticity) with their thicknesses should be noted and described.

Compactness. The relative compactness of the several layers should be found, as measured by the degree of resistance to penetration of a pointed instrument.

Cementation. The colour and nature of any cementing material, if determinable, should be given.

Chemical Composition. Notes should be taken of any peculiarities such as abundance of organic matter, salts, iron oxides, etc.

Selection of Samples. A 5 lb. sample of soil from each layer should be obtained with pick and shovel from exposed back slopes or from test pits dug at locations indicated by the borings. Each sample should be placed in a canvas bag, tied securely, properly labelled and sent to the laboratory for examination. A sufficient number of samples should be taken to determine the range in test results for what appears to be the same layer.

The author does not recommend the use of canvas bags for the conveyance of soil samples. Such a method of conveyance may involve the loss of some of the finer portions of the sample, and stout tins are suggested as being preferable for this purpose. (See also page 89.)

Presentation of Data.

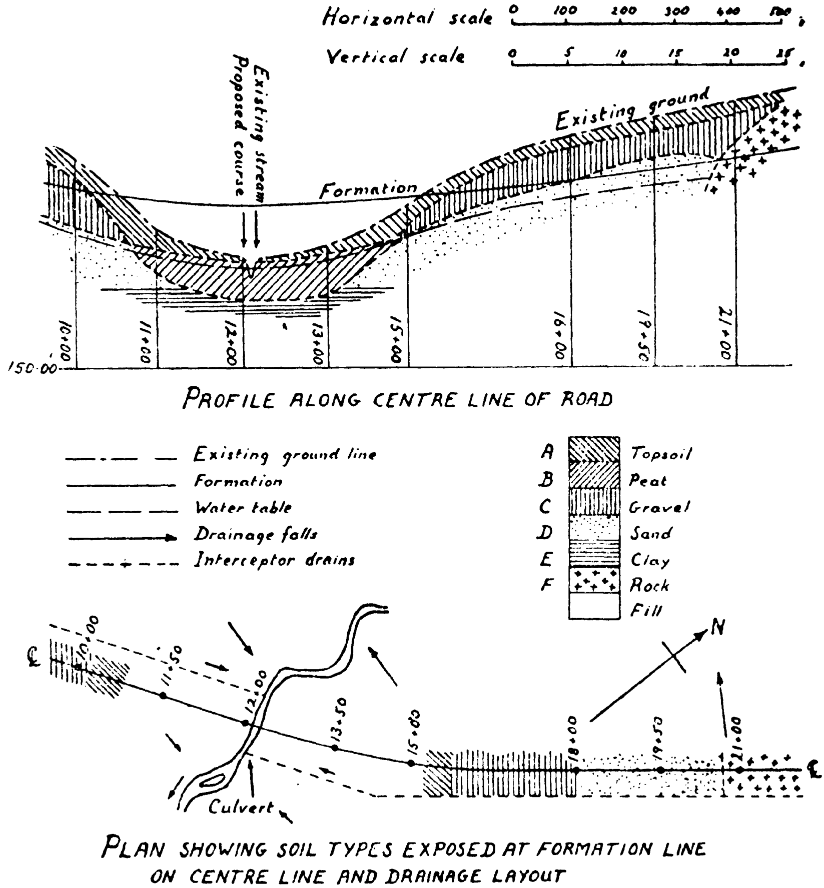
The A.S.T.M. Manual on Presentation of Data, sponsored by Committee E.1 on Methods of Testing, Reprint, April 1945, gives valuable information on this subject, and is based on Theory of Probability Methods. It includes :—

Supplement A. Presenting + Limits of Uncertainty of an Observed Average.

Supplement B. Control Chart Method of Analysis and Presentation of Data.

Special Investigations.

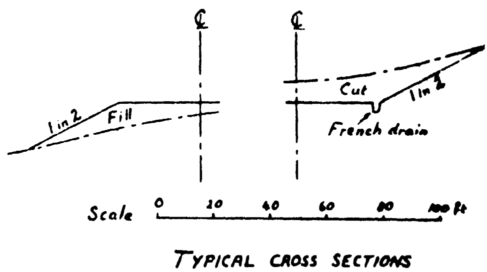
These may deal with problems relating to (*a*) settlement of subgrades, (*b*) stability of foundations, (*c*) compaction of subgrades and base courses. Settlement problems involve the taking of undisturbed samples and requires the use of a fully equipped soil mechanics laboratory. The same remark applies to stability problems involving



Soil type	C	D	E
Stone (>2 mm) %	60	4.1	0
Sand (2.0–0.06 mm) %	39	55	41
Silt (0.06–0.002 mm) %	1	4	26
Clay (<0.002 mm) %	0	0	33
Liquid limit	NP	NP	45
Plastic limit	NP	NP	20
Plasticity index	NP	NP	25

NP = Non Plastic

TESTS ON CHIEF TYPES OF SOIL



Notes:– All topsoil (Type A) is to be preserved and used for soiling at the completion of the work.
All peaty soil (Type B) is to be cut out and replaced by approved filling.
All filling to be spread and rolled in layers not exceeding 9 in.
Sand to be spread 2 in. thick as a subbase.

Fig. 57.—Typical soil survey drawing showing essential requirements for contract drawings.

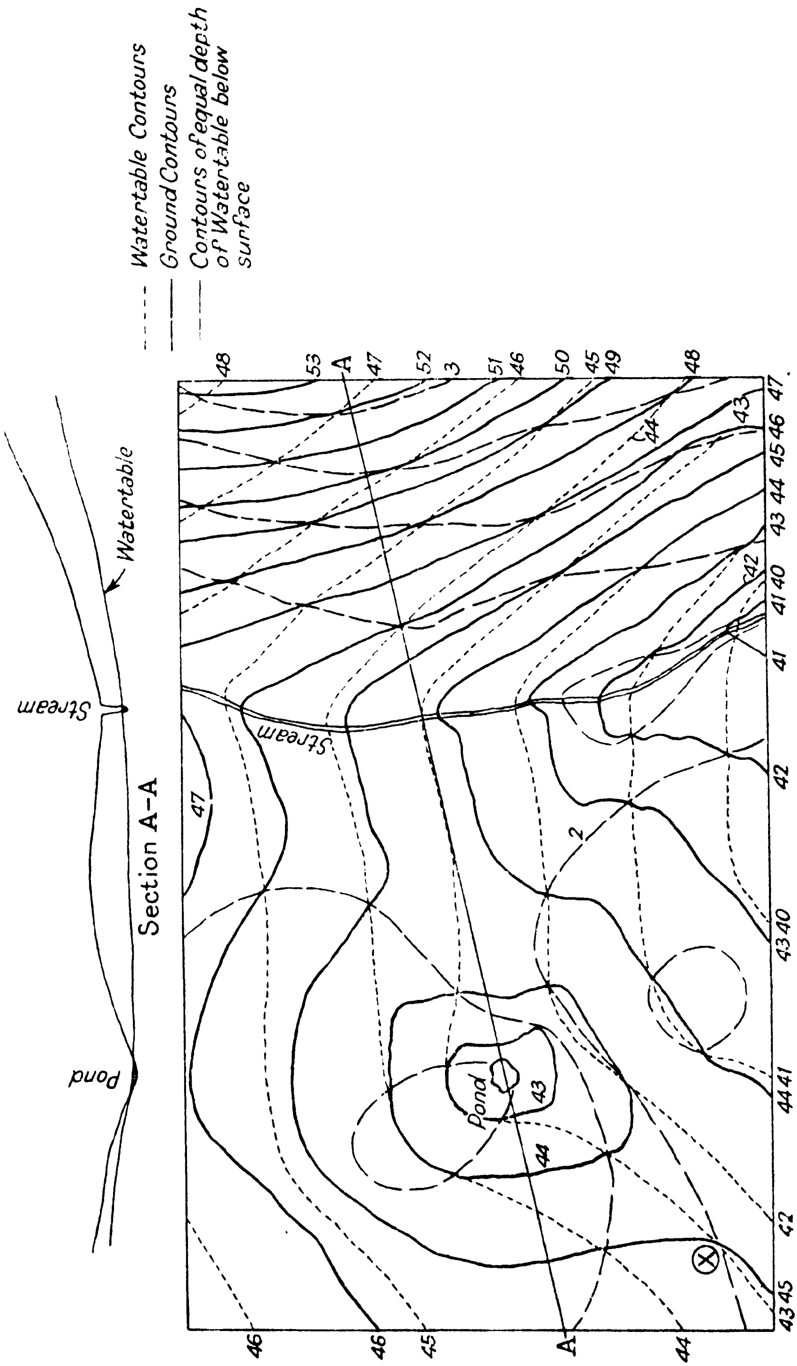


Fig. 58.—Plan showing contours of equal depth of water table below the surface.

sandy and gravelly soils, although this latter case is considerably simplified if the soil is purely cohesive and its shear strength determined in the field by means of direct compression tests carried out *in situ* (see Chapter 5). Compaction problems can to a large extent be worked out in the field (see Chapter 7).

Soil Profile Sections. Drawings of this type are illustrated in Fig. 57.

Ground Water Investigations.

Water-table contours are plotted as shown in Fig. 58. The plan of the borings is first plotted on the plan of the area with the reduced level of the water-table shown beside each boring site. The water-table contours can then be sketched in by connecting points possessing the same reduced level, or by ordinary interpolation methods.

To draw contours of equal depth of ground water below ground level, the ground water contours are superimposed on the ground level contours (see Fig. 58). From the intersections of the two families of curves, points are selected at which the differences between ground level and water-table level are constant. Thus at point X, this difference is 2 feet, and this point is joined to Y, where the difference is also 2 feet.

It should be noted that water-table levels vary with the time of year.

Seismographic and Electrical Methods.

Seismographic and electrical methods of investigation are now being applied to soil survey problems in America. In the former method, the seismograph used consists of a portable control unit, detonator and three detectors. The detonator and detectors are set up in a straight line at measured distances apart and are connected to the control unit. A small explosive charge is set off in the detonator and simultaneously a tuning fork is struck at the control unit. An image of the sound waves caused by the tuning fork is projected on to a strip of motion picture film in such a way that it marks a uniform series of time intervals each of one-thousandth of a second duration. The initial sound wave caused by the explosion in the detonator travels down through the soil to any hard rock which may underly it, is carried horizontally over the hard rock, and is deflected upwards to actuate sensitive recording instruments in the detectors. These impulses are transmitted to the control unit from the detectors by electrical means and are there recorded as a wavy line on the moving film strip. The strip is developed and passed out of the control unit, when the relationship of the three sound tracks can be read off against the constant time interval pattern set up by the tuning fork. The readings obtained are recorded and the apparatus moved to another spot at which the whole process is repeated. After a suitable number of shots have been taken, the results are plotted on a map and a section of the strata prepared. Satisfactory results up to 200 feet in depth are claimed, the method being said to be applicable to conditions in which several intervening strata of different character exist. The method is, however, best adapted to the mapping of hard rock.

The electrical resistivity apparatus is used chiefly for mapping the location, type and depth of clays, silts and gravels, and can also be used for depths of up to 200 feet. The principle underlying the method is the utilisation of the different electrical conductivity of different types of soil ; changes in potential between four electrode rods driven into the surface at fixed distances apart on a straight line are measured, a current being applied to the outermost rods and readings taken of the ampere drop between the outer and inner rods.

CHAPTER FIVE.

THE TESTING OF SOILS.

Soil testing has developed greatly in recent years, but is still very far from finality. As followed at present, it falls into two categories: (*a*) the testing of loose or disturbed soils, (*b*) the testing of so-called " undisturbed " soils, or soils which for our purpose can be regarded as substantially undisturbed.

The testing of loose or disturbed soils has to a large extent become standardised as the result of the work of many research workers, culminating in the issue of standard methods for such tests by the American Society for Testing Materials and the American Association of State Highway Officials (A.S.T.M. and A.A.S.H.O. respectively); that of undisturbed soils has also developed, but to nothing like the same extent as for loose soils; thus there is still no standard method for finding the resistance to shear cohesion or the angle of internal friction of soils.

This chapter will deal firstly with the more or less standardised methods of test normally applied to loose soils (often known as " indicator tests ") and will conclude with a consideration of the methods commonly applied to the testing of undisturbed soils, including soils *in situ*. Comments on various aspects of the test will be given where-ever possible, not only in this chapter, but also in Chapter 8.

It should be emphasised at the outset that it is a relatively simple matter to carry out tests on soils, but that the interpretation of the significance of the results of such tests is by no means a simple matter, but one needing a considerable amount of experience.

Field Identification of Soils. (See also page 83, Chapter 4.)

Most coarse-grained soils are easy to identify by inspection. Clean gravel is readily identified by the size of its constituent particles; coarse and medium sands are also readily identifiable by visual inspection, but very fine sands are sometimes mistaken for silts. It is better in such cases to dry the sample prior to inspection ; dried samples of fine sands show no cohesion, and the grain-size becomes more obvious in the dried state.

Silt and clay are best seen by shaking up the soil in water in a glass jar and allowing it to settle, when the coarse material settles at the bottom of the jar. Silts feel harsher to the touch than clays ; clays have a characteristically greasy feel when dry, and their shrinkage on drying is usually greater than that of silts.

Peats and peaty soils are easily detected by their blackish to dark chocolate colour and the presence of fibrous organic material. Organic soils can be identified by rapid heating, when a peculiar odour is given off. It should be noted that all field identifications are essentially preliminary only, and should be checked up by laboratory examination.

Standard Method of Preparing Soil Samples for Mechanical Analysis and Determination of Subgrade Soil Constants. A.S.T.M. D421-39 and A.A.S.H.O. T87-42.

The A.A.S.H.O. method is the later of the two available standard methods and is accordingly described here ; both methods refer only to disturbed soil samples.

The apparatus required includes a balance sensitive to 0·1 gram, a mortar and rubber-covered pestle, Nos. 4, 10 and 40 mesh A.S.T.M. sieves, and a riffler for quartering the samples.

Sample sizes are as follows :—

For *mechanical analysis*, enough soil passing a No. 10 mesh sieve to give 115 grams of sandy or 65 grams of silty or clay soils.

For *physical tests*, a total of 300 grams of soil, allocated as follows :—

Test.	*Weight (grams).*
Liquid limit	100
Plastic limit	15
Centrifuge moisture equivalent	10
Field moisture equivalent	50
Volumetric shrinkage	30
Check tests	95

For the *standard compaction test*, 3,000 grams (about 6·1 lb.) of soil passing the No. 4 mesh sieve is required out of a total sample of about 4,000 to 5,000 grams. The sample is prepared as follows :—

The soil sample as received from the field is dried thoroughly in air and the aggregations broken up in a mortar, using a rubber-covered pestle, reduction of individual grains being avoided as far as possible. The sample is then reduced to the appropriate size by the method of quartering.

The soil thus obtained is weighed and the weight recorded as the weight of the total sample uncorrected for hygroscopic moisture. The test sample is separated into two portions by means of a No. 10 mesh sieve. The fraction retained on this sieve is ground in a mortar until the aggregations of soil particles are broken up into separate grains. The ground soil is split into two fractions by means of the No. 10 mesh sieve. The fraction retained on this sieve is set aside for use in the mechanical analysis.

Test Sample for Mechanical Analysis. The fractions passing the No. 10 mesh sieve in both the sieving operations described above are thoroughly mixed, and a sample of about 115 grams of sandy or 65 grams of silty or clay soils obtained by quartering, and set aside for mechanical analysis.

Test Sample for Subgrade Soil Constants. The remaining portion of the material passing the No. 10 mesh sieve is separated into two parts on a No. 40 mesh sieve. The fraction retained on this sieve is discarded, and that passing it thoroughly mixed and used for determination of the soil constants.

Test Sample for Compaction Test. The portion of air-dried soil allocated for this test is sieved on a No. 4 mesh sieve. That portion retained on this sieve is ground with a rubber-covered pestle until all aggregations of soil particles are broken up into individual grains. The ground soil is then sieved on a No. 4 mesh sieve. The fraction retained on this sieve after the second sieving is discarded ; the fraction passing the No. 4 sieve in both operations is thoroughly mixed and used for the compaction test.

Mechanical Analysis of Soils. A.S.T.M. D-422 of 1939 ; A.A.S.H.O. T94-42.

The purpose of this test is the quantitative determination of the distribution of particle size, i.e., the soil grading.

The *apparatus required* includes a balance weighing to 0·1 gram, electric stirring apparatus similar to that used in milk bars, a brass dispersion cup, 3·75 inches internal diameter at its top, and 2·6 inches at its base, 7 inches high, fitted with internal rod baffles; a hydrometer graduated in grams of soil per litre of soil suspension (see Fig. 59*A*) or one with a special shape bulb, graduated in specific gravity range 0·995 to 1·050 and reading 1·00 at 67° F. (19·4° C.) (see Fig. 59*B*), graduated measuring cylinder 18 inches high and 2½ inches diameter

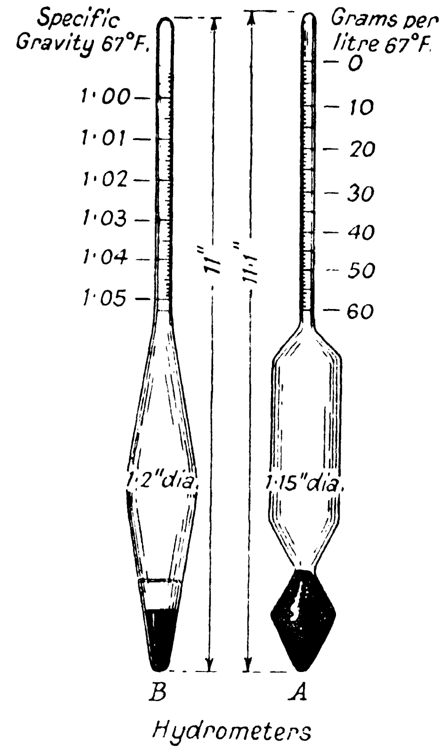


Fig. 59.—Showing types of hydrometers used in the mechanical analysis of soils.

graduated for a volume of 1,000 ml., a Fahrenheit thermometer reading to 1° F., a set of Nos. 20, 40, 60, 140 and 200 mesh A.S.T.M. sieves, a constant-temperature water bath, and a tall beaker of 400 ml. capacity.

The sample, weight 115 grams for sandy and 65 for silt or clay soils, passing a No. 10 sieve, is obtained by quartering (see Standard Method of Preparing Soil Samples for Mechanical Analysis and Determination of Subgrade Soil Constants, A.S.T.M. D421-39, A.A.S.H.O. T87-42.)

Hygroscopic Moisture is determined on a 10 gram portion of the soil; this soil portion is weighed, dried to constant weight at 110° C., reweighed, and the results recorded.

Hydrometer Test for Amount of Soil Passing 200 Mesh Sieve.

Dispersion of Soil Sample. The remainder of the sample is weighed and dispersed (i.e., broken up into its constituent particles) by either method *A* or method *B* below.

Method A, for soils having a plasticity index of 20 or less. The soil is placed in a tall beaker, and 200 ml. or more of distilled water added slowly and while being stirred constantly until the soil is thoroughly wetted. The mixture is allowed to stand for at least 18 hours, and is then washed into the special dispersion cup and distilled water added until the cup is within 2 inches of being filled. A deflocculating agent (20 ml. of a solution of sodium silicate crystals) is added, its density being: hydrometer A (Fig. 59), 36·5 at 67° F. (19·4° C.), hydrometer B (Fig. 59), 1·023 at the same temperature, Baumé hydrometer 3 deg. at 76° F. (24·4° C.). The contents of the cup are mixed by the special stirring apparatus for one minute. No stirring rate is laid down, and this may have an important effect on the result, as already shown by the author[22].

Method B, for soils having a plasticity index greater than 20. The soil is placed in a tall beaker and 100 ml. of 6 per cent. solution hydrogen peroxide added slowly and while being stirred constantly until the soil is thoroughly wetted. The beaker is then covered with a watch-glass and placed in an oven for 1 hour at 110° C. (The purpose of the peroxide is to assist dispersion of the soil). The beaker is removed from the oven, 100 ml. of distilled water added, and the mixture allowed to stand for at least 18 hours. The mixture is then washed into the dispersion cup and dispersed for one minute.

Hydrometer Test. After dispersion, the mixture is transferred to the glass jar and distilled water of the same temperature as that of the constant temperature bath added until the mixture has a volume of 1,000 ml. The jar and its contents are placed in the constant temperature bath and the suspension is stirred frequently with a glass rod to prevent settlement of the particles in suspension. When the suspension attains the temperature of the bath, the glass jar is removed and its contents thoroughly shaken for one minute, the palm of the hand being used as a stopper over the mouth of the jar. When shaking is finished, the time is recorded, the jar placed in the bath, and readings taken with the hydrometer at the end of 1 minute and 2 minutes, subsequent readings being taken at 5, 15, 30, 60, 250 and 1,440 minutes from the start.

The hydrometer is read at the top of the meniscus formed by the suspension around its stem ; if hydrometer A is used, it is read to the nearest 0·5 gram per litre ; if hydrometer B is used, it is read to the nearest 0·0005 specific gravity. Thermometer readings of the temperature of the bath are taken throughout the sedimentation period ;

after each reading after the first, the hydrometer is very gently taken out of the suspension in such a way as to disturb the liquid as little as possible, wiped clean and put aside. About 15 or 20 seconds before the time for a reading, it is again slowly and carefully placed in the suspension. The reading must not be taken until the hydrometer has come to rest.

Sieve Analysis.

After the final reading of the hydrometer, the suspension is washed on a 200 mesh sieve ; the fraction retained on this sieve is dried and then sieved on the Nos. 20, 40, 60, 140 and 200 mesh sieves.

Coarse Material.

The coarse material is determined from the data recorded during the preparation of the sample in accordance with the Standard Method of Preparing Soil Samples, A.S.T.M. D421, as follows :—

The weight of the oven-dried fraction retained on the No. 10 sieve is subtracted from the weight of the air-dried total test sample. The weight of the fraction passing this sieve is corrected for hygroscopic moisture. To this value is added the weight of the oven-dried fraction retained on the No. 10 sieve, in order to obtain the weight of the total test sample corrected for hygroscopic moisture. The fractions retained on both the No. 4 and the No. 10 mesh sieves are expressed as percentages of the corrected weight of the total test sample.

Percentage of Soil in Suspension.

For temperatures of the bath other than the hydrometer calibration temperature, the readings of hydrometer A are corrected by adding temperature-correction factors such as are shown graphically as ΔR in Fig. 60(*a*). A temperature-correction curve of this kind, or data corresponding to it, should be available for each hydrometer. If type B hydrometer be used, the readings should be corrected by means of the curve shown in Fig. 60(*b*). Such a correction-curve can be obtained by plotting the difference between the density of water at various temperatures against temperatures from 40° F. to 100° F. The percentage of the dispersed soil in suspension represented by different corrected hydrometer readings depends upon both the amount and the specific gravity of the soil dispersed. The percentage of dispersed soil remaining in suspension is calculated as follows :—

For hydrometer *A*,

$$P = \frac{R.a}{W} \times 100$$

For hydrometer *B*,

$$P = \frac{1606}{W}(R - 1)a \times 100$$

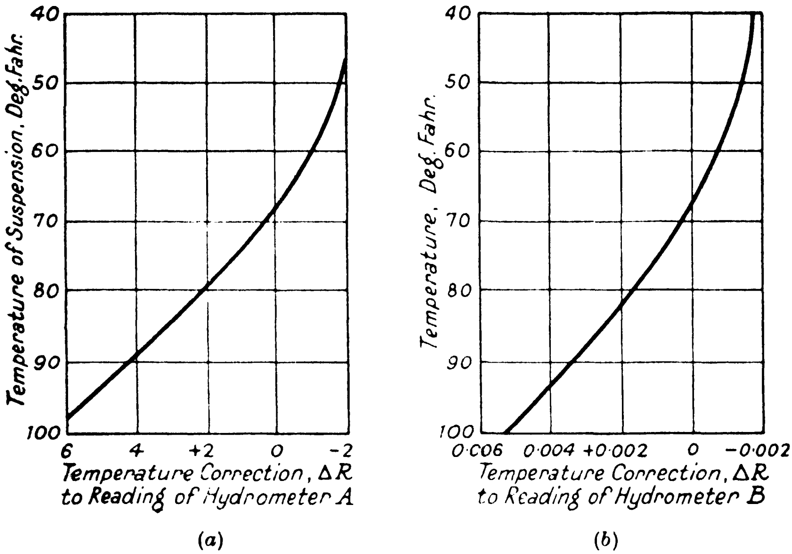


Fig. 60.—Temperature-correction curves for soil suspension hydrometer readings.

where P = percentage of originally dispersed soil remaining in suspension.

R = corrected hydrometer reading.

W = weight in grams of soil originally dispersed less hygroscopic moisture.

a = a constant depending on density of soil suspension.

For hydrometer A,

$$a = \frac{2{\cdot}65 - 0{\cdot}9984}{2{\cdot}65} \times \frac{G}{G - 0{\cdot}9984},$$ where G = specific gravity of soil particles, and the density of water at 67° F. = 0·9984.

For hydrometer B,

$$a = \frac{2{\cdot}65 - 1}{2{\cdot}65} \times \frac{G}{G - 1};$$ here the density of water is taken as approximately equal to 1. Corresponding values of a given by the above two equations for different values of specific gravity G of soil are :—

G	a
2·95	0·94
2·85	0·96
2·75	0·98
2·65	1·00
2·55	1·02
2·45	1·05
2·35	1·08

2·65 is a common value for the specific gravity of soil, so that in most cases a works out as equal to 1.

The maximum diameter of the soil particles in suspension, assuming that Stokes' Law applies, is given below :—

Time (minutes)	*Maximum grain diameter mm.*
1	0·078
2	0·055
5	0·035
15	0·020
30	0·014
60	0·010
250	0·005
1440	0·002

In the above table, it is assumed that L_1, the vertical distance through which the grains fall, is constant and equal to 32·5 cm., and that the coefficient of viscosity of the liquid through which they fall is equal to that of water at 67° F., i.e., 0·0102, and also that the specific gravity of the soil particles is 2·65. Figs. 61(*a*), 61(*b*), 62(*a*) and 62(*b*) show curves whence correction coefficients for elevations

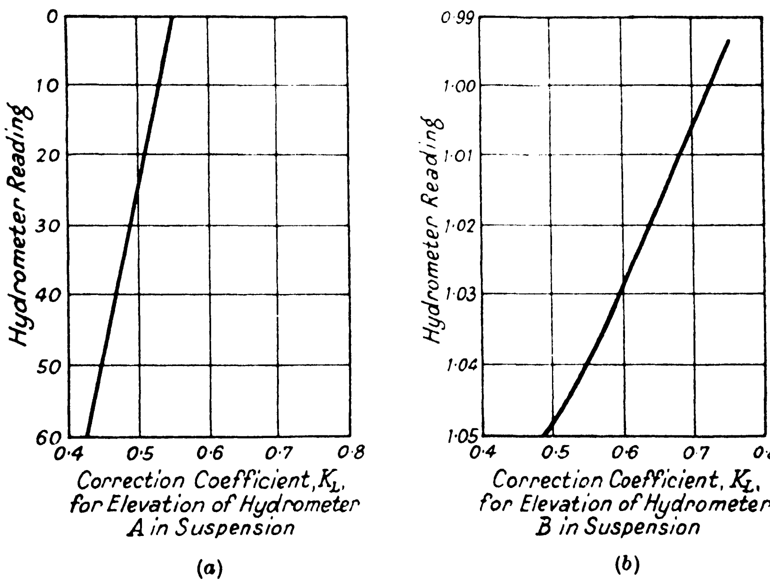
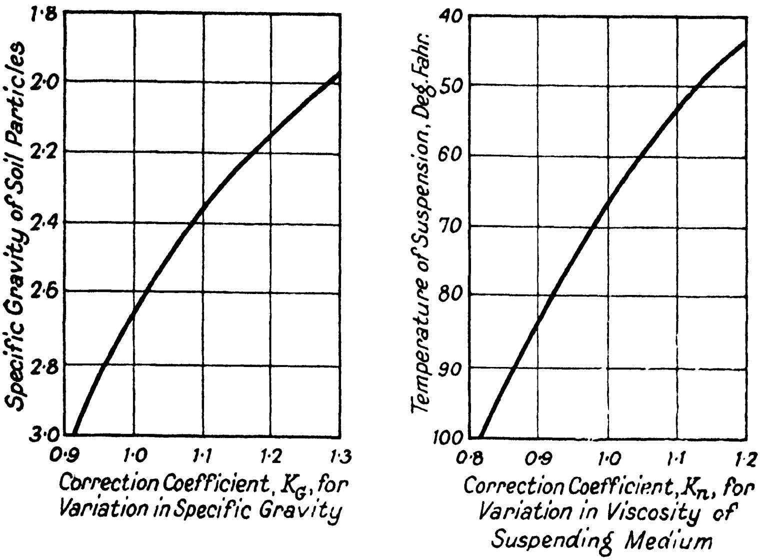


Fig. 61.—Correction coefficient curves for elevation of hydrometer.

of hydrometers in suspensions, variations in the viscosity of water and in the specific gravity of soils can be obtained.



(*a*) (*b*)

Fig. 62.—Correction coefficient for specific gravity and viscosity.

Plotting the Results. The percentages of grains of different diameters are plotted on semi-logarithmic paper to obtain a " soil-grain diameter accumulation curve " as shown in Fig. 63. The results are reported in percentages of :—

Particles larger than	2 mm.
Coarse sand	2 mm. to 0·25 mm.
Fine sand	0·25 mm. to 0·05 mm.
Silt	0·05 mm. to 0·005 mm.
Clay	0·005 mm. to 0·001 mm.
Colloids	below 0·001 mm.

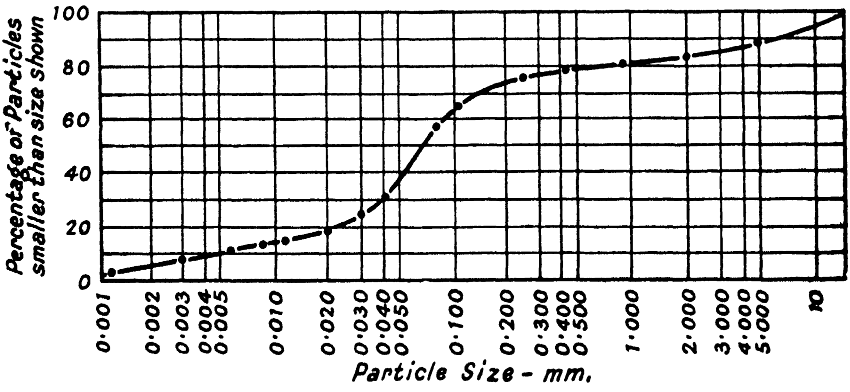


Fig. 63.—Grain size accumulation curve.

It should be noted that the sizes given above do not agree with those recommended by the Soils Laboratory of the Massachusetts Institute of Technology given on page 40.

The Field Measurement of the Bulk Density of Soil in situ.

The usual procedure adopted in finding the bulk density of soil *in situ* is to obtain and weigh a sample of the soil and to determine the volume of the space which it occupied by pouring in a measured quantity of loose sand into the space. The gross weight per cubic foot of soil extracted can be found directly, while the dry weight per cubic foot can be obtained by finding the moisture content of the extracted soil and allowing for the moisture present.

The apparatus used is a confectioner's glass jar, size 7 lbs., fitted with a metal conical head and pouring tap (see Fig. 64). The latter is joined to a conical spout to which is attached a brass rim. The glass jar contains dry sand of known bulk density.

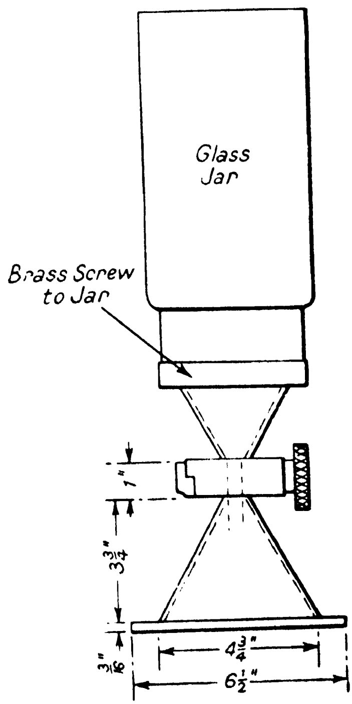


Fig. 64.—Sand bottle and standard pouring funnel for soil density measurement.

A flat area of the soil about a foot square is levelled off and a round hole about 4 inches in diameter and 4 inches deep is excavated in the soil by means of the tool shown in Fig. 65. The excavated soil is collected and weighed, and a representative sample is placed in an airtight tin for the determination of the moisture content. After having been weighed, the glass jar and pouring funnel are placed so that the pouring spout and base covers the hole in the soil. The tap is opened, and after a short time the sand fills the hole and no further movement of sand takes place in the jar. The tap is then turned off and the jar removed and re-weighed.

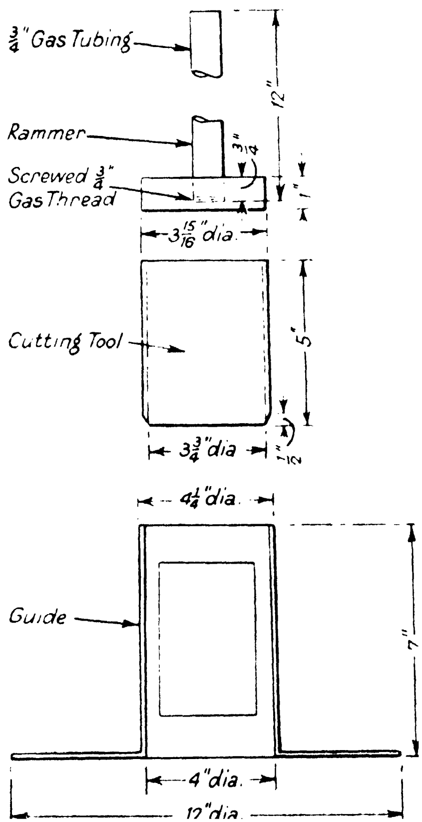


Fig. 65.—Core cutting tool for soil density measurement.

The difference in weight of the jar and attachment before and after the above operation gives the weight of the sand required to fill the hole and the conical spout. A correction for the volume of sand needed to fill the conical spout is obtained separately by inverting the apparatus on to a plane surface such as a sheet of glass and measuring the volume of sand contained in the conical head.

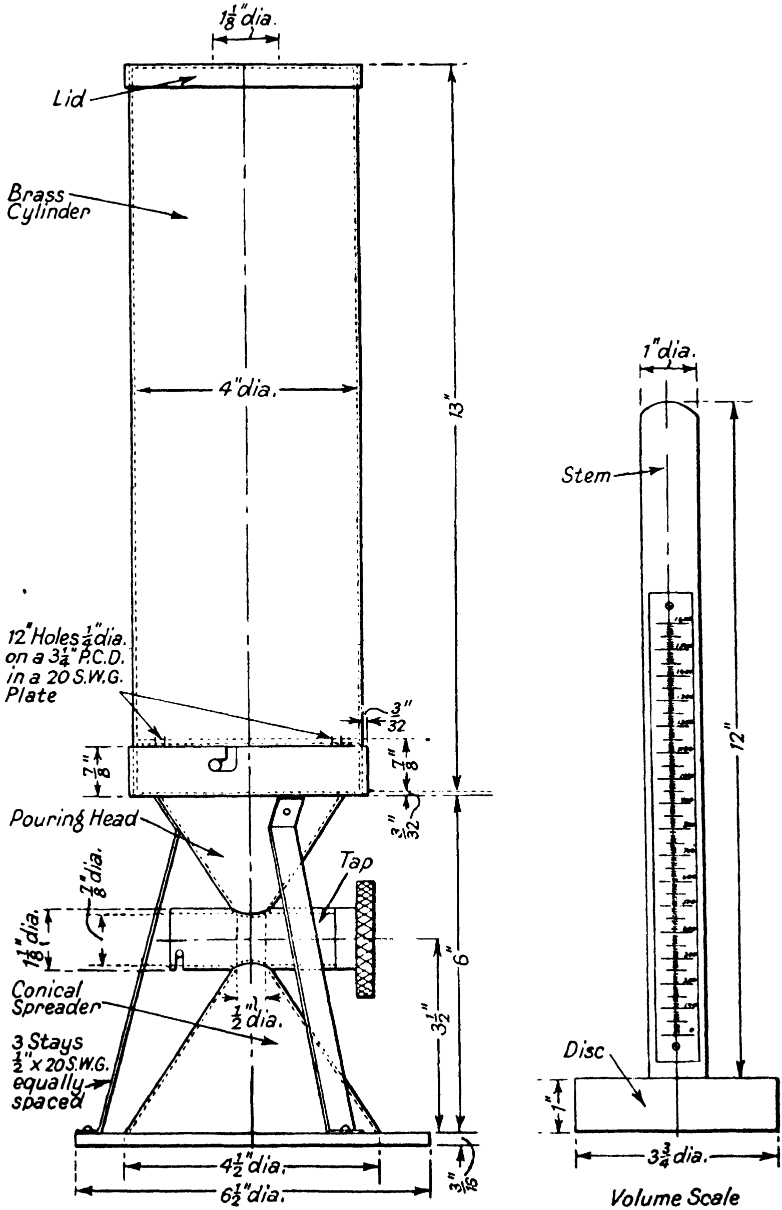


Fig. 66.—Showing modified sand-bottle apparatus developed by Road Research Laboratory for field determinations of soil density.

The density of the dry soil $D_s = \dfrac{100D_w}{100+w}$, where D_w = wet density of soil and w is the water content of the soil expressed as a percentage by weight of the dry soil.

The above method involves the weighing of the sand bottle before and after each density measurement, but a modified method used by the Road Research Laboratory obviates these weighings, although the soil removed from each hole has to be placed in a separate container and weighed. The apparatus used (see Fig. 66) consists of a brass cylinder 13 inches long and 4 inches internal diameter, with a bayonet-type attachment connecting it to a conical brass pouring head with a tap ; a volume scale, brass lid, straight-edge, supply of graded sand, cubical tin box side 7 inches or a cylindrical tin of the same volume with one spare lid having a 1 inch diameter hole in a corner or, in the case of the round tin, on the edge, 1 foot square tray (Fig. 67) with a central 4 inch diameter hole having upturned edges, and an excavating

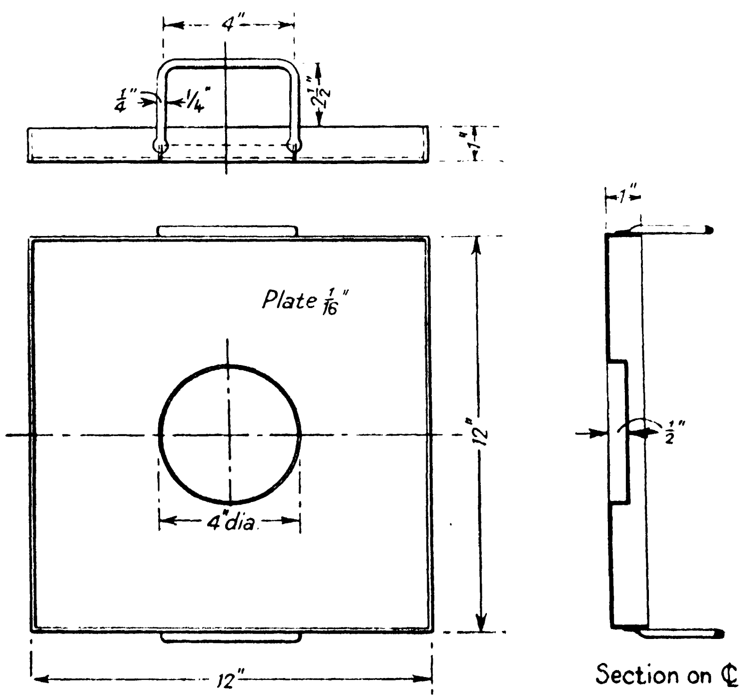


Fig. 67.—Showing steel tray used by Road Research Laboratory in field determinations of soil density.

tool. For calibration purposes a container of known volume with a rim is required.

The conical pouring head includes a conical spreader cone fitted with a brass rim on which the apparatus stands when in use. The brass volume scale is carried by a wooden stem 1 inch in diameter and 12 inches long which is fixed to a wooden disc $\frac{1}{8}$ inch smaller than the inner diameter of the cylinder. This scale is calibrated, one division being equal to 10 c.c. The sand used is dry sand passing No. 14 B.S. sieve and retained on No. 25 B.S. sieve. The following precautions should be observed :—

(1) The sand must be air-dry, and the same sand must be used for both calibration and field measurements.

(2) The rate at which sand is poured into the cylinder should always be the same ; this can be done by pouring the sand from a box having a 1 inch hole in the top.

The calibration of the apparatus is carried out as follows :—

A strip of paper 10 inches long by 1 inch wide is fixed vertically to the stem of the vertical scale. The brass cylinder and pouring funnel are placed in a firm flat surface such as a glass plate. The sand is poured from a container through a hole in its lid into the cylinder and struck off with the straight-edge. The volume scale with the lid inverted on to the wooden disc is placed on top of the sand. The tap is turned so that the sand from the cylinder fills the conical spreader. When the latter is full of sand, no further downward movement of the stem can occur, and a mark is made on the paper, level with the lid, which will now be resting on top of the cylinder. The sand is removed from the cylinder, and the whole process repeated several times in order to fix the mean position of the zero of the scale. The apparatus is then placed on top of the container, the volume of which is known, and the whole process is repeated, this giving a second mark of known volume. The paper is removed from the stem, and the distance between the upper and lower marks graduated into equal divisions, each division representing 10 c.c. The scale should cover the range 0 c.c. to 1500 c.c., equivalent to the volume of a hole $4\frac{1}{2}$ inches in diameter and $5\frac{1}{2}$ inches deep. From this scale a similar brass scale can be made and screwed into position on the stem.

In the field, an area of the soil 1 foot square is levelled and the rectangular tray placed on it. A hole about 5 inches deep and 4 inches in diameter is excavated in the soil, and the soil from this hole is weighed and a representative sample placed in an airtight tin for moisture content determination. The tray is replaced by the brass cylinder and pouring head, the base of which must cover the hole in the soil. The sand is poured into the cylinder, struck off with the straight edge, and the volume scale placed on top. The tap is turned on so that the sand fills the hole and the conical spreader. When the scale has come to rest, the volume of the hole is read off from it. The time of test by this method varies from 5 to 15 minutes according to soil conditions. Errors in volume are not likely to exceed about 1·3 per cent. as compared with about 1·2 per cent. by the usual method.

Standard Compaction Test on Soils. A.A.S.H.O. T99-38, A.S.T.M. D698-42T. Also known as the Proctor Test.

Scope of Test. The object of this test is to find the moisture content at which maximum density is obtained when the soil is compacted in a standard manner. The degree of compaction obtainable in the test is designed to be comparable with thorough compaction under field conditions as ordinarily encountered. The test also gives the value of this maximum density and its relation to the dry density of the soil in lb. per cubic foot after subtracting the weight of water present.

For practical reasons, the standard compaction test on soils is done on material passing the $\frac{3}{16}$ inch mesh sieve. If larger material is present and the test is done on the fines, as is usually the case, it is then necessary to compute the test values for the material from the data obtained from the fines, and this is possible provided that at least sufficient fines are present to fill the voids in the coarse material.

Apparatus. A cylindrical brass mould (see Fig. 68) capacity 1/30 cubic foot, 4 inches in diameter and 4·6 inches high, fitted with detach-

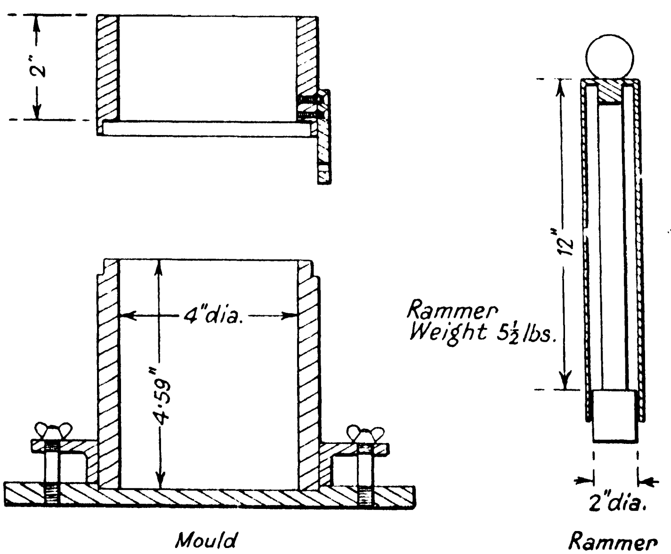


Fig. 68.—Standard compact apparatus. (A.A.S.H.O.)

able collar and base ; a standard rammer, weight 5·5 lb., with sleeve; aluminium containers, analytical balance weighing to 0·1 gram, drying oven, $\frac{3}{16}$ inch B.S. sieve, steel straight-edge.

Procedure. 3,000 grams of the soil is oven dried at 105° C., passed through a $\frac{3}{16}$ inch mesh B.S. sieve and thoroughly mixed in the dry state. Water is added in sufficient quantity to make the soil slightly plastic, after which it is again thoroughly mixed. When hand mixed, the material should be broken into small crumbs and thoroughly worked with the hands. The sample is compacted into the mould in three equal layers giving a total compacted depth of about 5 inches, each layer being compacted by 25 blows of the rammer dropping freely from a height of 12 inches. During compaction, the mould is supported on a rigid foundation. The collar is then removed, the top of the soil specimen is carefully trimmed to the top of the mould with a steel straight-edge, and the weight of the soil contained in the mould is found. After being extracted from the mould, a small sample of the soil is taken whence the moisture content is determined.

Calculations. The wet density of the soil in the mould can be obtained in lbs. per cubic foot from $D_w = 30W$, where D_w = wet density of soil and W = weight of wet soil in the trimmed soil. If w = moisture content of sample (per cent. of the dry soil), then the dry soil density is

$$D_s = \frac{100D_w}{100+w} \text{ in lb. per cubic foot.}$$

When a number of determinations has been made and the dry soil density has been plotted against moisture content, it will be found that the curve connecting these two quantities has a peak corresponding to maximum density. This is referred to as the maximum dry soil density, and the corresponding moisture content as the " optimum moisture content " for this method of compaction.

Adjustment of Test Values for Coarse Material.

Let the percentage by weight of material retained on the $\frac{3}{16}$ inch mesh sieve be x, and let the density of the stone be w lbs. per cubic foot. The percentage of coarse material can readily be expressed as a percentage y of the fine material where $y = \dfrac{100x}{100-x}$. Then the volume of coarse material corresponding to 100 lb. of fine dry soil is $\dfrac{y}{w}$ cubic foot, the volume of fines $= \dfrac{100}{D_s}$ and the total volume $= \left(\dfrac{y}{w} + \dfrac{100}{D_s}\right)$

cubic feet, the total weight of this material being $\dfrac{100}{\left(1+\dfrac{y}{100}\right)}$ lb. Hence

the adjusted density is $\dfrac{100\left(1+\dfrac{y}{100}\right)}{\left(\dfrac{y}{w}+\dfrac{100}{D_s}\right)} = \dfrac{D_s\left(1+\dfrac{y}{100}\right)}{\left(1+\dfrac{y}{100w}D_s\right)}$ lb. per cu. ft.

The adjusted optimum moisture content of the total soil, where x = percentage of coarse material, is $\dfrac{M}{\left(1+\dfrac{y}{100}\right)}$ per cent., where M is the optimum moisture content for the fines. This calculation assumes that the voids in the coarse material are completely filled with fines at maximum density, and this assumption is valid up to about 40 per cent. of coarse material, after which the density falls off unless the moisture content of the soil fines is increased to compensate for the additional resistance to compaction caused by the interlocking of the coarse aggregate.

To obviate the trouble of the above calculation for coarse material, a modified Proctor test uses a 6 inch diameter mould and between 55 and 63 blows with a 10 lb. hammer falling vertically through 18 inches ; five layers of soil are used.

The standard compaction test of the A.A.S.H.O. detailed above is laid down as being restricted to soil passing the $\frac{3}{16}$ inch mesh sieve. If larger matter is present in the soil mass it is assumed to act merely as a displacer. Work done by the Road Research Laboratory has shown that the presence of coarse material in the sample hinders the compaction of the finer material, but that the resultant decrease in density is compensated almost exactly by the presence of stone, while the reproducibility of the test is not affected by the increase of the maximum size of stone from $\frac{3}{16}$ inch to $\frac{3}{4}$ inch.

W. H. Campen and J. R. Smith[25] have shown that the strength of soil mixtures at optimum moisture content and maximum density varies widely and in general is quite low, the strength increasing, however, as the bulk density increases. A method of obtaining equal strengths in all soil mixtures used for comparative test purposes is suggested as follows :—

Maximum density and optimum moisture content is determined on a given soil by three compactive efforts whose energies are in the ratio of about one to two to four. The strength of the soil mixture at these three maximum densities is found, and the strengths obtained are plotted against densities. From the resulting graphs the densities can be found for the soil which will give equal strength. The strength of the soil is taken as the load required to produce $\frac{1}{4}$ inch total deformation with a 2 inch diameter bearing block on the surface of a soil mass compacted into a mould 8 inches in diameter and 4 inches high ; this load is called the " Bearing Index " of the soil. The energy in foot-pounds per cubic inch of compacted material required to develop the desired strength may be determined by plotting energy against strength.

Besides the Proctor and Modified Proctor Compaction tests, another test, known as the Dietert test, has been used in U.S.A. The method of test is shown in Fig. 69. The mould used is smaller than the standard mould already described, the method of compaction being by mechani-

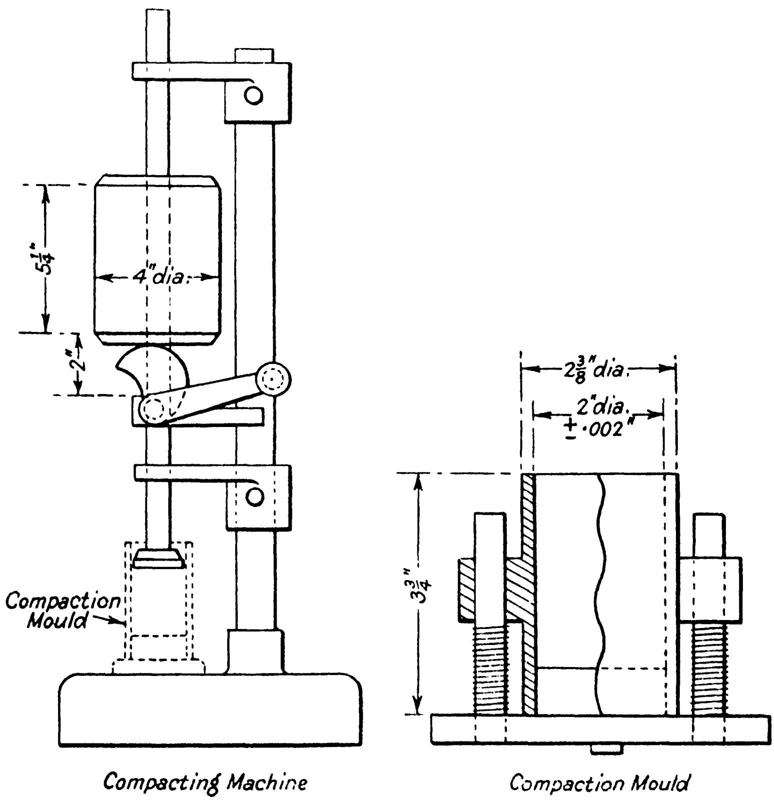


Fig. 69.—Dietert Compactor.

cal means, a revolving cam causing a weight to fall vertically on the entire surface of the soil. This test has not yet been adopted to a very large extent outside America.

The Road Research Laboratory has shown that the mixing of up to 70 per cent. of a single-sized stone aggregate with a silty clay soil produced only a slight reduction in the density of the soil mortar if the stone content is kept below 25 per cent.; beyond that figure, the dry soil density was considerably reduced. Compaction was considerably impeded by the addition of the coarse aggregate, but not by difference in stone size in the range $\frac{3}{8}$ inch to one inch. It was also found that the correction for coarse material applied in the standard compaction test procedure is liable to appreciable error when the stone content is high.

Methods of Determination of Water Content and Index Properties of Soil. A.S.T.M. D423-39 and D424-39, A.A.S.H.O. T89-42, T90-42.

Liquid and Plastic Limits. The *liquid limit* of a soil is defined as the water content (expressed as a percentage of the dry weight of the soil) at which two sections of a soil cake placed in the cup of the liquid limit device (see Figs. 71 and 72*a*) and separated by a groove, just touch

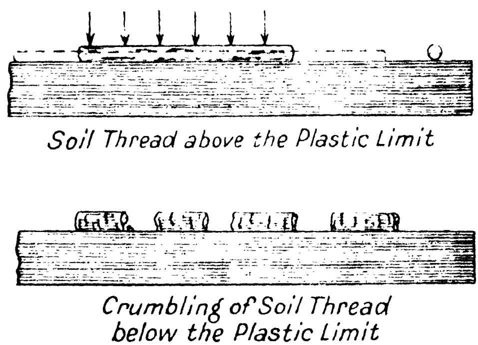


Fig. 70.—Phenomena occurring during plastic limit test.

over a length of $\frac{1}{2}$ inch when jarred exactly 25 times under standard conditions. This moisture content is obtained by plotting the curve of the water content of the various mixtures to an arithmetic scale and the corresponding number of blows of the liquid limit device to a logarithmic scale. This relation is found to be a straight line for any soil, so that the water content for 25 blows is easily found.

The *plastic limit* of a soil is defined as the water content (expressed as a percentage by weight of the dry soil) at which the soil will just crumble when rolled into a thread $\frac{1}{8}$ inch diameter (see Fig. 70).

The *plasticity index* of a soil is defined as the difference between the liquid limit and the plastic limit.

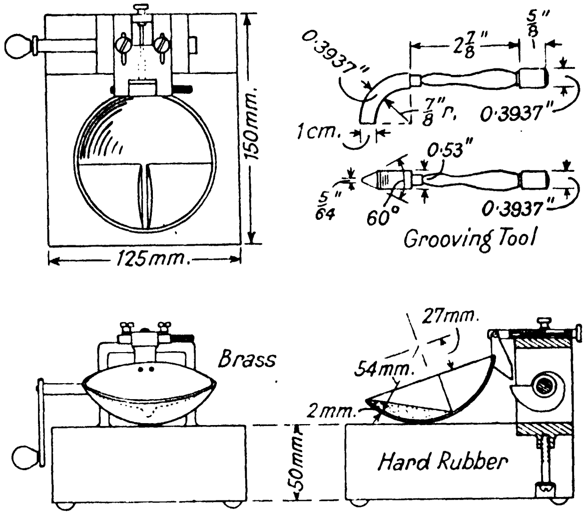


Fig. 71.—Liquid limit device.

The above tests apply only to soils having some degree of plasticity. The apparatus used for the tests includes a liquid limit device, grooving tool, glass plate, aluminium containers, burette, spatula, analytical balance weighing to 0.001 gram, drying oven and desiccator. The liquid limit device must be so adjusted that the height of fall of the cup is exactly 1 cm.

The sample used is the soil mortar passing a No. 36 B.S. sieve, the soil having been oven-dried at 105° C. prior to testing. Care should be taken to break down all soil lumps but not to break down individual soil grains. About 100 grams of the soil mortar is taken and mixed thoroughly with a spatula with water in a dish or on a glass plate to a suitable consistence, a portion of about 15 grams being set aside for the plastic limit test.

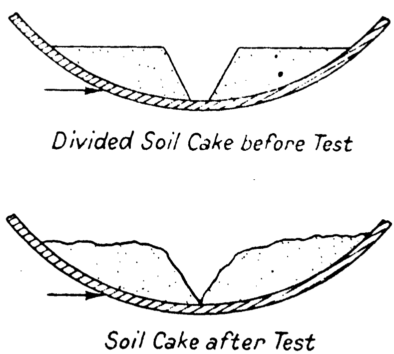


Fig. 72(*a*).—Phenomenon occurring during liquid limit test.

Liquid Limit Test. In dry atmospheric conditions, this test should be carried out as quickly as possible, especially when the soil is of a sandy nature. This is necessary in order to avoid undue loss of moisture by evaporation.

50 to 80 grams of the prepared sample are placed in the cup of the liquid limit device (see Fig. 71). It should be noted that the test is not sensitive to the amount of soil used if the quantity is enough to permit of the cutting of a full groove in the wet soil. The surface of the soil is smoothed off with the spatula and a groove cut in it with the grooving tool. In cutting the groove, the tool should be held perpendicular to the surface of the cup as it is drawn through the sample along a diametral line perpendicular to the axis of rotation of the cup (through centre of cam and point of contact of cup and base). The shoulders of the grooving tool should remove soil for a length of about 1½ inches.

The crank of the device is turned at the rate of about two turns per second until the bottom of the groove is closed for a length of ½ inch (see Fig. 72*a*). The operation of mixing, grooving and determining the number of blows required is repeated until three successive determinations show satisfactory agreement. About 5 to 10 grams of the sample in the vicinity of the groove is transferred to an aluminium container ; the procedure outlined above is repeated to give two points on the flow curve between 20 and 40 taps, and two between 10 and 20 taps. It is advisable to obtain the points corresponding to the greatest number of blows first, adding water by means of a burette to obtain the consistency corresponding to the smaller number of blows. Additional material from the prepared sample should be added to replace the material taken for samples, if a full groove cannot be obtained.

The container and soil are now weighed to 0·001 gram and the weight recorded. The container lid is removed, the lid and body of container being placed side by side in the drying oven at 105° C., and left there for 5 hours. The dried sample is removed from the oven, the lid replaced and the whole left in a desiccator to cool; after cooling, the container and dried sample are again weighed and the weight recorded.

The water content in per cent. dry weight is computed from the ratio of the weight of the water removed by drying to the weight of the dry sample. The flow curve is plotted on semi-logarithmic paper and the liquid limit read off as the water content corresponding to 25 blows of the liquid limit device.

Plastic Limit Test. A sample of about the size of a ½ inch cube is taken from the prepared sample used in the liquid limit test, and is rolled on a glass plate until the moisture content of the soil is reduced.

The soil is folded and again rolled until it assumes the shape of an $\frac{1}{8}$ inch diameter thread, finally crumbling. It is placed at once in an aluminium container, weighed and the weight recorded.

The sample is now dried with the container open in a drying oven at 105° C. for five hours, after which it is removed from the oven, cooled in a desiccator and re-weighed. The water content is calculated and equals the plastic limit, expressed as a percentage by weight of the oven-dried soil. A study of the factors involved in the measurement of the liquid limit of soil has been made by the Road Research Laboratory, as a result of which it is concluded that :—

1. A hand-driven apparatus used on a papier-maché pad gave results 5 per cent. higher than a similar apparatus placed on a substantial bench.

2. A mechanical apparatus gave results 6 per cent. lower than a hand apparatus, due, it is thought, to the extra rigidity afforded by the heavy iron base on which the former was mounted.

3. Identical results were obtained when a steel cup was used instead of a brass cup on the same apparatus.

4. Brass cups weighing 181 grams and 331 grams respectively, when used on the same apparatus, gave almost identical results.

5. When the wooden block of a hand apparatus was replaced by rubber which was softer than the wood, the test values were increased by 8 per cent.

6. The Casagrande or the standard American grooving tool gave identical results in the same apparatus.

7. Slight variations in the width of the grooving tool did not affect materially the results obtained.

8. The value of the liquid limit for any particular soil was not affected materially when the test was carried out at temperatures varying between 0° C. and 45° C.

Various workers collaborating with the author have found that the personal element affects the results of this test ; it was also found that extensive manipulation of clay soils prior to making the test has the effect of increasing the value of the liquid limit by as much as 10 per cent.; this is no doubt due to increase in the surface area of the soil particles by grinding down caused by such manipulation. In view of the foregoing. it may well be doubted whether it is worth while weighing the dried sample to 0·001 gram.

Centrifuge Moisture Equivalent of Soil. A.S.T.M. D425-39, A.A.S.H.O. T94-42.

The *centrifuge moisture equivalent* of a soil is the amount of water it retains after saturation and subjection to a force equal to 1,000 times the force of gravity for one hour.

The apparatus used includes a Gooch crucible 1½ inches high and 1 inch diameter at the top and ¾ inch at the base (see Fig. 72*b*) outside

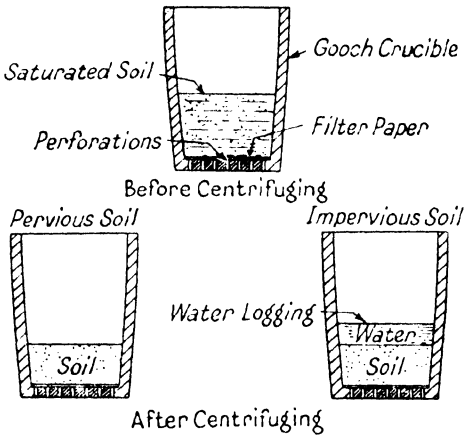


Fig. 72(*b*).—Phenomenon occurring during the centrifuge moisture equivalent test.

dimensions (the bottom of the crucible is perforated to allow of the egress of water during the test), a circular piece of filter paper just large enough to cover the inside base of the crucible, a Babcock trunnion cup fitted with a brass cap and with a suitable device for supporting the crucible ½ inch above the bottom of the cup so that water may leave the sample freely during the test, a centrifuge capable of exerting a force of 1,000 times gravity on the sample, and a balance sensitive to 0·1 gram.

A 5 gram sample of soil is taken from the thoroughly mixed portion of material passing the No. 40 mesh sieve which has been obtained in accordance with A.A.S.H.O. T87-42 Standard Method of Preparing Disturbed Samples. Tests are made in duplicate.

The sample is put in the Gooch crucible on a piece of wet filter paper just covering the bottom of the crucible. The latter is placed in distilled water and the sample allowed to take up moisture until completely saturated. It is then put in a humidifier for 12 hours to ensure uniform distribution of soil moisture throughout the soil mass. All free water on the surface on the sample is poured off and the crucible placed in the Babcock trunnion cup. The sample is centrifuged for an hour, and the crucible and its contents immediately weighed and then oven-dried to constant weight at 110° C. and re-weighed. If free water is observed on the top of the sample after centrifuging, the soil is said to have waterlogged. This water (see Fig. 72*b*) is not removed, but is weighed with the sample.

The centrifuge moisture equivalent of the soil

$$= \frac{(A - b) - (A_1 - b_1)}{A_1 - (c + b_1)} \times 100$$

where A = weight of crucible and contents after centrifuging
A_1 = ,, ,, ,, ,, ,, ,, drying
c = ,, ,, crucible
b = ,, ,, wet filter paper
b_1 = ,, ,, dry ,, ,,

The variation between the two values obtained in the duplicate tests should not exceed 1 per cent. for values up to 15, and 2 per cent. for values exceeding 15.

Field Moisture Equivalent of Soils. A.S.T.M. D426-39.

The field moisture equivalent of a soil is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance.

The apparatus used includes an evaporating dish about 4½ inches in diameter, a spatula with a blade about 3 inches long and ¾ inch wide, a burette, suitable containers such as matched watch glasses which will prevent loss of moisture during weighing, and a balance sensitive to 0·1 gram.

A sample of about 50 grams is taken from the thoroughly mixed portion of the soil passing the 40 mesh sieve obtained in accordance with the Standard Method of Preparing Soil Samples A.S.T.M. D421-39 and A.A.S.H.O. T87-42. The air-dried sample is placed in the evaporating dish and mixed with distilled water; the latter is added in small amounts and the sample thoroughly mixed after each addition of water. As soon as the wetted soil forms into balls under manipulation, the sample is smoothed with a light stroke of the spatula and a drop of water placed on the smoothed surface. If the drop of water disappears in 30 seconds, a few drops of water are mixed with the sample, and the process repeated until the drop of water does not disappear in 30 seconds but spreads over the smoothed surface, leaving a shiny appearance (see Fig. 73*a*). A small portion of the soil on which the last drop of water was placed is removed and put between two watch glasses. The weight of the wet soil and container is found, and the sample is then oven-dried to constant weight and again weighed.

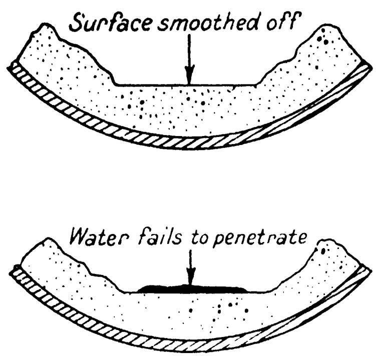


Fig. 73(*a*).—Phenomena occurring during the field moisture equivalent test.

$$\text{The field moisture equivalent} = \frac{\text{weight of water added}}{\text{weight of oven-dried soil}} \times 100$$

Shrinkage Factors of Soils. A.S.T.M. D427-39, A.A.S.H.O. T92-42.

This method of test gives the following soil constants :—

Shrinkage limit.
Shrinkage ratio.
Volumetric change.
Lineal shrinkage.
Specific gravity.

The apparatus required includes a $4\frac{1}{2}$ inch diameter evaporating dish, spatula with blade about 3 inches long and $\frac{3}{4}$ inch wide, circular shrinkage dish with flat base, about $1\frac{3}{4}$ inch diameter and $\frac{1}{2}$ inch high, 12 inch steel straight edge, glass cup about 2 inches in diameter and 1 inch high with a top rim ground smooth and level, glass plate with three metal prongs for immersing the soil pat in mercury, 25 ml. glass measuring cylinder graduated to 0·2 ml., balance weighing to 0·1 gram, and enough mercury to fill the glass cup to overflowing (see Fig. 73*b*).

Thirty grams of thoroughly mixed material passing the 40 mesh sieve is placed in the evaporating dish and thoroughly mixed with distilled water sufficient to fill the soil voids completely and to make the soil pasty enough to be readily worked into the shrinkage dish without the inclusion of air bubbles. (In the case of sandy soils, this amount of water is equal to or just above the liquid limit, and in the case of clay soils, the amount may exceed the liquid limit by about 10 per cent.).

The interior of the shrinkage dish is smeared thinly with vaseline to prevent the soil from sticking to the dish. A volume of wet soil of about one-third the volume of the dish is put in its centre, and the soil caused to flow to the edges by tapping it gently on a hard surface.

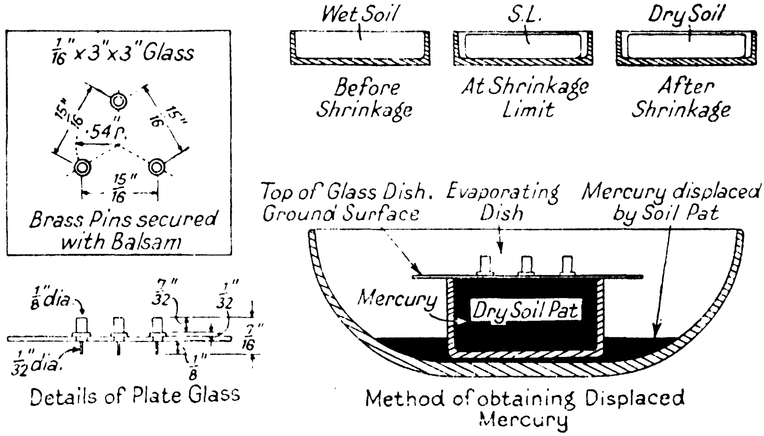


Fig. 73(*b*).—Apparatus for determining the volumetric change of subgrade soils.

A second and similar volume of soil is added and the dish tapped until the soil is thoroughly compacted and all included air has been brought to the surface. More soil is added and the tapping continued until the dish is completely filled and excess soil stands above its edge. The excess soil is struck off with the straight edge, and all soil adhering to the outside of the dish is wiped off. The dish is immediately weighed, and the soil pat allowed to dry in air until its colour changes from dark to light. It is then oven-dried to constant weight at 110° C., cooled and weighed, as is also the empty dish. The volume of the dish is found by filling it to overflowing with mercury, removing the excess by pressing a flat glass plate over its top, and measuring the volume of mercury contained in the dish. This volume is recorded as the volume V of the wet soil pat. The volume of the dry soil pat is found by removing it from the shrinkage dish, placing it on the surface of mercury just filling the glass cup, forcing it under the mercury by means of the glass plate with three prongs attached, taking care to see that no air is entrapped by the pat, and measuring the volume of mercury displaced by submersion of the pat, this volume being the volume V_0 of the dry soil pat.

The standard method of finding volume change, described above, may sometimes be somewhat more detailed than is justified, especially in cases where only a rough value is required. It is also a method which gives rise to difficulties with very sandy soils, owing to breaking up of the pat when placed under mercury. The writer has developed the following approximate method for finding volume change of soils under such circumstances :—

Approximate Method of Determination of Volume Change of Soils.

Wet soil at its liquid limit is heaped into a glass Petrie dish 2 inches in diameter, the dish being first smeared with vaseline to prevent the soil from sticking to its sides and base, and the soil is pressed into the dish with a spatula, care being taken to avoid the inclusion of any air bubbles, and the upper surface is struck off flush with the sides of the dish. After the pat has been prepared, it is left to dry in air for 24 hours, when it is finally oven-dried for a further 24 hours. It is then cooled, its diameter measured and compared with the original diameter. If greater accuracy is required, the change in height can also be found, but in most cases, the change in height is so small as compared with the change in diameter that it can be ignored.

An example of the method of calculation is as follows :—

The measured diameter of the soil pat is 1·9 inches after drying and the change in height is negligible. The radius after drying is 0·95 inches.

Volume of dry soil pat $= h \times (0{\cdot}95)^2\pi$, where h = height of pat

" " wet " " $= h \times (1)^2\pi$

$$\text{Volume change} = h(1^2 - 0{\cdot}95^2)\pi$$
$$= h(1 - 0{\cdot}95)(1 + 0{\cdot}95)\pi$$
$$= h \times 0{\cdot}05 \times 1{\cdot}95 = 0{\cdot}0975h\pi$$
$$\text{Percentage Volume Change} = \frac{100 \times 0{\cdot}0975h\pi}{(0{\cdot}95)^2h\pi} = \text{11 per cent. approximately.}$$

The *shrinkage limit* of a soil is the moisture content, expressed as a percentage by weight of the oven-dried soil, at which a reduction in moisture content will not cause a decrease in the volume of the soil mass, but at which an increase in moisture content will cause an increase in the volume of the soil mass. It is calculated from

$$S = M - \left(\frac{V - V_o}{W_o}\right) \times 100$$

where S = shrinkage limit

M = moisture content of wet soil in percentage of the weight of the oven-dried soil.

V = volume of wet soil pat.

V_o = volume of dry soil pat.

W_o = weight of the oven-dried soil pat.

The *shrinkage ratio* of a soil is the ratio between a given volume change, expressed as a percentage of the dry volume, and the corresponding change in moisture content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil; it equals the apparent specific gravity of the soil pat, and is denoted by $R = \frac{W_o}{V_o}$.

The *volumetric change* of a soil for a given moisture content is the volume change, expressed as a percentage of the dry volume, of the soil mass when the moisture content is reduced from the stipulated percentage to the shrinkage limit; this stipulated moisture content is usually taken as the field moisture equivalent. It is denoted by $C_f = (M_1 - S)R$, where M_1 = given moisture content. The volumetric change C_f also equals $(F - S)R$, where F = field moisture equivalent.

The *lineal shrinkage* of a soil for a given moisture content is the decrease in one dimension, expressed as a percentage of the original dimension, of the soil mass when the moisture content is reduced from an amount equal to the field moisture equivalent to the shrinkage limit. The lineal shrinkage $= 100\left(1 - \sqrt[3]{\frac{100}{C_f + 100}}\right)$.

The *specific gravity* of a soil is the weight of the oven-dried soil divided by the true volume of the soil particles. The specific gravity may be calculated from the data obtained from the volumetric shrinkage test from the relation

$$\text{Specific gravity} = \frac{1}{\frac{1}{R} - \frac{S}{100}}.$$

Volume Change of Soils. A.A.S.H.O. T116-42.

The test applies to both undisturbed and remoulded samples of soil. The apparatus required includes a mould of dimensions sufficient to permit of the making or use of a sample 4 inches in diameter and $1\frac{9}{16}$ inches high, a cylinder or upper mould serving as a guide for the moulding piston, a base, a moulding piston, a thickness gauge for controlling the thickness of the sample, a metal calibration blank 4 inches in diameter and $1\frac{9}{16}$ inches high, a volume change apparatus consisting of a mould, upper mould and base as just described, a perforated piston, porous stones above and below the sample, micrometer dial and dial support, loading beam, a 200 gram balance sensitive to 0·1 gram, No. 4 mesh A.S.T.M. sieve, drying oven, 100 ml. measuring cylinder, 12 inch straight-edge, spatulas, trowels and mixing pans.

The test is made on soil passing the No. 4 mesh sieve, selected by quartering or riffling, and the soil is mixed thoroughly with a predetermined amount of water. The amount of hygroscopic moisture is first determined (as per A.A.S.H.O. Method T88), the amount of water required to bring the air-dried sample to its desired moisture content is calculated, and the quantity of moist soil necessary to produce a test sample of the required density is found. An example of such calculation is as follows :—

Size of sample.

Diameter	4 inches.
Area	12·5664 square inches.
Thickness	1·5625 inches.
Volume	19·635 cubic inches = 0·011363 cubic foot.
Weight of dry soil in grams	= dry density in lbs. per cub. ft. × 0·011363 × grams per lb.
	= dry density × 0·011363 × 453·59 grams
	= dry density × 5·154 grams.

Thus the density of the soil in lbs. per cubic foot × 5·154 = No. of grams of soil required to produce a sample of the required size and density.

Density, moisture and amount of soil required per sample is obtained as follows :—

If w = hygroscopic moisture of soil in per cent.
w_1 = moisture content of soil in per cent. at beginning of test.
w_2 = additional amount of water necessary to obtain w_1.
D = desired dry density in lbs. per cubic foot.

Then weight of oven-dried soil per sample in grams $= W = D \times 5{\cdot}154$

and weight of air-dry soil per sample in grams $= W_1 = 5{\cdot}154D\left(1 + \frac{w}{100}\right)$.

Total weight of dry soil + water per sample in grams =

$$W_2 = 5{\cdot}154D\left(1 + \frac{w_1}{100}\right)$$

and weight of water to be added to air-dry soil $= w_2 = W_2 - W_1$.

Samples are prepared as follows :—

Enough soil to give 75 grams over and above that required for the sample itself is mixed with water to the desired moisture content. This should be done at a temperature of 70° F. and relative humidity 50% in order to keep evaporation losses at a minimum. The mixture is cured for 15 minutes under conditions which prevent loss of moisture. The calibration blank is then calibrated with the thickness gauge so that the readings obtained with the latter can be used to get a proper thickness of sample for test. The soil sample is compressed to the proper thickness, the moulding piston being placed in position and the compression load applied at a rate of 0·1 inches per minute. At the point at which the correct thickness of sample is reached, the load is held for 30 seconds and then released. The moulding piston is removed, porous stones placed above and below the sample, and a sealing load of 1,000 lbs. or 1/10th of the total moulding load applied, whichever is the less. The moulding apparatus is removed from the testing machine and the initial thickness of the sample determined.

The volume change is determined under a load of 22·3 lbs. per sq. inch under a lever-arm loading device similar to that used in a consolidometer. The compacted sample is inundated with water and the vertical increase in dimension of the sample found every 15 minutes during the first hour after inundation, then every hour for the first day and thereafter twice a day until the movement does not exceed ·001 inch during 12 hours.

The computation of volume change is as follows :—

V_1 = initial volume.

V_F = volume at end of test.

V_2 = volume at any moment between V_1 and V_F.

$$\text{Total volume change in per cent.} = \frac{V_F - V_1}{V_1} \times 100$$

$$\text{Partial ,, ,, ,, ,, ,,} = \frac{V_2 - V_1}{V_1} \times 100.$$

Since the volume is directly proportional to the thickness

$$\text{Total volume change in per cent.} = \frac{T_F - T_1}{T_1} \times 100$$

$$\text{Partial ,, ,, ,, ,, ,,} = \frac{T_2 - T_1}{T_1} \times 100$$

where T_1 = initial thickness,

T_2 = thickness at any moment during test,

T_F = thickness at end of test.

Interpretation of Results of Indicator Tests on Soils.

As a result of the study of the road behaviour of many types of soil used as sub-grades and foundations, the test properties of which were known, a considerable amount of work has been done in U.S.A. and elsewhere towards the correlation of the properties of soils with their probable behaviour under traffic. Some of the results of this work are given in Chapter 6 of this book (Classification of Soils).

A study of mechanical analysis and other soil classification tests applied to British soils has been made by the Road Research Laboratory and has shown that :—

1. The effect of the clay content of the soil upon its index properties depends upon the nature of the clay, and for this reason the relationship between these quantities exhibits considerable variability except for soils of the same type.

2. The centrifuge moisture equivalent increases with the " clay " content, but the relation is not linear.

3. For a given type of soil, the plastic and liquid limits are linearly related to the " clay " content.

4. The field moisture equivalent varies only slowly as the " clay " content increases and even for a given type of soil exhibits considerable variability, the cause of which is not known. Even as an index test, its value seems very doubtful.

5. The shrinkage limit is almost independent of the " clay " content, and averages about 12 per cent.

6. Linear relationships exist between the plastic and liquid limits, and therefore between the plasticity index and liquid limit. These were found to apply to all soils free from organic material.

7. For practical purposes, only mechanical analysis, liquid and plastic limit determinations appear to be necessary.

The author suggests that the volume change of soils is of great importance from an engineering point of view, and that the volume change test should therefore be included under 7, above.

Determination of Organic Content of Soils.

This is an approximate method designed for field use only. The apparatus required includes a 40 ml. beaker, 50 ml. measuring cylinder, balance weighing to 0·1 gram, and a riffler.

Three to 4 kg. of soil is riffled down to about 40 grams and dried. The ·4 ml. beaker is weighed, the soil placed in it and the soil and beaker weighed to obtain exact weight of soil. 5 ml. of 6 per cent. hydrogen peroxide is poured on to the soil and the mixture warmed gently to about 60° C., stirring gently during the warming. More hydrogen peroxide is added until bubbles cease to appear. When reduction of the organic matter seems to be complete, all the liquid is evaporated off, after which the soil and beaker is dried and re-weighed. The quantity of organic matter removed is calculated as the loss of weight and expressed as a percentage of the dry weight of the original sample.

It is to be regretted that more work has not been done on the organic content of soil met with by engineers, and that no standard test is yet available for its determination.

Permeability of Soils.

Much research is needed before this property of soils can be tested according to standard methods. One suggested method of test is that of E. S. Barber[26] as follows :—

Scope. This test is for the determination of the coefficient of permeability of granulated soil in an arbitrary state. No definition of permeability is given ; the test is made on loose soil passing the No. 10 mesh sieve.

Apparatus. The apparatus required consists of the permeameter shown in Fig. 74. The filter screen covering the bottom of the transparent tube is made of No. 200 mesh sieve cloth; distilled water is used as the permeating liquid.

Procedure. Air-dry soil is poured into the tube to a depth of 6 inches, and the tube is gently immersed in distilled water at room temperature to a depth of 10 inches above the screen. After the water has risen in the tube 10 inches to the level of the outlet, the tube is filled with distilled water at a temperature of between 5° and 10° C. to a depth of 36 inches above the outlet, care being taken not to disturb the soil. The time required for the water in the tube to drop 24 inches and 36 inches respectively is recorded. Finally, a thermometer is thrust into the soil and the temperature noted.

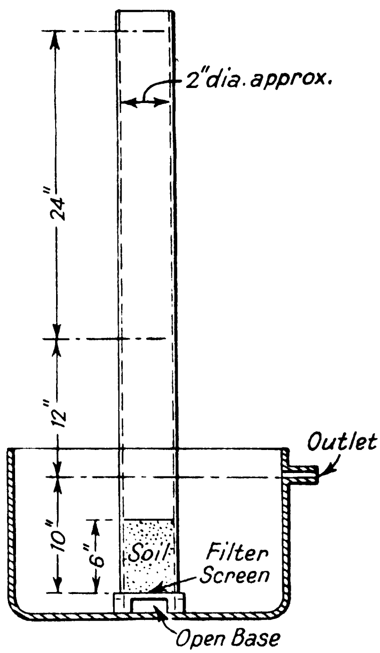


Fig. 74.—Permeameter for testing pervious granular materials.

The coefficient of permeability k in feet per day is obtained from the formula

$$k = \frac{276Cd}{t} \log \frac{h}{h_2}, \text{ where}$$

d = depth of soil in inches,

t = time in minutes for the water to drop from the initial level h to the final level h_2

C = temperature correction (see Table 8 below)

$$= \frac{\text{viscosity of water at room temperature}}{\text{viscosity of water at } 68^\circ \text{ F.}}$$

For the dimensions given in Fig. 74, the above formula reduces to

$$k = \frac{790C}{t}$$

TABLE 8.—VALUES FOR TEMPERATURE CORRECTION C IN BARBER'S PERMEABILITY FORMULA.

Temperature		Factor	
Centigrade	Fahrenheit	C	$790C$
15	59	1·13	896
20	68	1·00	790
25	77	0·89	702
30	86	0·80	629
35	95	0·72	568
40	104	0·65	516

Another method of test is that proposed by J. Mizroch[27]. This test is for the determination of the capillary rise of water in soils with a view to the evaluation of their susceptibility to damage by frost. It is made upon the material passing the No. 10 A.S.T.M. sieve. The soil, plugging the top of a glass tube, is subjected to the pull of an increasing head of water which compresses the soil and reduces its moisture content until air is forced through the soil by a maximum head of water which is called the capillary rise.

Apparatus. The apparatus required consists of a glass filter tube similar to that shown in Fig. 75, a cork disc fitted to the shoulders of the filter tube, a glass tube of the same diameter as the lower part of the filter tube and at least 1 metre long, a cylindrical glass jacket 5 cm. in diameter and long enough to accommodate the filter tube and extension suspended vertically in the centre of the cylinder, a No. 200 mesh A.S.T.M. sieve disc to fit the filter tube.

Procedure. The apparatus is assembled as shown in Fig. 75. When the soil under test contains particles passing the No. 200 mesh sieve, a filter paper is placed on the sieve. Powdered soil is poured without tamping into the filter tube to a height of 4 cms. Water is admitted to the jacket until the apparatus is filled to a level slightly above the top of the soil. After five minutes' inundation, the level of the water in the cylindrical glass is lowered to that of the bottom of the cork disc. The excess water is allowed to drain from the soil; the elevation of

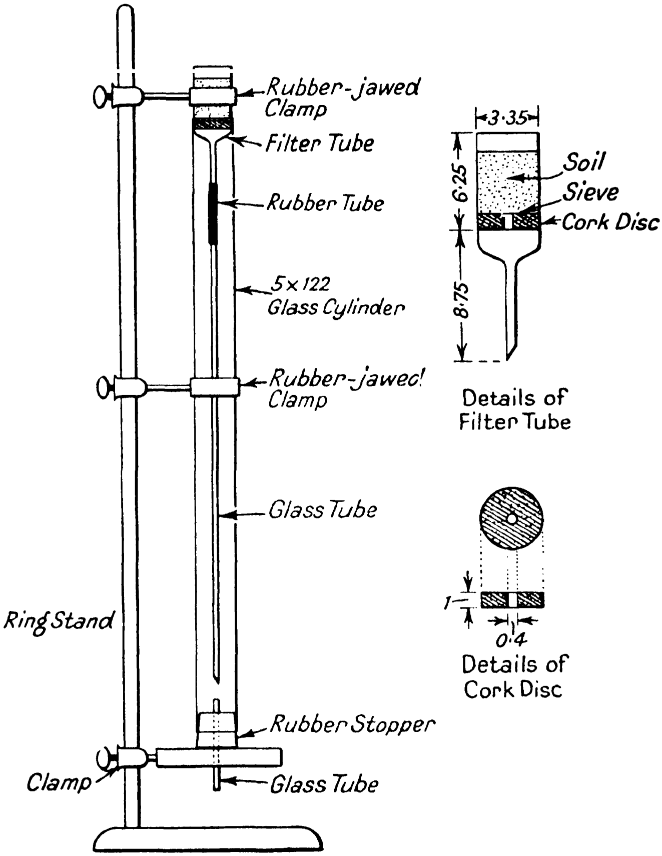


Fig. 75.—Capillary apparatus. (Dimensions in cm.).

the water in the jacket is then lowered by 2 inch increments with a five minute pause for additional drainage of the sample between each increment. The distance in inches between the top of the water in the jacket and the top of the soil when the water column in the filter tube breaks is reported as the capillary rise of the soil.

The California Bearing Ratio Test.

This test, commonly known as the C.B.R. test, is an *ad hoc* penetration test developed by the California State Highways Department for the evaluation of the load-bearing capacity of base-courses and sub-grades which are to carry flexible pavements. It consists of a determination of the load-penetration relation for a plunger of an area of 3 square inches which is forced into a sample of soil prepared in a standard manner. Samples are tested at optimum moisture content and again after inundation with water, and Table 9 gives the values adopted by the users of the test as standards for different penetrations applying to a soil with a C.B.R. value of 100 per cent.

TABLE 9.—STANDARD PRESSURES USED IN CALIFORNIA BEARING RATIO TEST.

Penetration of plunger in inches	Standard pressure, i.e., resistance of soil
0·1	1000 lbs. per square inch
0·2	1500 ,, ,, ,, ,,
0·3	1900 ,, ,, ,, ,,
0·4	2300 ,, ,, ,, ,,
0·5	2600 ,, ,, ,, ,,

The measured pressures obtained at the specified penetrations are expressed as percentages of the pressures in Table 9, and these percentages are termed the C.B.R. values for the particular soil. For design purposes, the C.B.R. value at 0·1 inch penetration is normally used.

This test is empirical and is not based on mathematical reasoning; it is only of use if data are available showing the results of running a known intensity of traffic over a pavement of known properties laid on a soil of known C.B.R. value. This information has been obtained by the U.S. Corps of Engineers and summarised in a series of design curves (see Fig. 77).

It is unfortunate that the exact relation between the C.B.R. of a soil and k, its modulus of sub-grade reaction (see page 143) is not understood at present. For this reason it should be pointed out that the C.B.R. method of determining the thickness of foundation for a given wheel load can only be applied to a foundation which is to carry flexible surfacing. Comparison of this method of calculating soil thickness for various loads with other methods will be found in Chapter 8 (Loads on Soils and the Design of Base Courses and Sub-grades), while other observations on the results of the test will be found on page 135 of the present chapter.

Apparatus. The apparatus required includes C.B.R. equipment complete with jack, frame, cylinders, rammers, dial gauges, drying oven, ¾ and $\frac{3}{16}$ B.S. (or equivalent A.S.T.M.) sieves, direct-reading balance and galvanised iron soaking tank (see Figs. 78, 79).

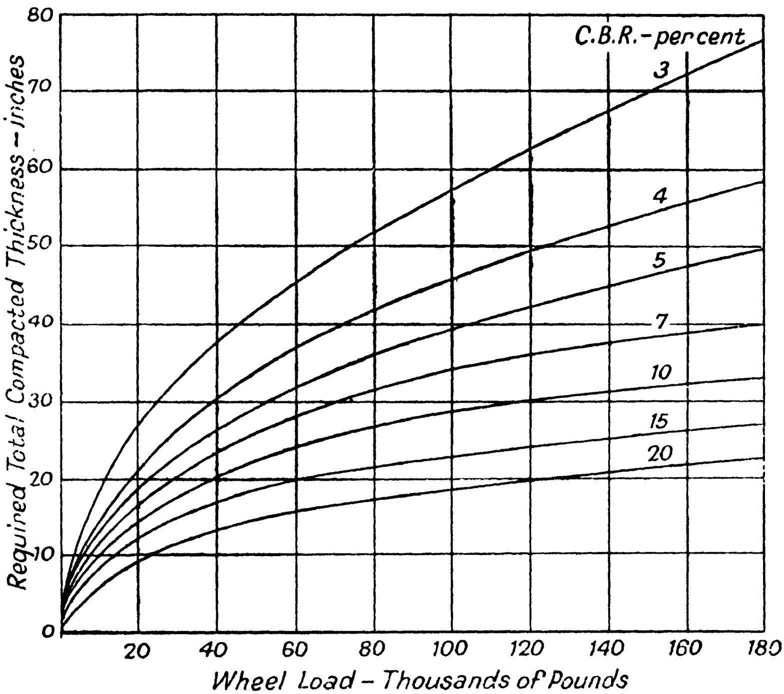


Fig. 76.—California bearing ratio test. Design chart showing relation between California bearing ratio and foundation thickness (for flexible pavements only).

Procedure. The test is made on soil passing the ¾ inch mesh sieve, all the material retained on this sieve being replaced with an equal weight of material passing the ¾ inch but retained on the $\frac{3}{16}$ inch mesh sieve. The C.B.R. optimum moisture content is determined by a procedure similar to that used for maximum density, but using the 10 lbs. compacting rammer dropped through 18 inches, and compacting the sample in five equal layers. The Proctor cylinder may be used for this purpose, but if over 50 per cent. of the material is retained on the $\frac{3}{16}$ inch mesh sieve the test should be made in the 6 inch C.B.R. mould, using 55 blows per layer for compaction purposes. A moulding cylinder is weighed, the collar, cylinder and base-plate clamped together, and the soil sample compacted at optimum moisture content.

The collar and base-plate are removed and the top of the sample trimmed, the sample being then weighed to determine its density. The base-plate is replaced on the trimmed end of the specimen so as to leave a flat working surface, the collar is attached and a filter paper placed on the exposed soil surface. The perforated plate (see Fig. 78), with adjustable stem, is inserted, and annular weights of a total of at least 10 lb. placed on it. The measuring bridge is placed on the collar, and the perforated plate stem adjusted so that a reading on the dial gauge can be taken. The whole mould is immersed in water in such a way that the water level stands about 1½ inches above the

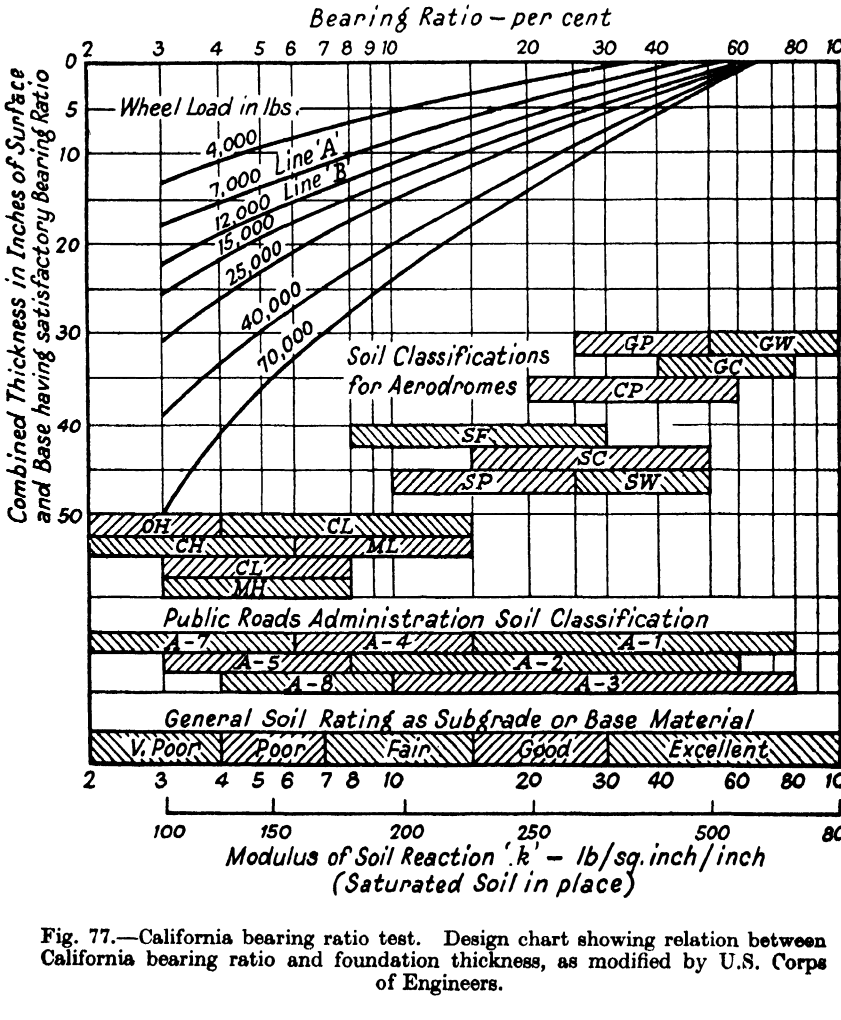
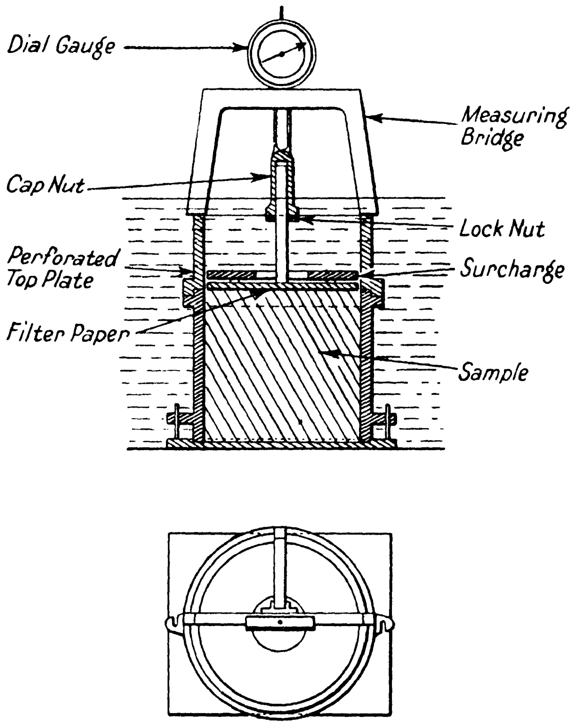


Fig. 77.—California bearing ratio test. Design chart showing relation between California bearing ratio and foundation thickness, as modified by U.S. Corps of Engineers.

surface of the sample (see Fig. 78). Dial gauge readings are taken twice a day to determine the expansion of the sample caused by absorption of water; these readings are continued until expansion ceases (usually after 3 or 4 days). After soaking, the annular weights



Assembly for soaking the Sample

Fig. 78.—Apparatus for making the California bearing ratio test.

are removed and the surface water drained off by loosening the collar and allowing the soil to drain for 15 minutes. The perforated plate and filter paper are removed and the sample with base-plate and collar are placed in the testing machine (see Fig. 79). The annular weights should be kept in place at this stage except in the case of highly cohesive soils. The mould is jacked up until the plunger is just seated on the surface of the soil and the pressure gauge registers 10 lbs. or some other slight load. The dial gauge on the frame is adjusted to bear on the top of the collar and the reading is taken. The load is applied in such a way that the rate of penetration of the plunger is about 0·05 inches per minute, load readings being taken at penetrations of 0·05, 0·1, 0·2, 0·3, and 0·5 inches. After penetration, the moisture content is determined from a sample of the top inch of the soil.

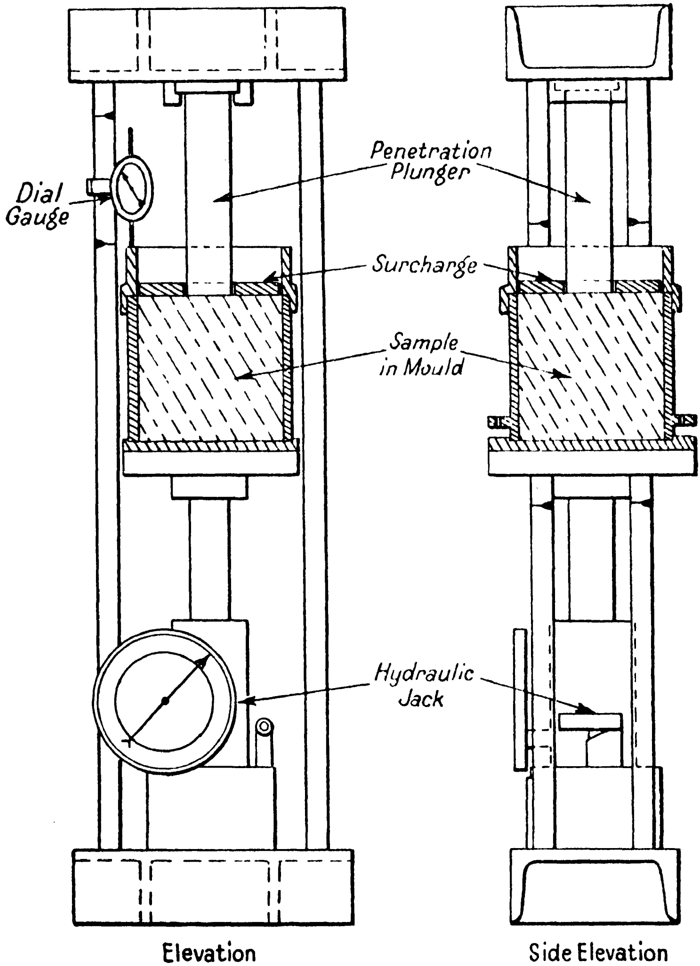


Fig. 79.—California bearing ratio test. Showing soil sample under test in compression machine.

Fig. 76 shows a typical design chart for use in the test, while Fig. 77 shows a similar but more elaborate chart in use by the U.S. Corps of Engineers.

In using Fig. 77, line "*A*" applies to lightly trafficked roads, and line "*B*" to heavily trafficked roads. The range of bearing ratios given in the diagram are for typical soil and are based on 0·1 penetrations. The bearing ratio is intended to be applied to individual soils, whereas the modulus of soil reaction k should only be applied to a uniform mass of soil. It should also be noted that if the wearing course is less than one inch thick, the minimum thickness of the top layer of the base should vary from 4 inches for 40 per cent. bearing ratio material for a 4,000 lb. load to 8 inches for 80 per cent. bearing ratio material for a 70,000 lb. load

A technical study of this test has been made by the Road Research Laboratory, as a result of which it is pointed out that the assumption underlying the test evidently was that water would ultimately enter the soil and soften it, regardless of the condition of the soil when the road is constructed. This is quite a safe assumption to make. Further conclusions by the Laboratory were as follows :—

1. The densities of the compacted soil specimens at the commencement of the test were very much higher than the maximum field densities found in Britain for the same types of soil.

2. In spite of this, the densities and moisture contents of the top inch of the soaked test specimens were practically identical with the values for the corresponding saturated undisturbed soils *in situ*, but differed considerably from the value obtained for soils which had been artificially placed and which were not saturated.

3. Although the results of penetration tests on both compacted and soaked specimens were not reproducible within narrow limits, it was possible by making a number of tests to differentiate between subgrade materials of different qualities.

4. C.B.R. values were higher when compacted density was high and when clay content, liquid limit and plasticity index were low.

This test has been criticised on the grounds that it ignores the strength of the wearing surface. In addition, no distinction is made between a good and a bad base-course, since the method of design gives a figure for the total thickness of a homogeneous material. A case has in fact been quoted in which an aerodrome runway was taking 25,000 lb. wheel loads on soils which according to C.B.R. tests should be 32 inches thick, but which had not failed although it was only 20 to 21 inches thick.

The U.S. Waterways Experiment Station has carried out a critical study of the California Bearing Test, more especially as regards the laboratory methods involved, and has come to the following conclusions thereon :—

1. The C.B.R. test is extremely sensitive to changes in moulding water content and dry density.

2. The C.B.R. of a soil rises with increase in moulded dry density. For a given density and compaction, samples moulded drier than the optimum have a higher C.B.R. than those moulded wetter than the optimum, except in the case of highly plastic clays, where the reverse is the case.

3. Surcharge during the soaking period affects the C.B.R. only where it tends to cause consolidation or to prevent swelling ; surcharge during the penetration test affects the C.B.R. of cohesionless soils only.

4. In some cases samples soaked from both top and bottom have a C.B.R. of only one-half to one-third of similar samples soaked from the bottom only.

5. Dynamic compaction of the soil gives a range of structure varying from very rigid to very spongy as the moulding water content rises, while static compaction gives one type of structure irrespective of moulding water content. This latter conclusion is of extreme importance, and makes it advisable to consider whether dynamic compaction ought ever to be used in the preparation of samples for this test.

Wetting and Drying (Durability) Test for Compacted Soil-Cement Mixtures. A.S.T.M. D559-44.

This test determines the soil-cement losses, moisture changes and volume changes produced by repeated wetting and drying of compacted specimens of soil-cement mixtures of known composition, density and moisture content.

Apparatus. The apparatus used includes a cylindrical metal mould internal capacity 1/20 cubic foot, internal diameter 4 inches and height about 4·6 inches, with a detachable collar about $2\frac{1}{2}$ inches high, for compaction of soil (see Proctor Test on page 110 for fuller details); metal rammer weighing $5\frac{1}{2}$ lb. and having a 2 inch diameter circular face, equipped with a suitable arrangement to control the vertical drop ; closed cylindrical sleeve slightly less than 4 inches diameter or similar device for removing compacted specimens from mould ; 25 lb. scale sensitive to 0·01 lb., and a 100 gram balance sensitive to 0·01 gram ; thermostatically controlled drying oven capable of maintaining a temperature of about 110° C. ; a humidity chamber capable of maintaining a temperature of 21° $\pm$ 1·7° C., and a relative humidity of not less than 90 per cent. for seven-day storage of compacted specimens ; a tank suitable for submerging compacted specimens in water at about 21° C.; a wire scratch brush made of 2 inch × $\frac{1}{16}$ inch flat No. 26 gauge wire bristles assembled in 50 groups of 10 bristles on a $7\frac{1}{2}$ inch × $2\frac{1}{2}$ inch hardwood block.

Procedure. The test is carried out on soil-cement mixtures compacted in the mould, before hydration of the cement, to maximum density at optimum moisture content ; representative roadway soil is used, but modified as now to be described. If all the soil passes a No. 4 mesh sieve, enough soil-cement mixture is prepared at optimum moisture content to provide two compacted specimens 4 inches in diameter and about 4.6 inches high. If some of the soil is retained on the No. 4 mesh sieve, the maximum size material permitted in the test is $\frac{3}{4}$ inch, and compensation is made for any material larger than $\frac{3}{4}$ inch by replacing it with an equal dry weight of material passing No. $\frac{3}{4}$ inch sieve and retained on No. 4 mesh sieve of the same gradation as that contained in the original sample. The amount of soil-cement mix prepared is then enough for two specimens as described above.

The prepared soil-cement mix is compacted in the mould in a manner similar to that used for the Proctor Test (see page 110), except that before placing and compacting the next layer, smooth compaction planes produced during compaction of each layer are scarified to form grooves at right angles to each other about $\frac{1}{8}$ inch wide, $\frac{1}{8}$ inch deep and about $\frac{1}{4}$ inch apart. During compaction, a representative soil-cement mixture moisture sample is taken from the batch, weighed, dried in an oven at 110° C. for at least 12 hours ; from this specimen, the moisture content at compaction can be determined.

The remainder of the compacted specimens are removed from their moulds, weighed and their density determined ; the volumes are found by averaging the diameters measured to the nearest 0·02 inch at right angles to each other at the centre and the quarter points of the specimen and averaging the heights measured to the nearest 0·02 inch at the quarter points on the circumference of the specimen.

One specimen is marked as No. 1, and is used to obtain data on moisture and volume changes during the test ; the other is marked as No. 2, and is used for the determination of soil-cement losses during the wetting and drying (durability) test. After measurement as just described, the specimens are put in the humidifying room for 7 days, No. 1 specimen being weighed and measured each day to find its volume and moisture content.

The testing of the specimens is carried out as follows :—

After storage in the humidifying room, they are put under tap water at room temperature for 5 hours, removed from the water, No. 1 specimen being weighed and measured. Both specimens are placed in an oven at 71° C. for 42 hours and removed and weighed and No. 1

specimen measured. Specimen No. 2 is given two firm strokes on all areas with the wire scratch brush to remove all material loosened during wetting and drying. The specimen is weighed after brushing and the oven-dry weight of material removed by brushing is calculated.

The foregoing constitutes one cycle of the durability test ; 12 cycles in all are used for this test. In all the tests (volume change, moisture content and wet and dry test loss) the changes recorded are given as percentages of the original oven-dry weight of the specimen.

This test is used widely for determining optimum cement contents in soil-cement stabilisation work (see Chapter 9). It suffers from the following disadvantages:—The use of the steel-wired brush involves a very definite personal factor, some operators brushing more vigorously than others ; it is very tedious, and it takes several weeks to complete.

Freezing and Thawing Test of Compacted Soil-Cement Mixtures. A.S.T.M. D560-44.

This test resembles A.S.T.M. D559-44 described above, but uses freezing and thawing instead of brushing as the testing agent.

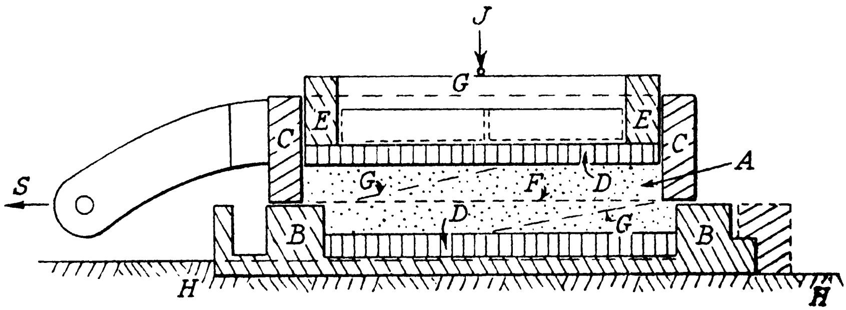
Apparatus. The apparatus used is similar except that a freezing cabinet working to $-23°$ C. is required in addition to a steel brush.

Procedure. The preparation of samples is also similar and the procedure is the same except that freezing and thawing replaces the brushing process. The freezing and thawing is carried out as follows : At the end of the storage in the humidity room, the specimens are placed in water-saturated felt or blotters and the whole put in a refrigerator and kept at $-23°$ C. for 22 hours and removed. After 22 hours in the humidity room, specimen No. 2 is brushed with a standard steel wire brush, this constituting a single cycle of freezing and thawing. Twelve cycles are used if possible, and the losses in the test are measured and recorded as in D559-44.

Compression Tests on soil-cement mixtures are carried out on 3 inch soil-cement cubes prepared at optimum moisture content and cured in a humid atmosphere at room temperature for 7 days.

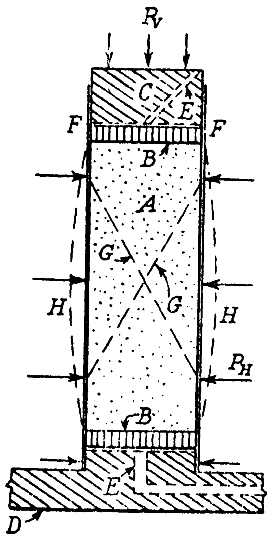
Shear and Compression Tests on Soils.

Shear and compression tests on soils are not yet standardised, although a good deal of data is available [28] on the subject, which is very involved. Three well-known tests of this kind are (a) the direct shear test, already referred to in Chapter 2 of this book, the apparatus of which is shown in Fig. 80, (b) the unconfined compression test, which does not require detailed description here, (c) the tri-axial compression test. This latter test is done on a long thin cylinder of soil (see Fig. 81) contained in a rubber bag, the outside of which is subjected to hydraulic pressure. The lateral pressure thus applied increases the crushing strength of the soil by an amount which is related to the angle of internal friction ϕ.



A. Sample.
B. Lower Frame.
C. Upper Frame.
D. Porous Stones.
E. Piston.
F. Principal Plane.
G. Secondary Planes.
H. Base.
J. Normal Force.
S. Shearing Force.

Fig. 80.—Apparatus for making direct shear test of soil.



A. Sample.
B. Porous Stones.
C. Piston.
D. Base.
E. Drainage Channels.
F. Rubber Tube.
G. Planes of Failure.
H. Bulge after failure.
P_V Vertical Pressure.
P_H Horizontal Pressure.

Fig. 81.—Tri-axial shear or compression apparatus. See also Fig. 44.

This test is free from some of the disadvantages of the direct shear test, in which the distribution of stress in the soil sample is probably not uniform. According to D. W. Taylor, the average difference in the value of the maximum friction angle ϕ is of the order of $1°$ to $2°$. Terzaghi's method of finding the angle of internal friction[29] was developed from Mohr's Circle of Stress, and utilises the well-known relation:

$$\frac{p_m}{p} = \tan^2\left(45° - \frac{\phi}{2}\right)$$

where p_m = lateral pressure,
p = applied vertical pressure,
ϕ = angle of internal friction.

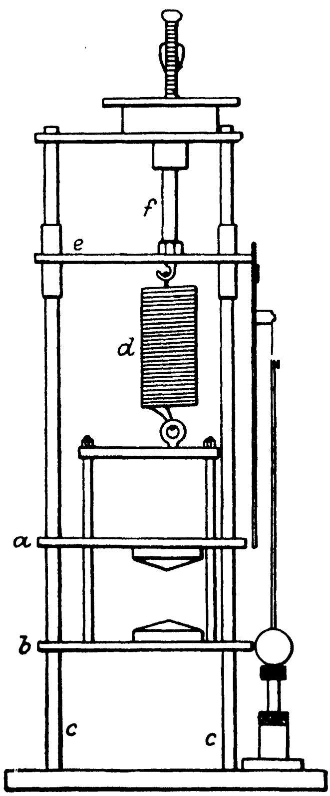


Fig. 82.—Showing portable apparatus for making compression tests on soils. (Cooling and Golder.)

L. F. Cooling and H. Q. Golder[30] have described a portable apparatus for the making of compression tests on clay soils. It consists of a spring-loaded device (see Fig. 82) in which a cylindrical specimen of clay soil is subjected to an axial compression stress, the load-deformation curve being drawn on graph paper by an autographic recording mechanism. The sample used is a cylinder of clay $1\frac{1}{2}$ inches in diameter and $3\frac{1}{4}$ inches to $3\frac{1}{2}$ inches long; the ends are hollowed out by means of a cutter so shaped that it produces ends which will fit the cone-shaped end-pieces. These cones reduce barrelling and tend to preserve the cylindrical shape of the specimen during compression; they also help to centre the specimen in the machine.

Experience has led to the use of cones with base angles of between $10°$ and $15°$, the larger angle being used for testing softer types of clay. The cones are smeared with a thin film of oil before being placed in position. The specimen is compressed longitudinally between the conical bearing surfaces of plates *a* and *b* (Fig. 82). The upper bearing plate is fixed, while the lower bearing plate slides on the vertical rods *c*, friction between the sliding surfaces being reduced by the use of adjustable steel bearing screws carrying hard steel balls in their ends. Plate *b* is moved upwards by the pull of the compression spring *d*, the connecting rods passing through clearance holes in the plate *a*. The upper end of the spring is attached to the plate *e*, which is fitted with brass sleeves sliding on the rods *c* to locate the plate *e* in a horizontal plane. The combined nut and handle *f* moves the plate *e* by means of a long rod, screwed 26 threads to the inch, a thrust-race being inserted between the top plate and the handle.

In order to be able to test materials of widely differing strengths, it is advisable to use a series of springs of different stiffnesses. During the test, it will be found that the cross sectional area of the sample increases, and this increase should be allowed for in the calculation of the maximum stress in the sample. It is assumed that the volume of the specimen remains constant and that its shape remains cylindrical. The maximum stress can then be obtained by calculating the stress at a few points near the maximum load and allowing for the increase in area.

The samples are obtained by means of a brass sampling tube 6 inches long and $1\frac{1}{2}$ inch internal diameter, one edge of which is bevelled outside to form a cutting edge. The tube, greased inside and out, is forced into the soil for a depth of about 4 inches and then twisted through 360° to shear off the soil at its base ; the surrounding soil is removed to release the tube and its contained sample. The clay is trimmed off flush with the end of the tube, and a conical recess is formed in the trimmed face by means of a special cutter. A brass cone $1\frac{7}{16}$ inch diameter is oiled and placed on the recessed end of the sample, and the cone and sample pushed to the other end of the tube by means of a steel rod. When the sample projects from the end of the tube it is cut off to give the correct length of the specimen, and the sample is ready for testing.

Failure takes place by shear on a diagonal plane, and the angle of failure, measured from the vertical, varies with different samples. Theoretically, this angle equals $\left(45^\circ - \frac{\phi}{2}\right)$, whence ϕ can be calculated.

It should be noted that this method cannot be used for material containing bedding planes or slickensides (lines of natural shear), but physical features of this kind are only found in the more consolidated types of soils such as shales.

Some of the practical aspects of the shear testing of soils have been described by G. P. Tschebotareff[29], who points out that consideration of this problem falls under two headings: (*a*) tests on undisturbed soils, (*b*) tests on disturbed, remoulded soils. In the case of undisturbed soils, no standardisation of method seems feasible at the present time. The great variety of possible field conditions under which such a soil may be subjected to shearing stresses and deformations, combined with the variety of factors which affect the shearing resistance of soils, makes it inadvisable to impose for such cases any definite method of shear testing.

In the case of any particular type of clay material, the following factors affect its shearing strength :—

1. The " pre-consolidation load," i.e., the greatest previous pressure due to overburden, to drying, or to other causes operating during the geological history of the soil deposit. This load very largely determines cohesion values, although it may be of little interest as such to engineers.

2. The percentage of consolidation which takes place under normal applied loads. The shearing resistance of a clay increases with its density as consolidation proceeds.

3. The rate of application of the shearing force affects the results obtained ; the more rapid the rate of application, the lower the shearing resistance of the sample.

4. The remoulding of clay soils may in some cases reduce their shearing strength, and such clays may or may not regain such strength during the passage of time.

5. The degree of resistance to shear varies with the total deformation of the specimen.

6. The use of the tri-axial shear or compression machine appears likely to yield useful data concerning the principal stresses acting within the sample.

7. The size of the sample may affect the results obtained ; little information is at present available on this point. The method of preparation of the sample may also have some effect.

Modulus of Sub-grade Reaction of a Soil. (k).

The modulus of sub-grade reaction k of a soil is a stiffness coefficient which expresses the resistance of the soil structure to deformation under load, measured in pounds per square inch of pressure per inch of deformation (in the direction of the loading force). This coefficient is used extensively in calculations for the necessary thickness of concrete road slabs, as developed by H. M. Westergaard. A full discussion of the method by L. W. Teller and E. C. Sutherland is available[31]. Three methods have been used for finding k :—

(*a*) Load-deflection tests in which loads are applied at the centre of rigid circular plates of relatively small size, the pressure-intensity on the soil being uniform over the entire area of the plate.

(*b*) Load-deflection tests in which the load is applied at the centre of slightly flexible rectangular or circular plates of relatively large dimensions. In this case, bending of the plate occurs, and the pressure-intensity under the plate is not uniform throughout the area of its contact with the soil.

(*c*) Load-deflection tests on full-size pavement slabs, using Westergaard's deflection formulæ to find k.

The first of these methods is the simplest, but it is necessary to find the effect of the size of the plate to ensure that it is not too small, since the ability of a soil to sustain a given unit pressure varies within limits with the area over which the pressure is applied to the soil. It has already been pointed out that the supporting power of a soil varies with its moisture content, so that it is necessary to ensure that the soil on which the bearing plate is placed is in the same physical state as that which will obtain under the structure which is to be carried by the soil.

Westergaard's theories postulate a homogeneous, isotropic, elastic condition in the soil, but it is known that the soil structure is in fact imperfectly elastic, that the condition of elasticity is affected by moisture content, and that the resistance to load of the soil is dependent on the magnitude of the deformation and on the area over which the load is applied, as mentioned above. These conditions impose limits on the manner in which the coefficient of sub-grade reaction k can be applied. It is, however, thought that for cohesive soils, approximate but usable values for k can be obtained from tests with rigid bearing plates.

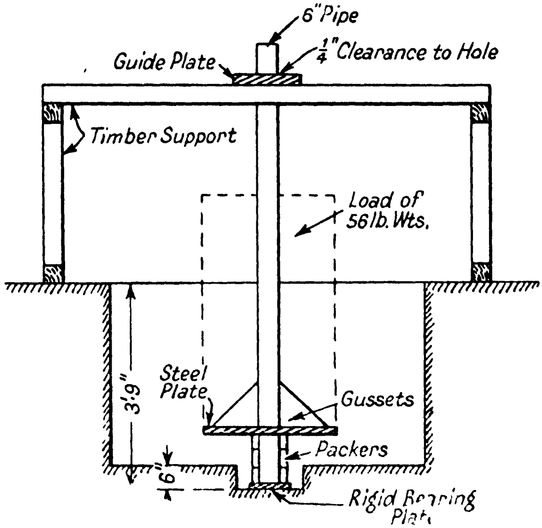


Fig. 83.—Method of making field bearing test on soils.

Further details regarding the theory and practice of the use of bearing plates are given by W. S. Housel[32], while Fig. 83 shows an arrangement for making field bearing tests on soils.

CHAPTER SIX.

CLASSIFICATION OF SOILS.

Objects of Classification of Soils.

The purpose of a soil classification is to provide an accepted and systematic method of describing the various types of soil found in site exploration, conveying concisely all the essential information. This procedure has for its objects :—

1. The elimination of the personal factor in descriptions of soils.
2. The emphasis of the important engineering characteristics of a soil.
3. The recording of soil data in such a way that accepted methods of design of foundations can be readily applied.
4. The avoidance of the use of unsuitable material in foundations.

Most methods of classification of soils used for engineering purposes are based on particle size. Thus gravels, sands, silts and clays form the basic divisions of soil types. The first two of these (the coarse-grained soils) are usually referred to as non-cohesive, while the last two are referred to as cohesive soils. Peat falls into a class by itself, while it is also desirable to distinguish inorganic soils from organic soils.

Soil Classification for Highway Engineering Purposes.

In modern applications of soil mechanics to engineering problems, more progress has been made in soil classification in the branch of highway and aerodrome engineering than in any other branch of civil engineering, possibly because highway foundation problems involve the use of loose or partly consolidated soil, the study of which has proceeded somewhat further than the study of so-called solid elastic soils. The U.S. Corps of Engineers has used work by A. Casagrande shown in Table 10 for its soil classification as applied to aerodrome runways, and in this classification, the nature of any particular type of soil is indicated by simple symbols. Thus gravelly and sandy soils are designated G and S respectively, silty and clayey soils by M and C, while the prefix O refers to organic silts and clays. Each of the two main divisions of coarse-grained soils is further sub-divided into four types denoted by suffixes as follows :—

1. Well-graded, fairly clean material .... W
2. Well-graded material with good clay binder C
3. Poorly graded, fairly clean material .... P
4. Material containing an excess of fines .... F

Well-graded soils contain materials of all sizes, the large sizes predominating ; a poorly graded soil is one possessing a predominance of a single size. Materials containing an excess of fines include such soils as gravels with a high proportion of sand and silt.

The fine-grained soils, including the organic clays, are divided into silts (M), inorganic clays (C), organic clays (O), and are further subdivided into sub-groups of high (H) and low (L) compressibility. The former have a liquid limit exceeding 50 and the latter a liquid limit not exceeding 50.

An older and perhaps more widely used classification is that of the U.S. Public Roads Administration originally put forward in 1929, in which the soil is classified on the basis of grain-size and indicator tests into eight main groups known as Groups A1 to A8. The two schemes may be compared as under :—

U.S. Public Roads Classification.	*U.S. Corps of Engineers Classification.*
A1	GC, SG
A2	GF, SF
A3	GW, GP, SW, SP
A4 and A5	ML, MH
A6	CL, OL
A7	CH, OH
A8	Pt.

The advantages of the newer classification are as follows :—

1. The basis of classification is more logical and more easily used by non-specialists.
2. The soil designation indicates at once the nature of the soil.
3. The classification is more detailed (15 types instead of 8), but the amount of laboratory testing required is less.
4. Visual and manual examination facilitates the classification.

It should, however, be remembered that both of the above schemes of classification were worked out in U.S.A., and may not apply rigidly to countries other than U.S.A.

The Interpretation of Soil Classifications.

It has already been pointed out[32] that the engineering characteristics of a soil can be inferred in a general way from its classification, while Table 11B indicates the extent to which the principal properties of the soil may be inferred[33] from the soil type as found from the classification scheme of Table 10, and gives the ranges of thickness required for flexible pavements which have to carry wheel loads of 10,000 lb., a minimum thickness of 4 inches being assumed. (Table 11B, p. 158).

TABLE 10.—SOIL CLASSIFICATION SCHEME (U.S. CORPS OF ENGINEERS).

Soil Type	Type Prefix	Nature of Soil	Suffix	Designation
Coarse-grained Soils.				
Gravel and gravelly	G	Well-graded, fairly clean	W	GW
		Well-graded, clay binder	C	GC
		Poorly graded, fairly clean	P	GP
		Excess of fines	F	GF
Sands and sandy	S	Well-graded, fairly clean	W	SW
		Well-graded, clay binder	C	SC
		Poorly graded, fairly clean	P	SP
		Excess of fines	F	SF
Fine-grained Soils.				
Silts, very fine sands, rock flour	M	Low compressibility. Silts, rock flour, clayey fine sands	L	ML
		High compressibility. Micaceous or diatomaceous fine sandy and silty soils	H	MH
Clays (inorganic)	C	Low compressibility. Medium plasticity; sandy and silty clays	L	CL
	H	High compressibility. High plasticity; sandy and silty clays	H	CH
Clays (organic)	O	Low compressibility. Organic silts	L	OL
		High compressibility. Organic clays; medium high plasticity	H	OH
Fibrous organic soils.				
Peat		Very high compressibility.		Pt.

The U.S. Public Roads Classification of Soils.

In this system of classification, soils have been classified as to their engineering properties into eight groups designated A1 to A8. This method of classification does not eliminate overlapping, nor does it provide a rigid measure of soil behaviour. Thus some soils may possess some of the characteristics of two groups, so that the engineer should learn to assess the value of different soils used in construction works, and the difficulties arising in their use, rather from the physical constants and their relationship than from the group number of any particular soil. This is seen from the fact that clay soils from different localities classed as A6 or A7 soils may have a wide range of plasticity constants and therefore may have different values for fill and for subgrade construction. The soil classification now to be described can, however, be used to designate characteristics such as plasticity, permeability, bearing power, and resistance to frost heave.

Group A1.

Soils of this group are composed of material well-graded from coarse to fine, mixed with excellent binder; they are very stable under wheel loads irrespective of moisture conditions ; they can be rolled to high densities with either smooth-faced or tamping rollers ; they have practically no volume change under changing moisture conditions. These materials have high bearing capacity at high densities and function satisfactorily when used as bases for relatively thin wearing carpets.

Grading. The soil mortar, i.e., that fraction passing a No. 10 mesh sieve, should be graded as follows:—Clay, 5 to 10 per cent.; silt, 10 to 20 per cent.; fine sand, about 25 per cent.; coarse sand, 45 to 60 per cent.

Constants. The liquid limit is usually greater than 14 and less than 35 ; the plasticity index ranges from 4 to 9 ; the shrinkage limit from 14 to 20 ; the centrifuge moisture equivalent is less than 15 ; the field moisture equivalent is not of significance.

The characteristics of this group of soils are such that the test constants fall into a rather narrow band inasmuch as small variations in grading and binder characteristics result in a soil of the A2 type. Soils of the A1 group are not common and are usually found only in small deposits. They can be used as a base-course for bituminous surfacings if the plasticity index does not exceed 6, and they are very suitable for blanketing materials over less suitable soils.

Group A2.

The soils of this group are made up of coarse and fine materials mixed with binder, but they are inferior in quality to the A1 group on account of poor grading, poor quality binder or both. A2 soils can be compacted with either smooth-faced or tamping rollers, the density obtainable depending on the amount, grading and type of binder. When used as road foundations, A2 materials may be quite stable when fairly dry, but may soften during wet weather or become loose and dusty during dry periods, according to the amount and character of the binder. If used as base-courses, plastic soils of this group may lose stability because of capillary saturation by moisture, or on account of lack of drainage, while some types may be damaged by frost.

Grading. The sand content is not less than 55 per cent.

Constants. The liquid limit is usually less than 35 ; the plasticity index may vary from zero to 15 according to the amount and grading of the binder. The shrinkage limit does not usually exceed 25, and is of significance only with unsatisfactory binders. The centrifuge moisture equivalent does not exceed 25.

Soils falling in the group are of quite common occurrence. The group is usually divided into two types, i.e., the plastic and the friable types. The plasticity index of the plastic type ranges from 3 to 15 ; when it exceeds 6, the soil is not suitable as a base-course for even light bituminous surfacing, and may cause warping of concrete pavements if large changes of moisture content occur. The friable type usually has a plasticity index ranging from zero to less than 3, and can be used as a base-course material for bituminous surfaces where a moisture content sufficient to ensure adequate stability can be maintained, or where the soil is completely confined.

Soils of this group, either plastic or friable, can be considered as stable if well compacted, and are satisfactory for the construction of fills or the blanketing of plastic or silty soils. They are easily drained, but may have sufficient plasticity to give rise to detrimental volume changes. Bituminous materials, portland cement and other admixtures can be used with soils of this group with comparative ease to give stabilised soil foundations (see Chapter 9).

Group A3.

The soils of this group are composed entirely of coarse materials such as sand and gravel ; they lack stability under wheel load except when damp ; they have no deleterious volume change under changing moisture contents, but collapse entirely when approaching the dry state. They cannot be compacted by flat-wheel rollers, but only by sheepsfoot rollers or preferably by vibration methods, water up to optimum moisture content being used for the purpose. They drain rapidly, and when adequately confined, make suitable sub-grades for all types of pavement.

Grading. The fraction passing the No. 200 mesh sieve is less than 10 per cent.

Constants. Soils of this group have no plasticity. The shrinkage limit and field moisture content have no significance. The centrifuge moisture equivalent does not exceed 12.

A3 soils are of wide distribution, and many of them can be stabilised successfully with portland cement, tar or bitumen for use as stabilised soil foundations.

Soils of Fourth Group Subject to Frost Heave.

Group A4.

This group consists predominantly of silt soils containing only moderate to small amounts of coarse material and of sticky colloidal clay. When fairly dry or merely damp, A4 soils form a firm riding surface which rebounds but little on removal of load, but when water is absorbed rapidly, they may expand detrimentally and lose stability. They are subject to frost heave.

The soils in this group vary widely in textural composition and range, from the sandy loams at the coarse end of the scale to silt and clay loams at the finer end. The sandy loams can be rolled to comparatively high densities with either tamping or smooth-faced rollers and have high stability through a wider range of densities than have the silt and clay loams. Their volume changes are only small and do not produce severe pavement warping even if compacted in the dry state. The silts and silt loams of the group cannot be rolled to high densities on account of the high voids-ratio resulting from inferior grading and because of a lack of binder material. They are relatively unstable at all moisture contents, but especially at the higher moisture contents, when their stability is very low. Silts and some silt loams are not easy to consolidate, since the best rolling results can be obtained only through a very narrow range of moisture content ; i.e., the optimum moisture content is critical for this type of soil ; at this content, maximum compaction can be obtained with a smooth-faced roller. If consolidation be attempted at a moisture content above or below the optimum, the soil will bulge up ahead of and behind the roller, resulting in corrugations and non-uniform compaction.

The clay loams of this group are somewhat better graded than the silts and can be rolled to higher densities. Tamping rollers have proved more effective than smooth-faced rollers on heavy clay loams. The clay loams are stable at low moisture contents and high densities, but are liable to show detrimental volume changes as the moisture content rises.

Grading. The sand content is less than 55 per cent.

Constants. The liquid limit of soils in this group varies from 20 for sandy loams to 40 for clay loams, while the plasticity index varies from zero for coarse silts with no binder to 15 for clay loams. The shrinkage limit varies from 20 for the better graded sandy clay loams with good binder to 30 for silts. The centrifuge moisture equivalent varies from 12 to 50 according to the porosity and permeability of the soil. The field moisture equivalent does not exceed 30. When the centrifuge moisture equivalent exceeds the liquid limit, soils in this group are likely to be especially unstable in the presence of water. In any case, Group A4 soils are likely to be highly expansive if moisture changes occur, and to approach the A5 group when the field moisture equivalent exceeds the centrifuge moisture equivalent, and when the shrinkage limit exceeds 25.

The wide range of soils in this group extends from those which border the A2 group to those which approach the lower limit of the A5, A6 and A7 groups, and the border-line soils are often designated A4(2), A4(5), etc., indicating that they approach the latter group in characteristics, grading and values of test constants. Since the soils in this group are subject to frost heave, they should be covered with granular materials in areas where extremely low temperatures may occur and conditions conducive to frost heave exist. The thickness of cover necessary to prevent frost heaving varies from 18 to 48 inches. When wet, these soils may become elastic and show considerable rebound on removal of load.

The more plastic types in the group expand with increases in moisture content to an extent sufficient to cause warping at the joints of concrete slabs if the soils are placed at moisture contents lower than the optimum, while bituminous surfacings require substantial base courses when placed on subgrades consisting of any of the varieties of this group.

Soils of the Fifth and Sixth Groups unsuitable as Sub-grades for Thin, Flexible Type Base Courses.

Group A5.

This group is similar to the A4 group except that it includes very poorly graded soils containing materials such as mica and diatoms which are productive of elastic properties and very low stability. Soils of this group are likely to be elastic and to rebound upon removal of load even when dry, and these elastic properties interfere very considerably with the proper compaction of flexible-type base courses during construction and with the retention of good bond afterwards.

Grading. The sand content is less than 55 per cent., although exceptions to this may occur.

Constants. The liquid limit usually exceeds 35. The plasticity index usually ranges from 0 to 20, but in some cases may be as high as 60. The shrinkage limit is greater than 30 and less than 120, but usually exceeds 50 for the undesirable soils of the group ; the field moisture equivalent varies from 30 to 120.

The soils in this group are unsuitable for use as sub-grades for thin stabilised base courses under bituminous surfaces. They are subject to frost heave, and should be covered with granular materials when met with in areas subject to very low temperature conditions. They are usually difficult to compact because of their tendency to rebound upon removal of load, and it has been observed that pavements laid over sub-grade soils of this group crack excessively.

Group A6.

This group is composed of predominantly clay soils with moderate to negligible amounts of coarse material. In the stiff or soft condition, they absorb water only when manipulated. They can be compacted to relatively high densities by the use of heavy tamping rollers ; they possess good bearing capacity when compacted to their maximum practicable density ; they are compressible and rebound very little on removal of load ; they are highly expansive on increase of moisture content and productive of severe warping in concrete and bituminous surfacings if placed in a condition dry enough to permit of the absorption of large quantities of moisture.

Grading. The sand content is less than 55 per cent.

Constants. The liquid limit exceeds 35, the plasticity index is greater than 18, the shrinkage limit is less than 14, and the field moisture equivalent is less than 50.

The high plasticity indexes of this group of soils show the very cohesive nature of the binder material (clay and colloids) at the lower moisture contents. This cohesion decreases as the moisture content increases, and thus since such soils possess little internal friction, their stability is low at the higher moisture contents. They are consequently suitable only for use in fills and as sub-grades when they can be placed and maintained at relatively low moisture contents.

The very low shrinkage limits possessed by soils of this group are indicative of high volume change consequent on changes in moisture content, while the high shrinkage ratios indicate that the capillary pressure exerted while evaporation proceeds is of such an intensity as to compress the soil particles into a compact, dense mass. In the field, group A6 soils show shrinkage cracks on all surfaces exposed to drying.

The centrifuge moisture equivalent usually exceeds 25, and the high values obtained in this test indicate that water moves very slowly through soils of this group, even when under a considerable head. Thus such soils will take up water very slowly unless manipulated, and conversely, when once wet, they dry out very slowly. The flow of gravitational water through them is negligible, so that ordinary drainage methods are of little value.

Soils of the A6 group are confined in their general characteristics within closer limits than are those of either the A4 or A7 groups. Borderline soils are often designated as A6(4) or A6(7) soils. Soils of this group are not suitable for use as subgrades under thin flexible base-courses or bituminous surfaces on account of the large volume changes caused by moisture fluctuations and the loss of bearing power which occur simultaneously. When concrete slabs are placed over such soils, the sub-grades should be blanketed with a non-expansive material such as an A1 or A2 soil (friable type), or other insulating material.

Group 7. Soils to be used with care.

Group A7.

Soils of this group are similar to those of the A6 group except that at certain moisture contents they are elastic and deform quickly under load, rebounding appreciably on removal of load. This characteristic results from an inferior grading (steep grain-size curve throughout the silt fraction), extraneous matter such as organic material, mica flakes, lime carbonate, variations in grain shape, or dangerous clay fraction (see Chapter 3).

Alternate wetting and drying of A7 soils under field conditions leads to rapid and detrimental volume changes.

Soils of this group are more difficult to compact by rolling than those of the A6 group ; heavy tamping rollers are best for the purpose. Soils of this type have high bearing capacities when compacted to high densities, but are subject to excessive volume change unless properly compacted at a moisture content sufficiently high to ensure minimum air voids. They have produced severer warping of concrete slabs than have soils of any other groups.

Grading. The sand content is less than 55 per cent.

Constants. The liquid limit for soils of this group is greater than 35 and the plasticity index exceeds 12. The shrinkage limit may vary from 10 to 30, while the field moisture equivalent may vary from 30 to 100. The major difference between soils of the A7 and A6 groups is in their elasticity. This property is indicated by the higher shrinkage limits and field moisture equivalents associated with soils of the A7 group. These higher constants may be due to poor grading or to poor quality binder.

Like the A4 group, the A7 group covers a wide range of soils varying from those bordering on the A4 or A5 groups of silts and loams and the A6 groups of clays to those approaching the lower limits of the A8 group which contain excessive organic material. Such border-line soils may be designated A7(4), A7(8), and so on. Since the soils of this group are even more expansive than those of the A6 group, the same precautions in their use should be observed. In view of their elastic and rebound tendencies, they should be compacted with great care if used as sub-grades for concrete slabs, and should not be subjected to excessive loading immediately ahead of paving, if early cracking of the slab due to rebound is to be avoided. It is of course always preferable to use a blanket course of better quality material in such cases. In areas subject to low temperatures, frost heave may be expected in such soils.

Group 8. Soils to be avoided.

Group A8.

The soils in this group are composed of very soft peat and muck. They contain excessive quantities of organic material and of moisture, and are obviously unsuited for use in sub-grades or fills.

Grading. The grading has no significance.

Constants. The liquid limit varies from 35 to 400, the plasticity index from 0 to 60, but is usually less than 25 ; the shrinkage limit varies from 30 to 120, and the field moisture equivalent from 30 to 400, these latter figures indicating the presence of partly decomposed organic matter.

The tendency towards the retention of large amounts of capillary moisture far above the normal water table makes this group of such a type that its use in any form of engineering construction should be avoided.

The foregoing notes are summarised in Table 11A.

In addition to the results of the indicator tests already described, the density-moisture relations of soils in the compacted state are indicative of their value for fill construction and as foundation materials generally. The maximum dry weight is highest for granular well-graded soils in the A1 group and decreases to a minimum for those in the A8 group.

Since the results of indicator tests have been correlated with the service behaviour of soils in highway construction, it is possible to estimate the required thickness of sub-base, base-course and surfacing required for any type of soil, and these thicknesses are given in Table 11B.

It should be noted, however, that the thicknesses referred to were obtained by observation and not by laboratory or other purely scientific approach. The values are the result of the experience of highway engineers concerned with the most appropriate use of soil materials in road and aerodrome foundations and may be used with confidence. The thicknesses given will vary with variations in the soil constants, in the degree of soil compaction obtained, in the natural soil moisture and in local climatic conditions. Thus a soil of the A6 group with a plasticity index of 20 and a natural moisture content of 18 per cent. will require less thickness than another A6 soil with a plasticity index of 50 and a natural moisture content of 30 per cent. When used in a dry climate and where the distance to ground water is great, the first of these two soils can be thinner than in a locality in which the ground water is high. It is clear, therefore, that some degree of judgment on the part of the engineer is required. The foregoing notes should be read in conjunction with Chapter 7 (Drainage and Compaction of Soils).

An abbreviated and modified soil classification based on that given above is available as follows[34]:—

Granular Soils.

*A*1 *Materials.* These are well-graded predominantly granular materials seldom occurring as such in natural deposits. They are ordinarily produced by combining gravel or sand or both with suitable quantities of fine material to yield stable mixtures. When the fine-soil fraction contains enough clay to become plastic, the mixture is classified A1a. When the fine-soil fraction contains so little clay as to lack plasticity but consists principally of fine sand or silt, it is classified as A1b.

TABLE 11A.—SUMMARY OF SOIL CHARACTERISTICS AND CLASSIFICATION.

GROUP	A-1	A-2		A-3	A-4	A-5	A-6	A-7	A-8
		Friable	Plastic						
General stability properties	Highly stable at all times	Stable when dry may ravel	Good stable material	Ideal support when confined	Satisfactory when dry, loses stability when wet or by frost action	Difficult to compact, stability doubtful	Good stability when properly compacted	Good stability when properly compacted	Incapable of support
Physical Constants.									
Internal friction	High	High	High	High	Variable	Variable	Low	Low	Low
Cohesion	”	Low	”	None	”	Low	High	High	”
Shrinkage	Not detrimental	Not significant	Detrimental when poorly graded	Not significant	”	Variable	Detrimental	Detrimental	Detrimental
Expansion	None	None	Some	Slight	”	High	High	”	”
Capillarity	”	”	”	”	Detrimental	”	”	High	”
Elasticity	”	”	”	None	Variable	Detrimental	None	”	”
General Grading Limits	Uniformly graded coarse—fine, excellent binder	Poor grading, poor binder	Poor grading, inferior binder	Coarse material only, no binder	Fine sand cohesionless silt and friable clay	Micaceous and diatomaceous	Deflocculated cohesive clays	Drainable flocculated clays	Peat and muck

TABLE 11A.—*Continued.*

Textural Classification.									
Approximate limits :									
Sand, per cent.	70–85	55–80	55–80	90–100	55 max.	55 max.	55 max.	55 max.	55 max.
Silt, per cent.	10–20	0–45	0–45	(*)	High	Medium	Medium	Medium	Not significant
Clay, per cent.	5–10	0–45	0–45	(*)	Low	Low	30 min.	30 min.	,,
Physical Characteristics.									
Liquid limit	14–35**	35 max.	35 max.	NP***	20–40	35 min.	35 min.	35 min.	35–400
Plasticity index	4–9**	NP–3***	3–15	NP***	0–15	0–60	18 min.	12 min.	0–60
Field Moisture Equivalent	Not essential	Not essential	Not essential	Not essential	30 max.	30–120	50 max.	30–100	30–400
Centrifuge Moisture Equiv,	15 max.	12–25	25 max.	12 max.	Not essential	Not essential	Not essential	Not essential	Not essential
Shrinkage limit	14–20	15–25	25 max.	Not essential	20–30	30–120	6–14	10–30	30–120
Shrinkage ratio	1·7–1·9	1·7–1·9	1·7–1·9	,,	1·5–1·7	0·7–1·5	1·7–2·0	1·7–2·0	0·3–1·4
Volume change	0–10	0–6	0–16	None	0–16	0–16	17 min.	17 min.	4–200
Lineal shrinkage	0–3	0–2	0–4	,,	0–4	0–4	5 min.	5 min.	1–30
Compaction characteristics.									
Max. dry weight per cu. ft.	130 min.	120–130	120–130	120–130	110–120	80–100	80–110	80–110	90 max.
Optimum moisture, per cent. of dry weight (approx.)	9	9–12	9–12	9–12	12–17	22–30	17–28	17–28	
Minimum field compaction required, per cent. of maximum dry weight per cu. foot	90	90	90	90	95	100	100	100	
Rating for fills 50 feet or less in height	Excellent	Good	Good	Good	Good to poor	Poor to very poor	Fair to poor	Fair to poor	Unsatisfactory
Rating for fills more than 50 feet in height	Good	Good to fair	Good to fair	Good to fair	Fair to poor	Very poor	Very poor	Very poor	,,

* Percentage passing No. 200 sieve, 0 to 10.
** When used as a base course for thin flexible surfaces the plasticity index and liquid limit should not exceed 6 and 25 respectively.
*** NP stands for non-plastic.

TABLE 11B.—SOIL CLASSIFICATION AND ITS INTERPRETATION

	Major Divisions	Type of Soil	U.S. Classification	U.S. Corps of Engineers	Identification: Dry strength	Identification: Other factors	Chief Indicator Tests	Value as foundation (no frost action)	Potential frost action	Shrinkage expansion elasticity	Drainage characteristics	Compaction characteristics and equipment	* Max. dry density lbs. cu. ft.	Voids ratio e	Thickness for 10,000 lb. wheel load ins.
Coarse-grained Soils.	GRAVEL AND GRAVELLY SOILS	Well-graded gravel and gravel-sand mixtures: little or no fines	A-3	GW	None	Gradation	Mechanical analysis	Excellent	None to very slight	Almost none	Excellent	Excellent tractor	> 125	$e < 0{\cdot}35$	4
		Well-graded gravel-sand. Clay mixtures, excellent binder	A-1	GC	Medium to high	Gradation binder, wet and dry	Mechanical Analysis. Liquid and plastic limits	Excellent	Medium	Very slight	Practically impervious	Excellent. Sheeps foot roller	> 130	$< 0{\cdot}30$	4
		Poorly graded gravel and gravel-sand mixtures, little or no fines	A-3	GP	None	Gradation	Mechanical analysis	Good to excellent	None to very slight	Almost none	Excellent	Good. Tractor	> 115	$< 0{\cdot}45$	4
		Gravel with fines, very silty gravel, clayey gravel, poorly graded sand-clay mixtures	A-2	GF	Very slight to high	Gradation binder, wet and dry	Mechanical Analysis Liquid and plastic limits	Good to excellent	Slight to medium	Almost none to slight	Fair to practically impervious	Good: close control essential. Pneumatic tyred roller : Tractor	> 120	$< 0{\cdot}40$	4–5
	SANDS AND SAND-CLAY SOILS	Well-graded sands, gravelly sands. Little or no fines	A-3	SW	None	Gradation	Mechanical analysis	Good to excellent	None to very slight	Almost none	Excellent	Excellent Tractor	> 120	$< 0{\cdot}40$	4–5
		Well-graded sand-clay mixtures. Excellent binder	A-1	SC	Medium to high	Gradation Binder wet and dry	Mechanical analysis. Liquid and plastic limits	Good to excellent	Medium	Very slight	Practically impervious	Excellent. Sheeps foot roller	> 125	$< 0{\cdot}35$	4–5
		Poorly graded sands: little or no fines.	A-3	SP	None	Gradation	Mechanical analysis	Good to fair	None to very slight	Almost none	Excellent	Good. Tractor	> 100	$< 0{\cdot}70$	4–11
		Sand with fines, very silty sands, clayey sands, poorly graded sand-clay mixtures	A-2	SF	Very slight to high	Gradation	Mechanical analysis. Liquid and plastic limits	Good to fair	Slight to high	Almost none to medium	Fair to almost impervious	Good; close control essential. Pneumatic tyred roller	> 105	$< 0{\cdot}60$	4–12

* Applicable only to soils S.G. 2·65—2·75.

TABLE 11B.—*Continued.*

Fine-grained Soils.	FINE GRAINED SOILS WITH LOW TO MEDIUM COMPRESSIBILITY	Silts (inorganic) and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	A-4 A-6 A-7	ML	Very slight to medium	Plasticity	Mechanical analysis. Liquid and plastic limits	Fair to poor	Medium to very high	Slight to medium	Fair to poor	Good to poor: close control essential. Pneumatic tyred roller	> 100	< 0·70	4–14
		Clays (inorganic) of low to medium plasticity, sandy clays, silty clays, lean clays	A-4 A-6 A-7	CL	Medium to high	Plasticity	Liquid and plastic limits	Fair to poor	Medium to high	Medium	Practically impervious	Fair to good: Sheeps foot roller	> 100	< 0·70	7–18
		Organic silts and silt clays of low plasticity	A-4 A-7	OL	Slight to medium	Plasticity. Odour	Liquid and plastic limits	Poor	Medium to high	Medium to high	Poor	Fair to poor. Sheeps foot roller	> 90	< 0·90	14–20
	FINE-GRAINED SOILS WITH HIGH COMPRESSIBILITY	Micaceous or diatomaceous. Fine sandy and silty soils, elastic silts	A-5	MH	Very slight to high	Plasticity	Mechanical analysis. Liquid and plastic limits	Poor	Medium to very high	High	Fair to poor	Poor to very poor	> 100	< 0·70	13 upwards
		Inorganic clays of high plasticity, fat clays	A-6 A-7	CH	High	Plasticity	Liquid and plastic limits	Poor to very poor	Medium	High	Practically impervious	Fair to poor. Sheeps foot roller	> 90	< 0·90	14 upwards
		Organic clays of medium to high plasticity	A-7 A-8	OH	High	Plasticity. Odour	Liquid and plastic limits	Very poor	Medium	High	Practically impervious	Poor to very poor	> 100	< 0·70	18 upwards
Fibrous organic soils with high compressibility		Peat and other highly organic swamp soils	A-8	Pt	—	—	Natural water content	Extremely poor	Slight	Very high	Fair to poor	Compaction impossible	—	—	14 upwards

When compacted, both classes have high inherent supporting value. The A1b type may be used as a base-course material. The A1a type makes a good road surfacing material (for very lightly trafficked roads), but on account of its capillarity and higher plasticity may lose its supporting value to a considerable extent when overlaid with an impervious wearing course. If it occurs as a natural sub-grade, or is used in construction, it will ordinarily require an overlying base-course; it should therefore be considered primarily as a sub-base material.

A2 Materials. These are essentially granular soils containing a higher percentage of material passing the No. 200 mesh sieve than A1 materials. They have high supporting value when dry, but because of their greater capillarity, less supporting value when wet than A1 materials. They are divided into two classes according to the character of the fine-soil fraction. In A2a materials, the fine-soil fraction is essentially silty and of low plasticity, while in A2b materials it is plastic. A2a soils are very similar to A3 materials but have greater capillarity. As a rule they require no sub-base course. The A2b soils, when used as sub-grades, require a sub-base course where water and frost conditions occur.

A3 Materials. These are gravel and sand, containing little or no silt or clay; they are highly permeable and cohesionless. When thoroughly compacted, they can fail only in shear, in which case the granular material immediately adjacent to the loaded area must expand, and in order to prevent such expansion, a base-course is required to exert a stabilising influence. These materials are easily drained because of their high permeability.

Fine Soils.

These soils are characterised by one or more of the following properties, making necessary substantial thicknesses of sub-base construction: High capillarity, high plasticity when wet, high shrinkage, high elastic rebound after compression. They usually require thicker bases than the granular soil groups, and their proposed classification is as follows:

A4 Materials. Relatively cohesionless silty soils susceptible to frost action and indicative of the need for adequate drainage.

A4(7) Materials. Moderately plastic silt and clay soil materials, mixtures usually susceptible to frost action and to softening due to moisture penetration.

A5 Materials. Similar to A4 materials, but susceptible to elastic rebound due to the presence of mica or diatoms.

A5(7) Materials. Similar to A4(7) materials, but susceptible to elastic rebound.

A6 Materials. Highly plastic colloidal clay soils subject to detrimental volume changes on change of moisture content and to softening.

A7 Materials. Plastic clays with low permeability, unlikely to be adversely affected by frost action, but subject to softening and volume change due to moisture fluctuations.

A8 Materials. Peat or muck, with very low supporting value, unsuitable for normal engineering purposes.

More detailed classifications are given in Tables 12, 13 and 14.

TABLE 12.—DESIGNATION AND USES OF GROUP A1 SOILS.

	A1a		A1b			
Characteristics.	Granular type road surface. Good sub-base and sub-grade and fill. Liable to frost damage; does not drain well.		Granular type base-course. Excellent sub-base, sub-grade and fill; may not drain well; no frost effect.			
Sieve analysis per cent. passing	Type A	Type B	Type A*	Type B		
				1″ max.	2″ max.	3″ max.
3 inch					—	100
2 ,,					100	65—100
1½ ,,					70—100	—
1 ,,		100		100	55—85	45—75
¾ ,,		85—100		70—100	50—80	—
⅜ ,,		65—100		50—80	40—70	30—60
4		55—85		35—65	30—60	25—50
10	100	40—70	100	25—50	20—50	20—40
20	55—90	—	50—90	—	—	—
40	35—70	25—45	35—70	15—30	10—30	10—25
200	8—25	10—25	8—25	5—15	5—15	3—10
†Dust ratio	Less than two-thirds		Less than two-thirds			
Characteristics of fraction passing 40						
Liquid limit	35 maximum		25 maximum			
Plasticity index	4 to 9		6 maximum			

* Sand-clay mortar; 100 per cent. should pass the 1″ sieve and 65–100 per cent. the No. 10 sieve. The sieve analysis shown is for the material passing the No. 10 sieve.

† = total passing No. 40 sieve ÷ total amount of soil.

TABLE 13.—DESIGNATION AND USES OF GROUP A2 and A3 SOILS.

	A2		A3	
	A2a	A2b	A3a	A3b
Characteristics	Excellent for sub-base, sub-grade and fill. May not drain freely; no frost damage.	Good for sub-base, sub-grade and fill. Does not drain well; liable to frost damage.	For drainage course. Excellent sub-base and sub-grade; not liable to frost damage.	For drainage course except at contact with fine-grained soil; excellent sub-base and sub-grade; not affected by frost.
Sieve analysis, per cent. passing				
No. 200 sieve	Less than 35	Less than 35	35 min.	—
Capillary rise*	36″ maximum	Less than 36″	2″—6″	6″ maximum
Coefficient of permeability†	—	—	2100—310 feet per day	310 feet per day minimum
Liquid limit Plasticity index	—	40 max. 15 max.	—	—

* For method of determination of capillary rise see Chapter Five.
† For method of determination of coefficient of permeability see Chapter Five.

TABLE 14.—DESIGNATION AND USES OF GROUPS A4 TO A7 SOILS.

Proposed Class Designation	Character of Soil.	Characteristics of fraction passing No. 40 mesh sieve.	
		Liquid limit = LL	Plasticity Index = PI
A4	Feebly plastic or non-plastic silty soils. Sub-base needed for drainage and prevention of frost heave. Makes very stable fill, but likely to erode on slopes.	40 maximum	Less than 20 per cent. of LL.
A4(7)	Moderately plastic silt loam soils ; sub-base needed for prevention of loss of stability during thaws. Makes stable fills.	40 maximum	Less than 40 per cent. of LL with minimum of 20 per cent. of LL.
A5	Feebly or non-plastic silty soils with elastic rebound on removal of load. Subject to damage by frost, and worst varieties may require thick sub-bases to offset elastic properties.	Exceeding 40	Less than 20 per cent. LL.
A5(7)	Moderately plastic silt loam soils. May be subject to damage by frost. Presence of clay makes rebound less dangerous than in A5 materials.	Exceeding 40	Less than 40 per cent. LL with minimum of 20 per cent. of LL.
A6	Highly plastic colloidal clay soils. Sub-base needed to offset volume change and softening of the sub-grade. Caution should be used if in fills. No danger of damage by frost.	—	60 per cent. LL minimum.
A7	Plastic clay loam soils. Sub-bases afford means of prevention of warping of rigid pavements and softening of sub-grade detrimental to flexible pavements. Not likely to be damaged by frost.	—	Less than 60 per cent. LL. with 40 per cent. LL minimum.

CHAPTER SEVEN.

DRAINAGE AND COMPACTION OF SOILS.

Drainage of Soils.

The drainage of a soil falls under three headings: (a) Disposal of surface water, (b) Removal of subsoil water from site, (c) Interception and diversion of subsoil and surface water draining from land adjoining site. In soil mechanics, our chief concern is the behaviour of subsoil water.

Very little fundamental work on the movement of subsoil water through soils is available. It has, however, been shown by E. C. Childs[35] that the mathematical analysis of field drainage is so far impossible, but that some progress may come from electrical analogy, since the equation of the flow of ground water to parallel drain lines is also the equation of the two-dimensional flow of electricity in a sheet conductor. Experiments have shown (*i*) a fall in the water-table with increase in drain diameter, (*ii*) an almost linear relationship between rainfall rate and height of water-table, (*iii*) the influence of the depth of the impermeable floor below the drains.

Bore-holes and other excavations may cause appreciable disturbance of the water-table, and lowering the level of the water in drains has a similar effect, lowering its height at the mid-point by an approximately equal amount. It should be noted that other things being equal, an open ditch is a more efficient drain than the same ditch piped and filled in.

Work by L. Mamanina[36] on the effect of insulating layers on the rise of capillary moisture in heavy loams has shown that of the three types of insulating material studied (gravel, sand and compacted loam) the best results were with coarse sand (0·09 inches to 0·02 inches size), while a layer of this sand 0·8 inch thick gave complete insulation. A 1½ inch layer of gravel had an appreciable insulating effect if the material was carefully cleaned, otherwise a considerable thickness was required, as was also the case with medium sand. Layers of compacted loam could not prevent the rise of capillary moisture, but could hinder and delay it.

Soil moisture may be either stationary, gravitational or capillary water. The former can be drained, but the latter two cannot; in fact, capillary heads can rise to a theoretical height of as much as 23 feet although such heights are not met with in practice, one recorded height of 3 feet in a silty soil being regarded as a very high figure.

The purpose of subsoil drainage is to provide a uniform load-bearing soil below structures, and to reduce frost damage in the case of sub-grades beneath roads and aerodromes. It is therefore advisable to lower the water-table or to raise the structure until the capillary fringe is at least a foot below the surface of any soil which is likely to undergo volume changes or deleterious softening in the presence of added moisture.

The quantity of water reaching the subsoil drains can be estimated by subtracting the surface run-off, the evaporation losses and the water taken up by vegetation from the rainfall falling on the surface.

Subsoil drains can only function at or below the water-table. Detailed observations of the height of the water-table at different seasons of the year should be made in order to determine the various heights which have to be reckoned with at any particular site.

Sub-grade moisture conditions beneath airport pavements have been studied by M. S. Kersten[37], who found that moisture conditions varied with soil texture and with climate, increasing with fineness of texture from sands at the one end to clays at the other. Many clays were found to be wetter than their plastic limit, while a majority of the soils other than sands were wetter than their optimum moisture content. As would be expected, a direct connection was found to exist between climatic conditions and moisture content, while rigid pavements covered higher moisture contents than flexible pavements in the same type of soil. In arid regions, the variations of moisture content in the upper 3 feet of sub-grade were slight.

The Compaction of Soils.

The following are the factors influencing the compaction of soils:— (*a*) The type of soil, (*b*) The moisture content of the soil, (*c*) The amount of compaction applied. A further factor not generally referred to is the method of compaction.

TABLE 15.—RECOMMENDED DEPTHS AND SPACING OF SUBSOIL DRAINS FOR VARIOUS CLASSES OF SOIL.[38]

Soil Class	Mechanical analysis of soil per cent.			Distance between drains in feet	
	Sand	Silt	Clay	2—3 feet deep	3—4 feet deep
Sand	80—100	0—20	0—20	100—150	150—300
Sandy loam	50—80	0—50	0—20	85—100	100—150
Loam	30—50	30—50	0—20	75—85	85—100
Silty loam	0—50	50—100	0—20	65—75	75—85
Sandy clay loam	50—80	0—30	20—30	55—65	65—75
Clay loam	20—50	20—50	20—30	45—55	55—65
Silty clay loam	0—30	50—80	20—30	40—45	45—55
Sandy clay	50—70	0—20	30—50	35—40	40—45
Silty clay	0—20	50—70	30—50	30—35	35—40
Clay	0—50	0—50	0—50	30—35	50—100

It is to be assumed that the figures given in Table 15 are based on experience and not on scientific experiment or deduction.

Effect of Moisture Content.

When the moisture content of a soil is low, the material is stiff and difficult to compress, and the dry soil density is low. When the moisture content is increased, the water lubricates the soil grains, the soil softens and becomes plastic and easier to compress, and the dry soil density increases. In addition, the air content is reduced, but the air in the voids becomes increasingly difficult to expel, since moisture and air together tend to keep the soil particles apart and to prevent any further increase in dry soil density ; maximum density is obtained at this point. Thereafter, any fresh increments of moisture force the soil particles apart, and since the specific gravity of water is less than that of soil, the dry density of the material is reduced. The curve of dry density approaches the saturation line as the moisture content reaches its optimum value, but never reaches it, since it is impossible to expel all the air entrapped in the soil voids by ordinary means of compaction.

H. C. Porter[39] has shown that the permanence of compaction is affected by both the moisture content at the time at which additional water is brought into contact with the soil, and also by the rapidity with which the additional water is brought into contact with the soil. For these reasons, loss of moisture from freshly compacted soil structures should be prevented by covering them with finishing coats as rapidly as possible after compaction ; they should also be protected as far as possible from sudden inundation.

Effect of Amount of Compaction.

Work by the Road Research Laboratory has shown that any increase in dry soil density due to heavier compaction is accompanied by a decrease in the optimum moisture content ; thus the lower curve in Fig. 84 shows the moisture-density relation for a soil compacted with

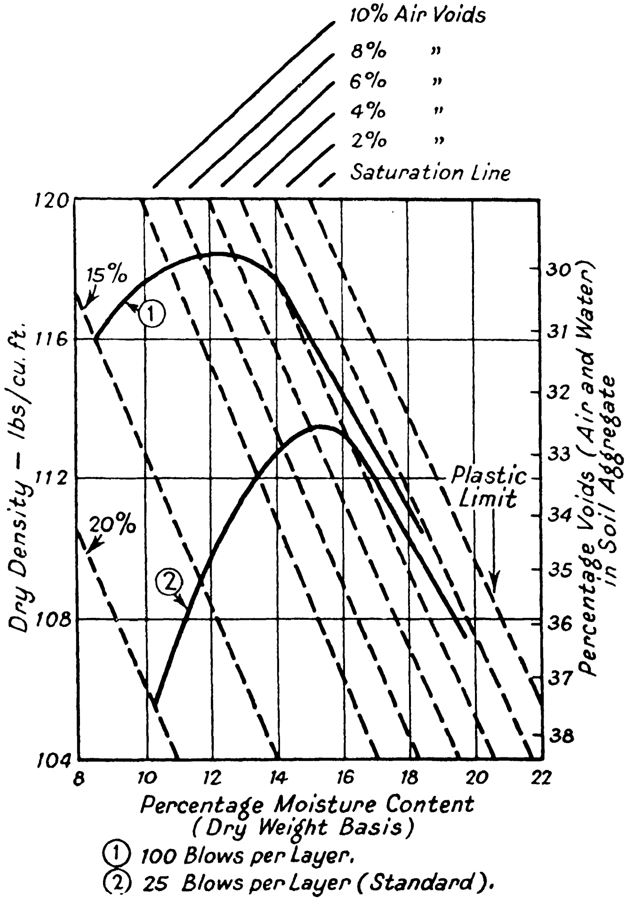


Fig. 84.—Showing effect of varying amounts of compaction on a brick-earth soil. (Road Research Laboratory.)

25 blows per layer, while the upper curve shows the corresponding relationship for 100 blows per layer. Other investigators (see Fig. 85) have obtained similar results. Thus C. A. Hogentogler[40] has found a linear relationship between the number of blows per layer and maximum density for ranges of 10 to 35 blows per layer, while the same worker[41] used a logarithmic scale of moulding pressures which gave a

linear relationship between maximum density, optimum moisture content and the logarithm of the moulding pressure (see Fig. 86).

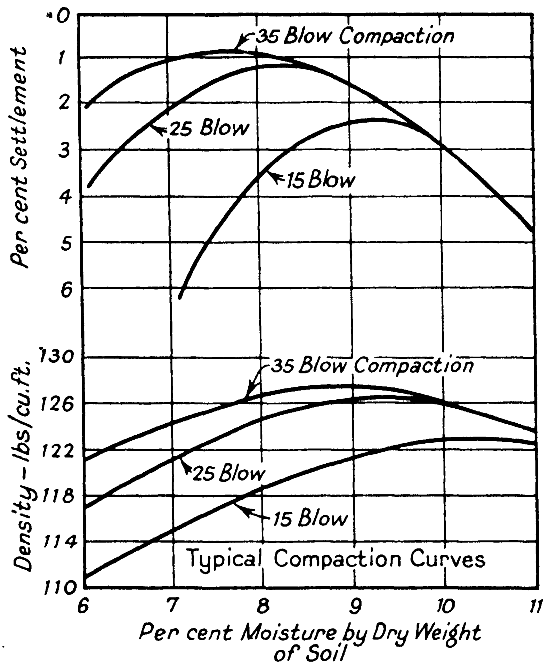


Fig. 85.—Showing effect of degree of compaction and of moisture content on amount of settlement. (Allen.)

From previous references in this book, it should be clear that the object of soil compaction is to increase shear strength, reduce compressibility, decrease water absorption and obtain a material less subject to weathering. Typical figures for dry sand show the connection between stability and density as follows :—

Density (lb. cub. ft.)	*Voids ratio (e).*	*Angle of internal friction ϕ.*
100	0·56	47°
94	0·75	32°—34°

These figures indicate that a thoroughly compacted sand is about twice as strong as a loosely compacted sand.

The effect of compaction on cohesive soils has been studied by S. J. Buchanan[42] (see Fig. 87), who found that the shear strength of the soil increases with the amount of compaction, but approaches a limit where further compaction adds little to the shear strength, suggesting that shear strength and density are closely related. The figure shows that the vertical interval between successive curves increases with increase in the amount of compaction, indicating an increase in the angle of internal friction ϕ. The effect of compaction on cohesion has been inferred by difference, and is shown by the dotted curve in Fig. 87. Increased compaction, therefore, increases both the cohesion and the angle of internal friction of a soil.

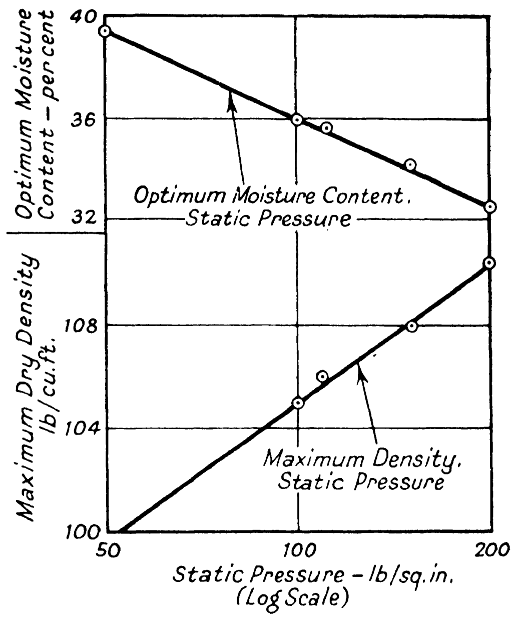


Fig. 86.—Showing effect of static pressure on maximum density and optimum moisture content. (Hogentogler.)

N. W. McLeod[43] has given results emphasising the importance of consolidating mechanically stabilised gravels to a high degree of density. Thus, using 6 inch diameter cylinders 6 inches high, it was found that crushing strengths increased from 225 to 350 lb. per sq. in. for an increase in dry density from 135 to 145 lb. per cubic foot.

Work by the Road Research Laboratory has shown that in the case of a highly plastic clay, a Proctor density of 95 lb. per cubic foot was increased by modified Proctor compaction to 117 lb. per cubic foot, the optimum moisture content being reduced by 10 per cent., corresponding figures for a sandy and silty clay being: Proctor density 115 lb. per cubic foot, modified Proctor density 125 lb. per cubic foot, and reduction in optimum moisture content 3 per cent.

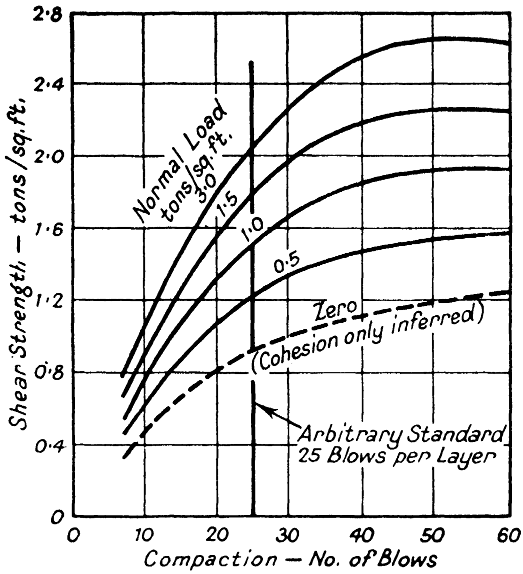


Fig. 87.—Showing relation between compaction and shear strength. (Slightly modified curves from Buchanan.)

The Effect of Soil Type.

There is a direct connection between soil type and degree of compaction, more especially as shown by the plastic limit of the soil, and maximum densities also vary with plastic limit. For standard compaction, it is suggested that the optimum moisture content is about 2 to 5 per cent. less than the plastic limit. Heavy clays have densities of from 90 to 105 lb. per cubic foot, while silty clays run up to 105 to 115 lb. per cubic foot. Higher densities are obtained only with sandy or gravelly soils in which there is little cohesion to resist compaction.

Table 16 gives an approximate classification of quality of soils for fills based on soil characteristics.

TABLE 16.—RELATION BETWEEN MAXIMUM DENSITY AND INDEX PROPERTIES OF COHESIVE SOILS.

Quality of soil for fills	Range of maximum dry densities, lb. per cu. ft.	Liquid limit	Plastic limit
Unsatisfactory or very poor	70—100	> 65	> 22
Poor	100—110	50—65	19—22
Fair	110—120	32—50	16—19
Good	120—130	24—32	14—16
Excellent	> 130	< 24	< 14

The effect of grading of soils on compaction. This problem has been investigated by J. Shaw[44] working with the author, as a result of which the best gradings for stabilised soils have been determined as shown in Table 17.

TABLE 17.—SHOWING GRADINGS FOR DENSEST AND MECHANICALLY STABLE SOILS.

B.S. Sieves	Suggested percentage passing	A.A.S.H.O. M56-42 percentage passing
1"	100	100
$\frac{3}{4}$"	87	70—100
$\frac{1}{2}$"	77	—
$\frac{3}{8}$"	72	50—80
$\frac{3}{16}$"	63	35—65
No. 7	55	25—50
36	36	15—30
200	18	5—15

Shaw's work showed that in the case of soils a minimum density exists at a moisture content intermediate between the loose, dry condition and the ordinary optimum moisture content (see Fig. 88) ; the effect is thought to be produced by swell clays or markedly adsorptive clays in the soil mortar.

The effect on compaction of the presence of stones in cohesive soils has been studied by L. Maddison[45], who found that the admixture of stone up to about 50 per cent. had very little effect on the moisture-density relationship of the soil mortar, the aggregate acting merely as a displacer. With stone contents exceeding 50 per cent., contact between the stones caused the material to be difficult to compact, and a higher optimum moisture content was then required to compact the soil mortar. With increasing amounts of stone, the soils become non-cohesive, and the soil grading is then the most important factor controlling soil density, the limits necessary to give a dense material after compaction being covered by A.A.S.H.O. Specification M56-42 (see Chapter 9). E. J. Yoder and K. B. Woods[46] have shown that maximum density does not necessarily mean maximum strength. Mixtures of crushed stone and soil gave the highest density and strength, with gravel-soil, sand-soil and dune sand-soil mixtures having lower strengths in the order given. Small quantities of soil mixed with granular materials were found to be desirable, but beyond a certain limit larger quantities of soil were detrimental.

Soil-gravel mixes had maximum density with 10 per cent. of soil and maximum strength with 7 per cent. of soil ; soil-stone mixes gave corresponding figures of 9 and 7 per cent. respectively, while for soil-sand and soil-fine sand mixes, the figures were 10 and 13 per cent., and 40 and 25 per cent. respectively. It was also found that optimum soil

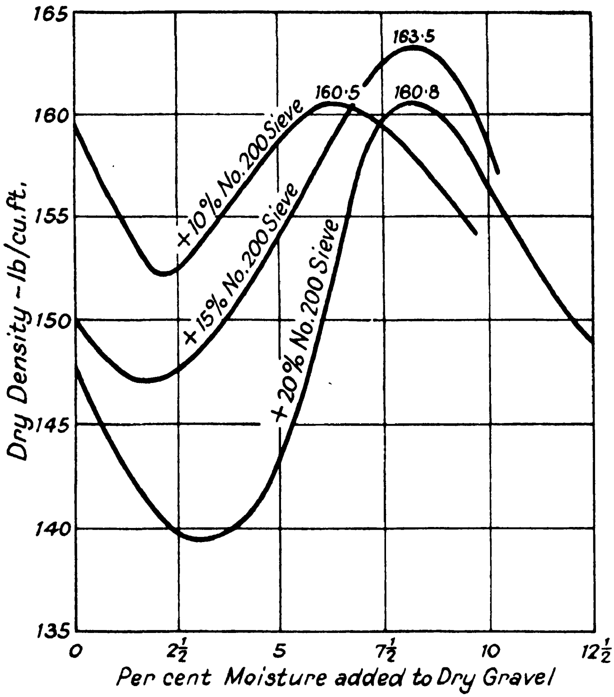


Fig. 88.—Typical moisture-density curves of mixtures of soil and 1-inch gravel. (Shaw.)

content decreases as the compactive effort increases, this being probably due to aggregate degradation caused by the compaction, and that for comparable densities, strengths are lower with over-optimum than with under-optimum amounts of soil, or in other words, it is preferable to have too little binder than too much.

The Effect of Admixtures on the Density of Soils.

This matter is important in stabilised base-course construction for roads and aerodrome runways. The use of calcium chloride has been found to increase the dry density of compacted material by as much as 11 per cent.[47], while the addition of other electrolytes increases the maximum density from 5 to 10 per cent. and decreases the optimum moisture content. Sodium chloride increases the density of soils, the greatest increase occurring with the first 2 per cent. of the salt, but the effect of adding portland cement is not very marked. The effect of adding chemical waterproofers such as " Vinsol " resin is peculiar; additions of up to about 0·5 per cent. had no effect on a silty soil, but slightly increased the density of a sand. In both cases a pronounced reduction (about 5 lb. per cubic foot) took place with 2 per cent. of this resin.

The Effect of Temperature on Soil Compaction.

C. A. Hogentogler[40] has shown that increases of temperature increase the maximum density and decreases the optimum moisture content of a soil, so that an increase in temperature corresponds to an increase in compaction, but that a softening of the soil also takes place. Table 18 gives particulars of this work; the red clay and Iredell soils referred to possessed high plasticity and volume change, while the Arlington soil was a typical silt-loam.

TABLE 18.—SHOWING THE EFFECT OF TEMPERATURE ON SOIL COMPACTION.

Name of Soil	Liquid limit	Plasticity index	Shrinkage limit	Max. dry density lb. per cubic foot			Optimum moisture content per cent.		
				35 °F.	75 °F.	112 °F.	35°	75°	112°
Arlington	27	7	19	109·2	110·7	112	16·2	15·2	14·4
Iredell	78	55	12	100·2	101·1	102	23·1	23·1	20·5
Red clay	65	47	10	109·2	111·1	113	16·9	16·9	15·2

Settlement of Fills as affected by Compaction.

Little work is available on this important problem, but H. Allen[48] has correlated settlement with degree of compaction by carrying out static compressibility tests on specimens 8 inches in diameter and 3 inches high under a load of 15,000 lb. per square inch, when it was found that by increasing the ramming from 15 to 25 blows, the maximum settlement was reduced from 2·4 to 1·2 per cent., while with 35 blows the settlement was further reduced to 0·9 per cent. The maximum settlement was found to occur at moisture contents somewhat below optimum.

No published data appears to be available on the subject of the effect of repeated stress on the settlement of cohesive soils, but L. Casagrande[49] has studied the effect of repeated loadings on compacted and uncompacted sand. The test specimens were 6 inches in diameter and 2 inches high, and compressive stresses of 14 lb. per square inch were applied several hundred times. The sand used was uniformly graded passing a No. 25 and retained on a No. 100 mesh sieve under conditions as under :—

Reference.	*Condition.*	*Percentage of voids.*	*Dry density (lb./cub. ft.)*
I	Loose damp	46·6	90
II	Loose dry	42·2	98
III	Compact damp	41·0	100
IV	Compact dry	35·2	109

The specific gravity of the sand grains was 2·70

It is evident from the above experiments that the more compact sands settle the least (see Fig. 89), but even in the case of sample IV,

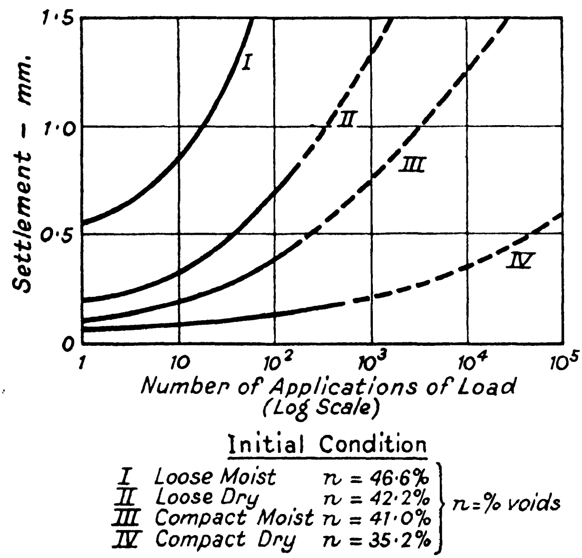


Fig. 89.—Showing settlement of sand under repeated applications of load. (L. Casagrande.)

the final amount of settlement was by no means negligible. For 100,000 load repetitions, easily obtainable in a few weeks on a densely trafficked road, the estimated settlement is about 1 per cent., and even if confined to the top 12 inches of soil, this would amount to $\frac{1}{8}$ inch, a serious matter in road and runway construction, especially if under rigid surfacings.

Other points in which compaction may affect our subject are those of elasticity, permeability, water absorption.

Elasticity. It is clear that the elastic properties of soils must depend largely upon their degree of compaction, and that the modulus of elasticity of soils is probably closely related to the modulus of subgrade reaction k (see page 143) on which the Westergaard theory of concrete slab design is based. It has been shown in Chapters 1 and 2 that a considerable use is made of elastic theories in structural foundation analysis, but much field work on soils *in situ* and further laboratory study of " undisturbed " samples of soils is still required before we can be said to have reached finality in this problem.

Permeability. Although the degree of compaction of a soil must affect its permeability, the effects are sometimes over-estimated. J. Kozeny[50] deduced a formula relating permeability and grain-size with voids for non-cohesive soils in which permeability is a function of $\frac{\epsilon}{1 - \epsilon^2}$, where ϵ is the ratio of voids to total volume.

Typical values for loose and compacted sand are :—

Condition	*Percentage of voids* ($\epsilon \times 100$)	*Dry density* (*lb. cub. ft.*)	*Relative permeability*
Loose	47	90	100
Dense	35	110	37

S. J. Buchanan[42] gives permeability figures for a silty clay as follows :

Number of blows	*Permeability* (*cm./sec.*)
6	13×10^{-9}
60	$2{\cdot}8 \times 10^{-9}$

Unfortunately comparisons of work of this kind are almost impossible until standard methods of measuring permeability both in the laboratory and in the field become available.

Weathering. The permanence of compaction of clays subjected to alternate wetting and drying has been studied by H. C. Porter[39], who found that if the wetted material is dried slowly and wetted slowly even to saturation, the structures of the specimens did not disintegrate or rupture, although, as might be expected, the clays expanded on wetting. After several cycles of slow drying and wetting had been applied, it was found that the specimens returned almost to their original density, so that the compaction could be regarded as permanent. When, however, a compacted soil sample is dried quickly (as by the hot sun and dry winds prevalent on the high veld of South Africa) and then suddenly wetted as by a storm, the soil cracks and disintegrates. For this reason it is advisable to protect soils containing clay material from sudden changes of moisture.

Compaction under Field Conditions.

The three chief methods used to compact soil in the field are (*i*) pressure, (*ii*) impact, (*iii*) vibration, the object in all cases being to induce closer packing of the soil particles and hence higher density. The stresses applied are both shear and compression, though it does not seem to be known which of these types of stress is the more effective in the compaction of soil.

The stresses under a concentrated load applied to a soil mass are obviously proportional to the applied load, while the soil density produced decreases with increasing depth. It should be noted that if too high a stress is applied, proper compaction does not occur, but plastic flow of the soil is produced instead. The decrease of stress with depth is one of the reasons why it is necessary to compact soil in thin layers.

Different types of soil give rise to different effects under pressure or vibration. Thus, Fu-Shen Fang[51] found that ramming was more effective on sandy clays than on sands, and that its effects extended to a greater depth, while vibration was more effective on the non-cohesive than on cohesive soils, this result agreeing with both laboratory and field investigation carried out by T. A. Wade and the author.

Effectiveness of Compaction Equipment.

The usual types of field compaction equipment include :—

1.	*Rollers.*	Smooth, steel-wheeled rollers.
		Pneumatic tyred rollers.
		Lorries and pneumatic tyred equipment.
		Tractors.
		Sheepsfoot rollers.
2.	*Rammers.*	Dropping weight type.
		Internal combustion type.
		Pneumatic type.
3.	*Vibrators.*	

Scientific study of this subject is as yet in its infancy, although W. Loos[52] has investigated the effectiveness of various methods of field compaction for both cohesive and non-cohesive soils. Unfortunately the paramount importance of moisture content was apparently not realised at that time, so that the work is merely qualitative. On loose sandy soil, none of the rollers succeeded in compacting the material, although with sands containing sufficient cohesive material, satisfactory results were obtained. The " dropping weight " method of compaction in which a 2 ton iron plate was dropped repeatedly from a height of about 7 feet by a self-propelling crane gave good compaction down to depths of 3 feet. Heavy " frog-rammers " weighing 500 and 1,000 kg. were also used, while a 1 ton rammer compacted a sandy soil to a depth of 16 inches, a ½ ton rammer giving a corresponding depth of 12 inches. It was found that compressed air rammers of smaller dimensions were less effective and more expensive. The " Losenhausen " 24 ton track-mounted machine employs unbalanced rotating weights which produce powerful vibrating forces, the machine being operated at the resonant frequency vibrating period for these weights, which is 13 to 15 cycles per second for sand. This machine was found to be most effective on sands and gravels, very high densities being recorded.

Vibration methods of soil compaction have been developed greatly in areas such as Northern Germany, where coarse aggregates are very scarce, and large areas of unstable, badly graded wind-blown sands are found. One type of compaction of this kind is that known as the Franki method, in which a steel tube with a concrete base is driven by a tamper dropped vertically inside the tube and falling on to the concrete base. When the required depth has been reached, the tube is gradually withdrawn and the space occupied by it replaced by tamped broken stone. A test performed on one soil of the above type showed that the permissible load could be increased from 2 kg. per square cm. to 4½ kg. per square cm. as a result of the compaction produced by this method.

A second method is that known as the Keller process, which utilises a vibrator consisting of a vertical spindle carrying eccentric weights and driven by an electric motor of from 40 to 50 h.p. The amplitude of the vibrations of the machine is 10 mm. and the speed 3,000 cycles per minute. The vibrator is jetted into the soil by means of a water jet, the quantity of water required being about 10 litres per second for fine sand and 30 to 50 litres per second for coarse gravel. After having been jetted to the required depth, the water supply is diverted and vibration begun, the vibrator being gradually drawn upwards. The hole left by the vibrator is filled with sand. The time taken for compaction to a depth of 20 metres is 10 minutes, the effective diameter of the compacted area being about 2 metres. Piling is unnecessary with this method unless the sand is overlain by a bed of clay, in which case the vibrator is jetted through the clay and the hole thus made is lined ; after withdrawal of the vibrator, the hole in the clay is filled with concrete.

All the above methods affect only the upper surface of the soil, but if hollow steel piles are used, compaction can be obtained at considerable depths, gravel being used to fill the pile holes as the piles are withdrawn. By this method, the voids in sand have been reduced from 42 to 35 per cent., and the dry soil density increased from 98 to 110 lb. per cubic foot. W. Bernatik[53] describes special methods of sand compaction in which water is injected into the soils and vibration methods applied simultaneously, high densities being recorded.

G. Lange[54] describes tests on the compaction of cohesive soils in which it was found that vibration methods were not effective, although a 2½ ton dropping weight falling through 5½ feet and a 1,000 kg. frog-rammer were found to be equally effective for layers up to 2½ feet thick ; the frog-rammer was found to be preferable to the dropping weight machine in that it gave a smooth surface finish, whereas the dropping weight machine did not. Scrapers of a capacity of 5·9 cubic yards were found to compact soil effectively up to 2 feet in thickness ; other workers found that a 10 cubic yard scraper gave good compaction up to a depth of about 18 inches.

It has been found from experience that heavy rubber-tyred traffic is very effective in soil compaction, probably on account of the kneading action produced by the tyres. Measurements made by the Road Research Laboratory at Harmondsworth show that scrapers compacting heavy clays at moisture contents above the optimum give densities approaching that obtained in the standard compaction test at the same moisture content. Thus with a clay of liquid limit 53, the relative compaction on layers 1 foot thick produced by scrapers was 90 per cent., dumpers giving a compaction of only 70 per cent. under the same conditions. In the case of a silty soil with a liquid limit of 25, a relative compaction of 104 per cent. was obtained from lorry traffic as against 95 to 100 per cent. with sheepsfoot rollers. A relative compaction exceeding 100 per cent. can easily occur if heavy compaction is applied to a soil at somewhat less moisture content than the optimum, and this confirms the view of the effectiveness of heavy pneumatic-tyred traffic for this purpose. It has been found by H. F. Peckworth[55] that the effect of compaction is progressive ; thus the dry density increased linearly from 100 to 109 lb. per cubic foot as the number of passes increased from 16 to 30. In the past it has been a common practice to use traffic for compaction purposes, but this is not recommended as a substitute for systematic rolling in view of the uneven distribution of such traffic.

Application of Soil Compaction Methods to Construction Works.

As a result of present-day knowledge of soil compaction methods, it is possible to draw up performance specifications for fill construction which lay down a required dry soil density. Such specifications usually leave the method of compaction to the contractor, but it is probable that as knowledge of the relative effectiveness of various types of compacting machinery develops, this will no longer be the case.

A typical performance specification for this purpose is A.A.S.H.O. M57-42 (see Table 19).

TABLE 19.—SHOWING REQUIREMENTS OF A.A.S.H.O. M57-42 FOR COMPACTION OF FILLS AND SUB-GRADES.

Type of Construction	Material specified	Remarks
Fills more than 50 feet high	Dry density to be at least 120 lb. per cub. ft. Soils of the A1, A2 or A3 groups to be used when available. If A4, A5, A6, A7 soils have to be used, special attention to design and construction is necessary. A8 soils may not be used.	Predominantly sandy and gravelly soils best. Clay soils undesirable and excluded unless very heavily compacted. Peaty soils excluded.
Fills less than 50 feet in height	The dry density must be at least 90 lb. per cubic foot; otherwise specification as above.	Lower density requirements greatly extends range of soils which can be used.
Sub-grades	Materials of the A1, A2 or A3 groups when available. A4 to A7 soils may be used if compacted to 95 per cent. of the maximum density, with a moisture content equal to or greater than optimum moisture content.	

The relative compaction specified by A.A.S.H.O. M57-42 is as given in Table 20.

TABLE 20.—SHOWING RELATIVE COMPACTION REQUIRED IN A.A.S.H.O. M57-42.

Maximum density obtained in standard compaction test	Minimum compaction required in per cent. of maximum density
Lbs. per cubic foot	
90— 99·9	100
100—109·9	95
110—119·9	95
120—129·9	90
120 and over	90

Field Control of Soil Compaction.

The apparatus required for the field control of soil compaction is as follows:—Compaction cylinder and rammer (see Chapter 5), sand-bottle apparatus for field density measurements (see Chapter 5), simple weighing equipment and drying oven.

The U.S. Public Roads Administration recommends that at least four density and moisture tests should be made in each 8 hour day, and not less than one test per 500 cubic yards of earthwork, but evidently the exact number of tests decided on in any particular case must depend very largely on experience. Travelling van laboratories are being used increasingly for this and for other classes of field control of soil operations.

The American Portland Cement Association suggests the following types of rollers for different soils under American conditions :—

TABLE 21.—TYPES AND PRESSURES OF ROLLERS FOR SOIL COMPACTION.

Nature of soil	Sheepsfoot rollers		Smooth rollers
	Area of feet (sq. in.)	Pressure on feet (lb./sq. in.)	Tons
Sandy	9—12	50—100	3—5
Sandy and light clay loams	7	100—200	5—8
Heavy clay ; stony soil	5—6	200—400	8—12

The three main variables in fill construction are the type of soil, its moisture content, and the methods used for compaction. The first of these variables may not offer much choice, but the two last are well within engineering control. R. R. Proctor[56] was the first to show that the dry density of soil compacted under standard conditions was a function of its moisture content. It is also known that in the case of cohesive soils air only is expelled by compaction, and that the difficulty of expulsion of this air increases as the air voids become smaller and also that such soils become stiffer as their moisture content decreases.

K. B. Woods[57] gives requirements for the degree of compaction required by the State of Ohio for fills above and below 10 feet in height, while the basic principles of soil compaction have been summarised by A. H. D. Markwick[58] as follows :—

1. The dry soil density is a measure of the closeness of the packing of the soil particles and hence of the state of compaction of the soil.

2. Compaction increases the closeness of packing and hence the dry density of the soil, but in general only air is expelled from the voids to make up for the increased volume of solids.

3. Soil density increases with the amount of compaction at a decreasing rate until a limit is reached at which only 3 per cent. or less of air is left in the soil.

4. With varying moisture content and equal amounts of compaction, a moisture content is reached at which maximum dry density occurs.

5. Both maximum density and optimum moisture content vary with the type of soil and the amount of compaction applied.

6. Extra rolling benefits only soils of relatively dry consistence rolled at moisture content below the optimum.

7. The rolling of clay soils which are too soft puddles the clay and is very harmful to the work.

8. Increase in compacted density increases strength and stability and reduces liability to settlement. The capacity to absorb water decreases with increase in dry density.

9. In general, the effectiveness of all compaction equipment decreases with increase in depth of soil; this is why it is preferable always to consolidate in thin layers only.

10. The heavier the compaction equipment, the more effective the compaction and the greater the depth to which the soil is compacted, provided always that the soil is not too soft to support the compacting equipment.

11. Ramming is effective on both non-cohesive and cohesive soils. Rollers are effective on soils with some cohesion, but not on sandy soils. Vibration is very effective on non-cohesive but not on cohesive soils. In tests on a silty clay, sheepsfoot rollers gave the best results.

The chief problems which have to be decided by the engineer who has to construct sub-grades or base-courses or fills, with any given soil, are the necessary thickness of layer, the best method of compaction, and the most economic number of passes required. The Road Research Laboratory at Harmondsworth has developed a method of test for this problem as follows :—

An area of soil about twenty feet long by about ten feet wide, of a thickness assumed for preliminary purposes, is thoroughly broken up and brought to a known moisture content with a rotary hoe. The soil is compacted until it seems from visual inspection that no further

TABLE 22.

TENTATIVE SUGGESTIONS BY THE ROAD RESEARCH LABORATORY ON EQUIPMENT FOR THE COMPACTION OF EARTHWORKS.

Type of filling		Suggested thickness (inches)	Suggested Compaction Equipment		Remarks
Class	Type		At or below optimum moisture content	Above optimum moisture content	
Hard rock	(a) boulders and large fragmental rocks	12 to 24	12 ton smooth roller; 2 ton dropping weight	—	—
	(b) fragments 3″ downwards	12 to 18	8 ton smooth roller; 2 ton dropping weight	—	—
Friable rock	(a) large fragmental rocks	—	—	—	—
	(b) fragments 4″ downwards	12	Sheepsfoot roller followed by 8 ton smooth roller	—	Undesirable type; should be blasted, crushed or disintegrated by weathering.
Loose materials	(a) shingle and loose gravel	8 to 12	Track-laying tractors and other track-laying equipment; 2 ton dropping weight	—	—
	(b) loose sand	8 to 12	Track-laying tractors (preferably with smooth cleats); vibrators; 2 ton dropping weight	—	Best compaction obtained if sand is damp.
Cohesive soils	(a) cohesive sand	8 to 12	Sheepsfoot roller; pneumatic-tyred roller and 5 ton smooth roller	Construction traffic plus light rolling with smooth roller	Material that is too soft and wet should be allowed to dry out before being compacted.
	(b) silty clay	6 to 8	Sheepsfoot roller; 8 ton smooth roller	Ditto.	—
	(c) heavy clay	6 to 8	Sheepsfoot roller; 8 to 12 ton smooth roller	Ditto	—
	(d) soil-aggregate mixtures	6 to 8	Pneumatic-tyred roller and 8 ton smooth roller	—	—

increase in compaction can be obtained ; the dry soil density is found by either the core-cutter or sand replacement method. This procedure is repeated at several different moisture contents, whence a curve relating soil density and moisture content is obtained just as in the standard Proctor test. The test is then repeated with the various types of plant or combinations of plant which may be available for compaction purposes. Similar tests are undertaken to determine the relation between soil density and the number of passes for each type of compacting equipment at the appropriate optimum moisture content. It should be possible to decide from the results of these two sets of tests the best type of equipment and the number of passes to use.

The foregoing procedure may appear somewhat lengthy, especially in cases where many different soil types are present, but should afford valuable information on a problem concerning which a great deal of research work is needed.

CHAPTER EIGHT.

LOADS ON SOILS AND THE DESIGN OF SUB-GRADES AND BASE-COURSES.

Loads on Soils.

Apart from the common cases of loads on soils caused by engineering structures, the more special case of loads caused by vehicles on highways and aerodrome runways and the stresses induced by such loads in the underlying sub-grade and base-course has received considerable attention in recent years, more especially during World War II, with the corresponding development of aerodromes.

A. H. D. Markwick and H. J. H. Starks[59] have pointed out that the tyres of vehicles exert the following types of stress on the road :—

(*a*) *Normal Stresses.*

1. Pneumatic tyres exert maximum normal stresses proportional to and approximately $1\frac{1}{2}$ times the inflation pressure. The maximum pressure is of the order of 150 lb. per square inch, and occurs only under the tyres of heavy lorries.

2. Solid tyres exert normal pressures higher than those found under pneumatic tyres.

3. The normal stresses appear to be independent of the speed of the vehicle, and are the same for moving and for stationary tyres.

4. Very high local intensities of normal pressure occur on sharp projections in the road surface. These pressures depend upon the hardness of the tread rubber of the tyre and upon the shape of the projection, and are almost independent of tyre size and inflation pressure.

(*b*) *Shear Stresses.*

1. In the case of pneumatic tyres, the shear stresses are directed inwards and give rise to horizontal compressive stresses in the road surface beneath the tyre ; they may reach 50 lb. per square inch in intensity.

2. In the case of solid tyres, the shear stresses are directed outwards, and give rise to horizontal tensile stresses in the road surface beneath the tyre.

3. The shear stresses caused by a pneumatic tyre running on a circular track depend on the wetness or dryness of the road. Under dry road conditions, rapid alternations in stress occur at the instant that the portion of the tyre in contact with the road leaves the surface ; on a wet road, these alternations do not take place.

4. The distribution of shear stresses under tyres is influenced considerably by the magnitude and direction of the resultant thrust of the wheel on the road.

Markwick[60] has also dealt with **loading conditions under aircraft tyres,** in which the pressures are considerably higher than is the case with the tyres of private cars, and are in fact comparable with those in lorry tyres. Thus in 1942, aircraft tyre pressures were of the order of 50 lb. per square inch, but it was expected that such pressures might increase to 85 lb. per square inch in a relatively short space of time. Tyre contact areas could be expected to be from 600 to 1,000 square inches for aircraft of 100,000 lb. all up weight, so that the major axis of the ellipse of contact, assuming a ratio of 1·7 to 1 for major to minor axis, would be from 3 to 4 feet. It has been shown by S. Timoshenko[61] that if the contact area be assumed circular, the maximum shear stress occurs at a depth of about two-thirds the radius of the contact area. An approximate analysis of the bearing capacity of a loaded circular area resting on a cohesive soil is given by

$$\text{Bearing capacity of soil } q = 5c + \frac{aw}{6}$$

where c = cohesion of soil,

w = unit weight of soil,

a = radius of contact area.

The use of numerical values shows that the term $\frac{aw}{6}$ is negligible, so that the bearing capacity of a clay soil is governed almost entirely by the cohesive strength of the soil and is independent of the wheel load. In a sandy soil, however, the unit bearing capacity increases with increase in wheel load. Markwick puts forward the following conclusions :—

1. The intensity of pressure on the soil is equal to the inflation pressure of the tyre.

2. In the case of purely cohesive soils, the ground will be stable provided that the cohesive strength (= in this case the shear strength) of the soil exceeds about one-fifth the tyre pressure.

3. On a purely granular soil such as sand, the allowable bearing pressure increases with tyre size in proportion to $\frac{\text{wheel load}}{3}$, and hence in the ratio of linear dimensions of the area of contact.

4. On a purely granular soil the bearing capacity increases rapidly with increasing angles of friction ; hence, densely compacted sand has a higher bearing capacity than loosely compacted sand.

5. On a purely granular soil, the surcharge caused by the dead weight of the surfacing permits of a considerable increase in the bearing pressure.

6. The allowable bearing pressure on a purely granular soil is considerably reduced when the water-table approaches the surface.

7. When a soil possesses both cohesion and internal friction, the bearing capacity increases considerably with increase in the angle of internal friction.

Additional factors arise under a moving tyre, such as impact and the effect of short-duration loading on the shear strength of the soil, but static loads are in fact considered to be more dangerous than moving loads.

M. G. Spangler and H. O. Ustrud[62] have studied **wheel load distribution through flexible type pavements** and conclude that :—

1. The pressure on a sub-grade beneath a flexible type pavement is distributed in accordance with the typical helmet-shaped surface indicated by the Boussinesq solution (see Chapter 1) and J. H. Griffith's concentration factor formula

$$\sigma_z = \frac{CP}{tn}$$

where C and n are disposable parameters (for typical values see below),

P = total tyre pressure in pounds,

t = pavement thickness in inches,

σ_z = average maximum pressure on sub-grade in lb. per sq. inch.

The average values of C and n may be taken as 0·075 and 1 respectively, with C varying between about 0·06 for a load of 1,000 lbs. and 0·09 for a load of 5,000 lbs.

2. The maximum pressure occurs on a relatively small area directly beneath the tyre contact area, and the value of this maximum pressure is independent of the inflation pressure of the tyre.

3. The maximum sub-grade pressure is nearly directly dependent upon the magnitude of the applied load, though there is some evidence of a non-linear relationship.

4. The maximum sub-grade pressure varies in inverse ratio with the thickness of the pavement and may be expressed by the empirical formula given above.

5. The above conclusions were arrived at as a result of laboratory experiments only and have not yet been checked by experiments in the field.

Spangler[63] also suggests that a flexible pavement has little or no resistance to deformation under applied load, and that its failure can only be the result of the failure of the sub-grade due to excessive deflection, causing the pavement to develop the familiar " alligator cracks " characteristic of such failures, but these assumptions have been disputed by P. Hubbard and H. G. Nevitt. Measurements show that the pattern of sub-grade pressure distribution is bell-shaped and that the maximum pressure is developed directly below the wheel load, and also that the magnitude of this pressure depends upon the thickness of the pavement, the amount of the load and the stiffness of the sub-grade soil. An empirical expression for the pressure on the sub-grade which takes these factors into account, and which meets the requirements of equilibrium, is

$$\sigma_r = \frac{CP}{t} e^{\frac{-\pi C^2}{t} r^2}$$

where σ_r = pressure on sub-grade in lb. per square inch,

P = wheel load in pounds,

t = thickness of pavement in inches,

r = radial distance from centre of load in inches,

C = sub-grade stress factor = $\dfrac{\cdot 0070 + \cdot 0000068 E_c}{\sqrt[3]{\dfrac{P}{1{,}000}}}$

E_c = modulus of compression of sub-grade in lb. per square inch.

e = base of Napierian logarithms

If the sub-grade soil is assumed to be quasi-elastic in character, and proper values of the modulus of compression can be determined, the maximum central deformation of the pavement and sub-grade can be determined by the theory of elasticity. The value of this maximum central deformation ΔZ_0 can be obtained from the formula :—

$$\Delta Z_0 = \frac{0{\cdot}9P\sqrt{\frac{C}{t}}}{E_c}$$

Each type of pavement has a safe allowable deflection which is a function of the thickness, and if this deflection is known, it can be found for any particular case ; but unfortunately little is known at present as to safe deflection values for the various types of flexible pavements, although a limit of 0·1 inch has been suggested.

The nature of the stresses under circular loaded areas have also been studied by L. A. Palmer[64], who concluded that on the basis of A. E. H. Love's complete solution of such stresses[65], it is indicated that the greatest shearing stresses may be confined within the flexible pavement and may not reach the sub-grade if the thickness of the flexible pavement is not less than the radius of a circle having an area equal to the plane of contact between the tyre and the flexible pavement. This of course means that the flexible pavement has to be of a relatively heavy type of construction. In the case of a uniform load on a circular area, the variation in shearing stress is relatively small if Poisson's ratio for the material under stress lies between the values 0·25 and 0·5, which is thought to be usually the case.

The real difficulty in all studies relating to the transmission of loads on highway and aerodrome surfacings is that usually one has to deal with three types of material, all differing in their physical and mechanical properties, i.e., the sub-grade, the base-course and the surfacing, and at the time of writing, no complete analytical solution of this problem appears to be available.

The construction of pavements on clay foundation soils has been dealt with in some detail by R. Glossop and H. Q. Golder[66], as follows :—

Determination of Thickness Required. Firstly the loads on the surfacing must be known, whence the stresses in the sub-grades can be found. The area of contact of the tyre of the vehicle can be found by measuring the major and minor axes of the ellipse of contact, whence the diameter of the equivalent circle is calculated. The load supported by the tyre can be divided by this area to give an assumed uniform pressure acting over the equivalent circle.

The elastic properties of the base-course and sub-grade are dissimilar, but it is assumed that together they form a semi-infinite homogeneous isotropic elastic medium and that the " bulb of pressure " distribution of stress shown in Fig. 90 applies. The vertical pressures on a horizontal plane can then be calculated from Jurgensen's tabulated solution of Boussinesq's equation integrated for a circular loaded area. The pressure is a maximum below the centre and diminishes radially as plotted in Fig. 91 for unit pressure at the surface, depths being expressed in terms of the diameter " d " of the loaded area.

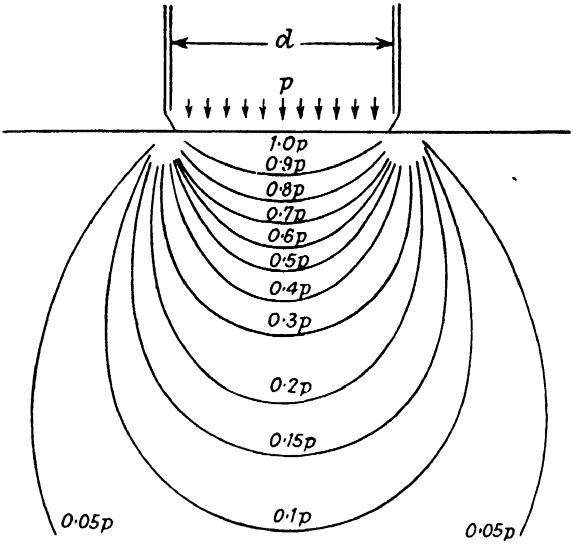
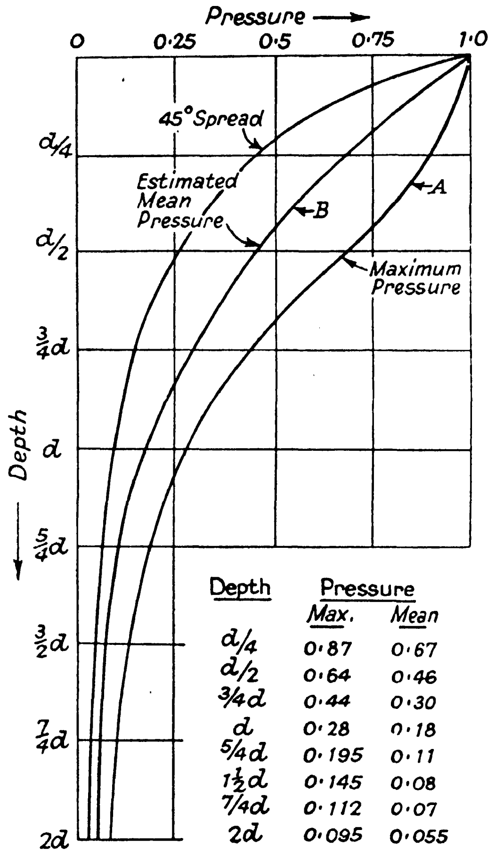


Fig. 90.—Showing stress-distribution below a circular loaded area.

The vertical pressure on the sub-grade obviously depends on the thickness of the pavement, and the required thickness of the pavement is that which keeps the maximum pressure on the sub-grade below a certain value which is a function of the shear strength of the sub-grade. Three possible limiting values to the pressure on the sub-grade have been considered, and observations made on actual failures in order to determine which hypothesis agreed most nearly with the facts :

(*i*) The first possibility is to keep the maximum pressure below π times the shear strength of the soil, since at this value the latter just reaches its plastic state.

(*ii*) An alternative method of design is to calculate the pressure which will cause failure of the sub-grade and then to multiply this value by an adequate factor of safety. Hencky gives the ultimate bearing capacity of a circular loaded area as 5·64s, where s is the shear strength of the soil, and although tests have indicated a possible higher value of 6·7s, it is wise to take the lower value for design purposes. Terzaghi



Depth	Pressure Max.	Pressure Mean
d/4	0·87	0·67
d/2	0·64	0·46
3/4d	0·44	0·30
d	0·28	0·18
5/4d	0·195	0·11
1½d	0·145	0·08
7/4d	0·112	0·07
2d	0·095	0·055

Fig. 91.—Showing diminution of pressure with increase in depth.

considers that a factor of safety of 2½ to 3 should be applied for structures liable to settlement, and since the problem is one of deformation rather than of shear failure, it is safer to use the higher factor of safety 3. This factor of safety must be applied to the mean pressure over the surface of the sub-grade.

The curve B in Fig. 91 was obtained by plotting mean pressure against depth, and $\dfrac{3 \times \text{mean pressure}}{5{\cdot}64}$ can be similarly plotted.

(*iii*) Finally a less conservative estimate, and one which is often used in the design of footings, is to keep the mean pressure below πs.

In the curve shown in Fig. 91, the tensile strength of the pavement is ignored, the load is assumed to be applied as a uniform pressure over a circular area, and a Boussinesq distribution of stress is assumed both in the pavement and in the sub-grade. It follows from Fig. 90 that the maximum stress $\frac{p}{\pi}$ occurs along a curve starting at the periphery and reaching a depth of nearly 0·33*d*, where *d* is the diameter of the loaded area. The stresses decrease on both sides of this curve, and along the surface of contact the shear stress is zero. At any depth greater than 0·33*d*, the maximum shear stress occurs along the vertical axis.

Golder[67] compares various methods of **determination of runway thickness,** and gives a new method of design for sandy soils. In this comparison, it is pointed out that Westergaard's analysis is probably sound mathematically, but suffers from the disadvantage that it is difficult to find *k*, the modulus of sub-grade reaction, easily and accurately (see Chapter 5). It is thought that results obtained by this method give thicknesses which are too small in the case of weak clays, and too large in the case of dense sands. The California Bearing Ratio Test is probably adequate for soft clays, but too conservative for stiff clays since the tests are carried out on soft material. The test is in any case entirely empirical, and if a new range of loads is introduced, a number of large-scale tests would be required to bring the design charts up-to-date. It is also pointed out that the methods given by Glossop and Golder are applicable only to clays and are probably conservative. Golder's new method for sands is as follows :—

Bearing tests to failure are carried out on a loaded area 4 feet square. Shear tests in a box shear apparatus are made on samples of sand taken from below the loaded area, and from these the value of the angle of internal friction ϕ is deduced, check tests being made in the tri-axial shear apparatus. Terzaghi's formula is used for the determination of the ultimate bearing pressure of the soil :—

$$q_u = cN_c + wD_f.N_q + wBN_w$$

where q_u = ultimate bearing pressure,

c = cohesion of soil,

w = unit weight of soil,

D_f = depth of footing,

$2B$ = breadth of footing.

N_c, N_q and N_w are coefficients depending on the value of ϕ ; the values of these coefficients can be obtained from Fig. 92.

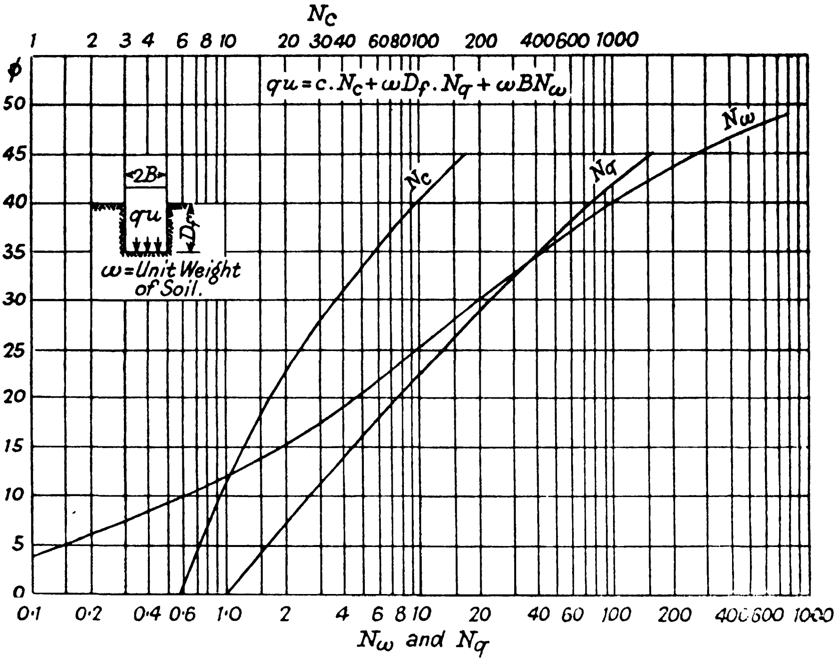


Fig. 92.—Bearing capacity constants for shallow footings. $Df > 2B$. (Terzaghi.)

The pressure q_u is the pressure applied to the sub-grade, and not the pressure on the pavement ; this latter pressure is denoted by p, and is assumed to be equal to the tyre pressure. A loaded rectangle b wide and $2b$ long is assumed, and the total wheel load $P = 2b^2p$. A spread of $26\frac{1}{2}°$ of the load through the pavement is assumed. Then loaded area on sub-grade $A_t = (2b + 2t.tan\ 26\frac{1}{2}°)(b + 2t.tan\ 26\frac{1}{2}°)$, where t is the thickness of the pavement, and the pressure q at depth $t = p.\frac{A}{A_t}$.

If $F = \dfrac{\text{ultimate bearing pressure } q}{\text{mean pressure applied to sub-grade}}$, then

$$F = \frac{cN_c + wtN_q + wBN_w}{p\dfrac{A}{A_t}}$$

$$= \frac{\frac{1}{2}w\ N_w\ (b + 2t.tan\ 26\frac{1}{2}°)\ + wtN_q + cN_c}{p.a.b}(a + 2t.tan\ 26\frac{1}{2}°)(b + 2t.tan\ 26\frac{1}{2}°)$$

Tan $26\frac{1}{2}° = 0{\cdot}4986$, and w = maximum density of a sandy soil, say 140 lb. per cubic foot, so that by substitution and reduction

$$F = wN_w(b + 2t) + 140N_qt + cN_c)\ \frac{(b^2 + 3bt + 2t^2)}{2pb^2}.$$ This is a new and general formula for the design of runway foundations.

A suitable loaded rectangle could be obtained from elliptical tyre prints, and different thicknesses of pavement used to give a suitable value for F, using Terzaghi's formula as above, together with Fig. 92 for the values of the coefficients N_w, N_q and N_c.

It is pointed out that inundation will halve the unit weight of the soil by buoyancy, and consequently that the bearing capacity would be halved if the ground becomes flooded. It is also concluded that in the case of a runway built on compact sand and gravel, the thickness may be independent of the total wheel load as long as tyre pressures are not increased.

Pressures in and beneath runways are also considered by C. A. Hogentogler and K. Terzaghi[68], who show that the mean intensity of pressure beneath a flexible surfacing may reach eight times the mean pressure under a concrete runway, although flexible surfacings are less affected by settlements than rigid slabs. The problem of concrete runway design is in fact the same as that of a concrete road except that wheel loads and contact areas are greater in the former case. H. M. Westergaard[69] has shown that concrete road slab analysis applies to concrete runways with only slight modifications. The theory used assumes an elastic sub-grade and the usual modulus of sub-grade reaction. W. H. Glanville[70] has shown that stresses induced by traffic depend mainly on the applied load and the thickness of the surfacing, and that the effect of changes in the value of this modulus is comparatively small, so that an exact knowledge of the value of k is unnecessary, and concrete thickness can be varied to suit good or bad soil, or made or unmade ground.

The theory of the transmission of stresses through flexible surfacings is by no means complete, but it is commonly assumed that such a surface spreads the vertical load through the frustum of a 45° cone, and that the pressure is distributed uniformly over the base of such a cone. If the load W acts over a circular area diameter d, and the permissible bearing pressure is denoted by q, the required thickness of surfacing t is given by

$$t = 0{\cdot}564\sqrt{\frac{W}{q}} - \frac{d}{2}$$

but this formula ignores both the shear strength of the surfacing and the stability of the subsoil. W. S. Housel[71] has attempted to allow for these factors by assuming that the load transmitted to the subsoil is reduced by an amount equal to that of a cylinder of surfacing beneath the load. The bearing capacity of the soil, which is taken to be purely cohesive, is $4c$, where c is the cohesion, and the safe bearing pressure which can be placed on the surfacing is

$$p = 4c + \frac{4st}{d}, \text{ where } s = \text{shear strength of surfacing.}$$

If, however, the foundation fails, it tends to heave up against the underside of the surfacing over an area greater than that of the load, giving an additional bearing capacity which is taken as $\frac{2st}{d}$. The total safe bearing pressure is thus suggested as

$$p = 4c + \frac{6st}{d}.$$

The ultimate shear strength of bituminous surfacings varies from 5 lb. per square inch upwards, while that of cement-stabilised soil may range from about 25 to 100 lb. per square inch. Housel's theory is based on an assumption of failure due to punching shear, but such failures are thought to be rare. According to P. Hubbard and F. C. Field[72], the failure of bituminous surfacings, as such, depends more upon their deflection properties than upon lack of stability in the soil; the experience of the author, more particularly in South Africa, does not support this latter view.

Empirical thicknesses of base-courses for various types of soil are available as follows[73]:—

Base-course (*Granular stabilised type*). For assumed maximum loads of 10,000 lb., the thicknesses in Table 23 are recommended.

TABLE 23.—SHOWING SUGGESTED THICKNESSES OF BASE-COURSE FOR VARIOUS TYPES OF SOIL.

Sub-grade Soil Group Classification	A1(b) Non-plastic	A1(a) Plastic	A2(a) Non-plastic	A2(b) Plastic	A3	A4 A4(7)	A5 A5(7)	A6	A7
Base-course thickness (inches)	0	5	5	6	5	8	8	8	8

In the opinion of the author the thicknesses given in the above table for A6 and A7 soils may prove insufficient in some cases.

Soil-Cement. (See Chapter 9). The thicknesses in Table 24 are recommended for highway pavements with average traffic not exceeding 50 trucks, with wheel loads between 4,000 and 10,000 lb. per day, or a total of 1,000 vehicles per day including the above-mentioned truck traffic.

TABLE 24.—SUGGESTED THICKNESSES OF BASE-COURSE FOR SOIL-CEMENT PAVINGS.

Sub-grade Soil Group Classification	A1(b) Non-plastic	A1(a) Plastic	A2(a) Non-plastic	A2(b) Plastic	A3	A4 A4(7)	A5 A5(7)	A6	A7
Base-course thickness (inches)	0	5	5	5	5	6	6	6	6

Sub-Bases. Assuming a maximum wheel load of 10,000 lb., the compacted thicknesses given in Table 25 are recommended for this purpose.

TABLE 25.—SUGGESTED THICKNESSES OF SUB-BASE.

Sub-grade Soil Group Classification	A1(b) Non-plastic	A1(a) Plastic	A2(a) Non-plastic	A2(b) Plastic	A3	A4 A4(7)	A5 A5(7)	A6	A7
Sub-base thickness (inches)	0	0–12[1]	0[2]	0–12[1]	0[2]	2–14[3-5]	4–14[3-5]	0–14[4-5]	0–14[4-5]

1. No sub-base is required over A1(a) and A2(b) soils where frost action is absent and the water table is low. In areas subject to frost action or where the water table is near the surface, the maximum thickness should be used.
2. With fine-grained A2(a) and A3 soils it is often necessary to modify the upper layer by admixture to a depth of several inches with binder soil, stone screenings or bituminous materials to produce stability.
3. On A4, A4(7) and A5(7) soils a sub-base course of the minimum thickness specified, consisting of stone screenings or similar material, is desirable to produce a firm support on which to place the overlying granular course. The maximum thickness is only required in areas subject to severe frost action or where the water table is close to the surface.
4. On A6 and A7 soils no sub-base is required where the water table is low enough to preclude access of capillary water to the sub-base. The maximum thickness is required where the water table is high, and for A6 soils, where frost conditions are severe.
5. A4, A4(7), A5(7), A6 and A7 soils are highly capillary and rapidly lose supporting power when saturated. The maximum thickness of sub-base is therefore required with such soils.

A summary of Tables 23, 24 and 25, giving total pavement thickness, is shown in Table 26.

TABLE 26.—SUGGESTED TOTAL PAVEMENT THICKNESSES.

Soil group Classification	A1(b) Non-plastic	A1(a) Plastic	A2(a) Non-plastic	A2(b) Plastic	A3	A4(7) A4	A5 A5(7)	A6	A7
Wearing course, inches	2	2	2	2	2	2	2	2	2
Base-course, inches	0	5	5	6	5	8	8	8	8
Sub-base, inches	0	0–12	0	0–12	0	2–14	4–14	0–14	0–14

The nature and quantity of clay content in base-courses has been dealt with by L. A. Deklotz[74], who has found that the combination of the plasticity index and liquid limit in the form $\frac{PI}{LL} \times 100 = D$ was of value as an indication of the quality of soil for engineering uses. Three soils of different qualities were added to each of three similarly graded samples of coarse material to produce a grading of maximum density. 13 per cent. passing the No. 40 mesh sieve, and two other gradings which differed only in the percentage passing the No. 40 sieve (23 and 33 per cent. respectively), were used, and the stability of the nine combinations was determined by means of the California Bearing Ratio Test.

These tests indicated that there is a definite relationship between the quantity $D \times$ percentage of soil and the stability, which should be of value in the design of sub-grades, base and surface courses in road construction. It also seems that the common practice of accepting or rejecting soils merely on their liquid and plastic limits is often uneconomical.

Unfortunately, the above work did not deal with the exact physical and mineralogical nature of the clay fractions used in the tests, which appears to detract somewhat from its value. J. Shaw, working with the author in 1943, found that for non-expansive clays of the kaolinite

TABLE 27.—TYPICAL WORKING BEARING PRESSURES FOR VARIOUS SOILS.

Material	Typical working bearing pressure in tons per square foot
Non-cohesive.	
Compact gravel or sand and gravel	5
Loose gravel or sand and gravel	3
Compact coarse sand	4
Loose " "	2
Compact fine "	3
Loose " "	1
Cohesive.	
Very stiff boulder clays, clay shales	6
Stiff clays and sandy clays	4
Firm " " " "	2
Soft clays and silts	1
Very soft clays and silts and peat	Not more than ½
Hoggin (compact)	6
Hard solid chalk	6

type, the addition of clay to a sand-clay soil increases density up to a certain limit, but that if expansive clays of the montmorillonite type were used, a marked reduction in density occurred.

The bearing capacity of uniform homogeneous soils can be inferred within broad limits from the soil type, and Table 27 gives typical values which have been put forward for this purpose. It is, however, always preferable to determine the bearing capacity from laboratory determinations of the shear strength or other properties of each individual soil.

CHAPTER NINE.

THE STABILISATION OF SOILS.

The term " stabilisation of soils " means the improvement of their bearing power by means of added material. Soil stabilisation has been applied mainly to road and aerodrome runway foundations, but no doubt may be used to an increasing extent in connection with other forms of engineering structures. The following are the chief types of soil stabilisation :—

1. *Graded Soil Mixtures.*
   - (a) Without admixture of materials other than soils.
   - (b) With the addition of water-retentive chemicals such as sodium or calcium chloride.
2. *Soils with special stabilising agents.*
   - (a) Bituminous binders such as road oils, bitumens, cut-back bitumens, bituminous emulsions, tars.
   - (b) Portland or other similar cements, or lime.
   - (c) Certain industrial waste products such as resins, resinified molasses, sulphite liquor from the paper industry, or lignin liquor from the wood-pulp industry.
3. *Electro-osmosis.*

General Suitability of Soil for Stabilisation.

Certain soils are unsuited for stabilisation in their natural state. Thus, cohesive soils when dry cannot be effectively pulverised, and when wet become too sticky to mix properly ; wet mixes of such soils are difficult to dry out, and tend to crack badly in doing so. As a first requirement, the soil must contain at least 50 per cent. retained on the No. 200 mesh sieve, while soils with liquid limits exceeding about 40 cannot be stabilised except possibly by heat treatment ; any soil rich in vegetable or other organic matter is not readily stabilised by any known treatment.

Graded Soil Mixtures.

(*a*) *Without admixture of material other than soil.*

These are often known as granular stabilised soils[75], and are used for the construction of lightly-trafficked roads in districts possessing suitable varieties of soil ; they may or may not be given a bituminous cover ; in the case of traffic of any intensity, such a cover is certainly necessary. It should be mentioned here that the question whether the soil-stabilised base is to be covered with a bituminous top or not is one which concerns the design of the soil mix below, since it has been found that many stabilised soils which are satisfactory when exposed freely to the air do not function so well when covered with a black top surfacing ; the latter condition requires less clay in the foundation. Suggested plasticity indexes are :—For base-courses, not more than 6; for surface courses, 4 to 9 ; for sub-bases, not more than 15. Materials with plasticity indexes of from 4 to 6 are suitable for all of these purposes.

The particulars now to be given are based on A.A.S.H.O. Standard Specification for Materials for Stabilised Base Course M56-42, A.S.T.M. Tentative Specifications for Materials for Stabilised Base Course D556-40T, A.A.S.H.O. Standard Specification for Materials for Stabilised Surface Course M61-42, and A.S.T.M. Tentatative Specifications for Materials for Stabilised Surface Course, D557-40T. (See Fig. 93.)

Surface Course Materials. For Type A (Sand-Clay Mortar), the following grading is suggested :—

Passing.	*Percentage by weight.*
1 inch sieve	100
No. 10 mesh sieve	65—100

The material passing the No. 10 mesh sieve should be graded as follows :—

Passing.	*Percentage by weight.*
No. 10 mesh sieve	100
,, 20 ,, ,,	55—90
,, 40 ,, ,,	35—70
,, 200 ,, ,,	8—25

For Type B (Coarse-graded Aggregate—Gravel, Crushed Stone, Slag) the following grading is suggested :—

Passing.	*Percentage by weight.*
1 inch sieve	100
¾ ,, ,,	85—100
⅜ ,, ,,	65—100
No. 4 mesh sieve	55—85
,, 10 ,, ,,	40—70
,, 40 ,, ,,	25—45
,, 200 ,, ,,	10—25

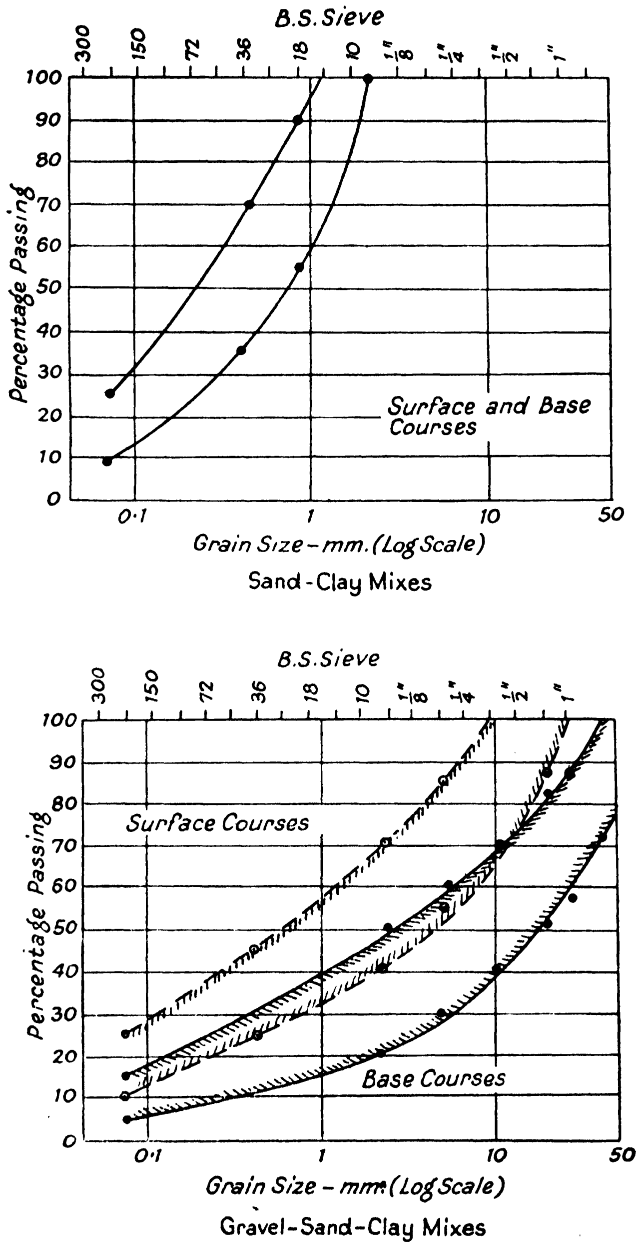


Fig. 93.—Specified limits of soil grading for graded soil stabilisation (American Association of State Highway Officials.)

A higher percentage passing the No. 10 mesh sieve is desirable in the case of mixtures of angular particles as compared with mixtures of rounded particles.

In the case of surface courses, the material passing the No. 40 mesh sieve should have a liquid limit not exceeding 35, with a plasticity index not exceeding 9, while the fraction passing the No. 200 mesh sieve should be less than two-thirds of the fraction passing the No. 40 mesh sieve.

Base-course Materials. For Type A (Sand-Clay Mortar) the grading requirements are as given above for surface courses. For Type B (Coarse-graded Aggregate), the requirements are as follow:—

Percentage by weight passing	Maximum sieve size		
	1 inch	2 inch	3 inch
3 inch sieve	—	—	100
2 ,, ,,	—	100	65—100
1½ ,, ,,	—	70—100	—
1 ,, ,,	100	55—85	45—75
¾ ,, ,,	70—100	50—80	—
⅜ ,, ,,	50—80	40—70	30—60
4 mesh ,,	35—65	30—60	25—50
10 ,, ,,	25—50	20—50	20—40
40 ,, ,,	15—30	5—15	10—25
200 ,, ,,	5—15	5—15	3—10

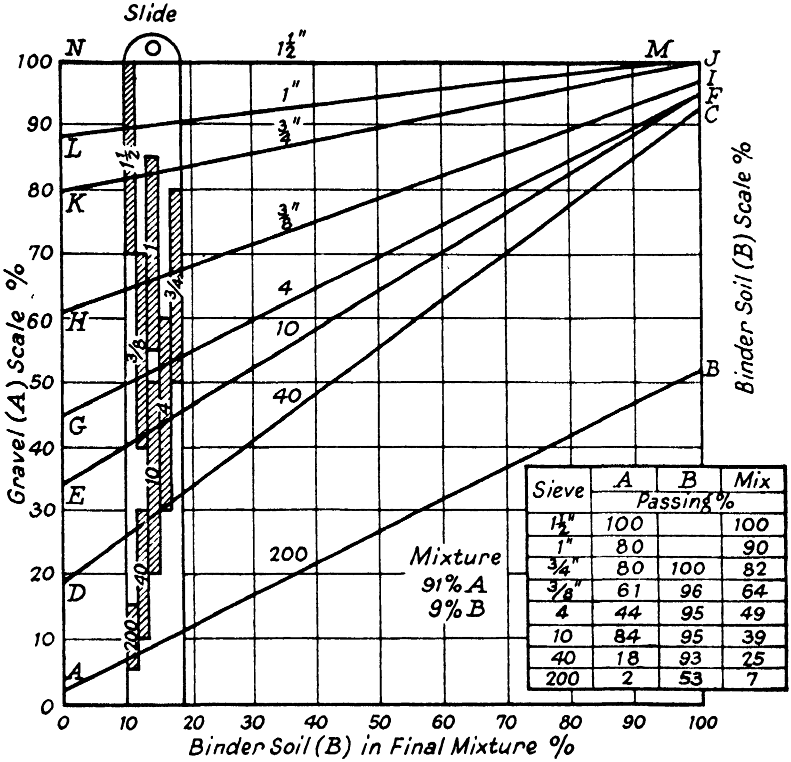
The plasticity index should not exceed 6, the liquid limit should not exceed 25, while the fraction passing the No. 200 mesh sieve should not exceed one-half and in no case two-thirds of the fraction passing the No. 40 mesh sieve.

Sub-base Materials. Type A—Drainage layers or substitute blankets used to the depth affected by frost in place of naturally silty soils which become unstable under frost and subsequent thaw. The material should be non-plastic granular soil with not more than 8 per cent. passing the No. 200 mesh sieve.

Type B—Sub-base suitable for use on soils of low supporting power and as a treatment over plastic soils with a plasticity index exceeding 15 which are subject to detrimental volume change.

The material should be similar to Type A sub-base, should have more than 65 per cent. of the soil passing the 40 mesh sieve retained on the 200 mesh sieve, with a maximum liquid limit of 35 and a maximum plasticity index of 15. In areas not subject to frost action, soil with a liquid limit of 40 and a plasticity index of not more than one-quarter of the liquid limit is suitable.

H. Aaron[76] has deduced a rapid graphical method of proportioning soil mixes (see Fig. 94). A piece of squared paper is pinned on to a



Sieve	A	B	Mix
	Passing %		
1½"	100		100
1"	80		90
¾"	80	100	82
⅜"	61	96	64
4	44	95	49
10	84	95	39
40	18	93	25
200	2	53	7

Fig. 94.—Rapid graphical method for proportioning stabilised soil mixes. (Aaron.)

drawing board, and a movable paper scale is blocked off, as shown in Fig. 94, with the limits of the specified grading marked on it. Pins are stuck in the vertical scales of the squared paper at points corresponding to the percentages passing the various sieves, on the left for the gravel, and on the right for the binder soil. Threads are used to join pin A and pin B, pin C and D, and so on. The movable scale is placed under the threads along the pins at the left-hand side of the diagram, and is moved to the right, keeping it parallel to its original position, until a line (indicated by its left-hand edge) is reached at which the greatest number of threads cross within the limits specified for the various sieve sizes marked on the scale. The intersection of this line with the horizontal scale at the bottom of the sheet indicates the percentage of binder soil to be added to the particular sample of gravel. The grading of the mix formed by this combination of gravel A and binder B is read off on the vertical scale at the points at which this line intersects the lines of the various threads crossing the sheet.

The Michigan State Highways Department also has a method of choosing a suitable mix of gravel and binder by which the ratio of the amounts of two given materials required to match a specified grading is calculated from the summations of the differences between the desired grading and the gradings of coarse material A and fine material B, the desired grading being denoted by grading C (see Table 28). The differences between the percentages passing each sieve for gradings A, B and C are calculated and added without regard to sign, when the ratio of the two summations is the ratio desired. In the table, the average of the specified grading limits have been used except in the case of the fraction passing 200, in which case the lower figure was taken. It will be seen that the summation of the differences between the required grading C and the gradings of the coarse material A and the fine material B is 22 for A and 80 for B, so that $\frac{A}{B} = \frac{22}{80}$, or $B = 3{\cdot}64A$, i.e., one part of B is taken to 3·64 parts of A to produce grading C. The final calculations of the required proportions is given in Table 29. It must, however, be noted that if both components are either finer or coarser in whole or in part than the desired grading, the ratio computed by the above method may be in error. In such cases, it is best to satisfy the grading requirements of the finer sieve fractions only and to neglect those of the coarser fractions.

TABLE 28.—MICHIGAN STATE HIGHWAYS DEPARTMENT METHOD OF CALCULATING PROPORTIONS OF STABILISED SOIL MIXES.

Sieve size	Specified grading, percentage passing	Required grading of mixture C, percentage passing	Grading of soil A, percentage passing	Difference of gradings C and A	Grading of soil B, percentage passing	Difference of gradings C and B
¾ in.	100	100	98	2	100	0
⅜ ,,	60—85	73	65	8	98	25
10 mesh	40—50	45	35	10	79	34
40 ,,	20—35	27	25	2	15	12
200 ,,	10—18	10	10	0	1	9
				22		80

TABLE 29.—MICHIGAN STATE HIGHWAYS DEPARTMENT METHOD OF CALCULATING FINAL PROPORTIONS OF DESIRED STABILISED SOIL MIXES.

Sieve size	Required grading of mixture C, percentage passing (see Table 28)	·78A		+	·22B		=	Final mix percentage passing
¾ inch	100	·78 ×	98	+	·22 ×	100	=	99
⅜ ,,	73	·78 ×	65	+	·22 ×	98	=	72
10 mesh	45	·78 ×	35	+	·22 ×	79	=	45
40 ,,	23	·78 ×	25	+	·22 ×	15	=	27
200 ,,	8	·78 ×	10	+	·22 ×	1	=	8

(*b*) *Addition of Water-Retentive Chemicals.*

In dirt roads, more especially those subjected to long periods of hot, dry weather, water-retentive chemicals such as common salt and calcium chloride have been used for stabilisation purposes. The use of such chemicals does not, however, obviate the necessity for control of the soil grading and soil type used.

It is thought by some engineers that the addition of deliquescent salts, by reason of their water-retentive properties, (*a*) assists compaction, (*b*) retards abrasion of the surface by traffic, (*c*) lessens surface ravelling. It is also thought that frost resistance is increased. But in view of the fact that all such substances are water-soluble, it would seem that any such effects are bound to be of a purely palliative nature.

F. L. Cuthbert[77] claims that material benefit accrues from the addition of such chemicals to dirt roads, especially as regards loss of aggregate from whip-off due to traffic, but detailed information on this subject from independent scientific sources appears to be rather scanty.

Soils with Stabilising Agents.

(1) *Bituminous Materials.*

Bitumen is used in soil stabilisation in two main forms, (a) as an emulsion containing 50 to 60 per cent. of bitumen in a watery medium, (b) as a light cut-back, i.e., a bitumen softened to fluid consistence with volatile solvents, or as a " road oil." Ordinary cut-back bitumen does not readily coat wet aggregate, and special types have been developed in recent years to overcome this difficulty. Tar, which does not suffer from this trouble to the same degree as bitumen, has also been used but to a lesser extent, although in America, where all of these processes have reached their highest development, bitumen has been more widely used, being usually cheaper and more abundant than tar.

Considerable quantities of water have to be added to a soil, more especially a cohesive soil, in order to obtain proper mixing of soil and stabilising agent, and the quantity of water depends not only on the type of soil but also on the local climatic conditions in which the stabilisation is to be undertaken. In humid climates such as that of Great Britain, such construction can usually only be undertaken successfully in the summer months. So far, little has been done in the direction of the successful mixing of dry soil with such binders. The factors involved in soil stabilisation with asphaltic materials have been studied by A. Holmes, J. C. Roediger, H. D. Wirsig and R. C. Snyder[78], who conclude that:

1. A mathematical relation exists between the liquid and plastic limits of a soil and its exudation value or the amount of liquid retained by it under a selected compaction loading. This relationship is

$$y = \frac{A - \log P}{B} x$$

where y = water exudation value, i.e., that quantity of water which a soil can contain without being squeezed out under consolidation or without producing a plastic mixture,

P = compaction loading on a 2 inch diameter by 1 inch high briquette,

A is a constant which is 6·471 for the liquid limit and 7·013 for the plastic limit,

B is a constant which is 7·1 for the liquid limit and 4·4 for the plastic limit,

x = either the liquid limit or the plastic limit.

2. The height of the test specimens should not exceed 1·5 to 2 inches if a proper evaluation of the factors involved is to be obtained.

3. The best distribution of moisture in the soil mass is obtained by wetting beyond the required moisture content and then drying the mass to the correct content.

4. With the exception of lime, chemicals have little effect in improving the stabilising properties of poor soils.

5. The load-bearing strengths of compacted soils can be greatly improved by using non-aqueous liquids instead of water for stabilising agents, although the dry soil density is thereby reduced.

6. Light petroleum distillate oils were found to be especially effective for this purpose.

7. The load-bearing capacity varies according to the 1·5 power of the thickness of the compacted soil. The greater the number of layers used to build up the required thickness of compacted soil, the greater the final strength of the mix and the higher the soil density, while tensile strength rather than shear strength is thought to be the determining factor at the point of failure.

8. In the preparation of soil-bitumen mixtures, good results are obtained when the ratio of the soil fraction passing the No. 10 mesh sieve, but retained on the No. 200 mesh sieve, to the fraction passing the No. 200 mesh sieve is not less than 1·7.

J. S. Jackson[79] has described a process for the stabilisation of soil whereby the material is waterproofed with a small percentage of waxy petroleum oil. Since the presence of excessive amounts of clay, more especially of abnormally active clay, renders the soil liable to absorb excessive amounts of water and so become unstable, a method was worked out for controlling the activity of the clay fraction other than by the addition of sand, which merely upsets the grading. It was found that the addition of a small amount of a flocculating agent such as slaked lime (1 to 2 per cent.) or of aluminium sulphate (0·44 per cent.) greatly reduces the ability of the clay fraction to absorb capillary water. It was also found that in general, for successful stabilisation by this method, 20 to 30 per cent. of the material passing 10 mesh sieve should also pass the 200 mesh sieve, and that at least 60 per cent. of that passing the 200 mesh sieve should be smaller than 50μ. With this type of soil, a good degree of waterproofing has been obtained with 3 per cent. of a suitable waxy oil; about 2 to 4 per cent. of wax, according to the type of oil base, appears to be necessary for the best results. There seems to be no serious danger of damage by frost when soils are stabilised by this type of process, but it is pointed out that drainage of the site is just as important as with other forms of engineering construction.

Stabilisation with bituminous emulsions has been studied by C. L. McKesson[80]. The process consists in replacing the binding action of the clay fraction of the soil by that of the bitumen after break of the emulsion has occurred. When mixing takes place, the emulsion is dispersed in microscopic spheres among the smaller particles of soil, and after break takes place, these spheres are drawn together by surface tension to form water-resistant films of bitumen on the soil grains. The latter are thereby prevented from re-absorbing sufficient water to cause loss of stability, and permanent stabilisation is claimed to occur. The following are the chief points to be observed :—

1. The choice of an emulsion with a rate of break slow enough to permit of adequate mixing before break takes place ; this rate of break must obviously vary with the type of soil used ; the finer the grain-size of the soil, the slower the rate of break.

2. An optimum proportion of emulsion within limits fixed by laboratory stability tests, and thorough mixing of emulsion and soil.

3. Efficient compaction at optimum moisture content to maximum practicable density.

4. Subsequent drying out of the soil mass to the lowest possible water content.

The last of these points has given rise to difficulties both in U.S.A. and in Britain, and attempts have been made to dry the mix to optimum moisture content by spreading in windrows prior to compaction. After this preliminary drying, the material is spread in such a way as to give a compacted layer 2 inch thick, which is immediately rolled with pneumatic-tyred rollers so that maximum density is obtained before the mix has dried to a point below the optimum moisture content ; the final thickness is built up in several such layers.

McKesson[81] had previously shown that soils should contain at least 5 per cent. of material finer than 1μ and at least 20 per cent. passing the No. 200 mesh sieve, and gives the following empirical formula for the binder content, subject to an over-riding minimum of 4 per cent. of binder :—

$$\text{Binder Content} = K(0{\cdot}05a + 0{\cdot}1b + 0{\cdot}35c)$$

where a = percentage of soil passing 200 and coarser than 5μ,
b = as above but finer than 5μ and coarser than 1μ,
c = as above but passing 1μ,
K is a constant which is unity for most types of soils.

It is evident that the above formula does not take into account the net binder content, i.e., the percentage of bitumen remaining after break ; it is also clear that the higher the percentage of fine fractions in the soil, the greater the quantity of binder required, so that it may be economical to add coarse material to the soil before adding the binder. The formula is in any case only *ad hoc*, so that stability tests are required before the final percentage of binder is decided upon in any particular case.

Cut-back bitumens and road oils are also used as stabilising agents for soils, and it has been stated that soils containing as much as 45 per cent.

of material finer than 50μ have been stabilised with rapid-curing cut-backs. L. Muir, W. Hughes and G. Browning[82] give a survey of American practice in this matter, the Asphalt Institute[83], New York, having also published data thereon, but a scientific knowledge of the basic principles involved is still lacking. It is, however, thought that bituminous materials which develop relatively high viscosity after curing give better results than those of low final viscosity, and American specifications for cut-back bitumens are so drafted as to permit the use of the former class of binder. It is suggested that as in the case of emulsions, *ad hoc* tests for water absorption and stability are indicated. Warm, dry weather is best for the laying of mixtures of this kind, and adequate drainage is also needed.

Sand-mix processes using cut-back bitumens have been confined to the use of dry sand in some countries, since normal cut-backs will not coat really wet (as distinct from damp) sand satisfactorily. For this reason, a specially prepared cut-back has been made which will coat wet sands, the only limiting condition then being frosty weather. In South African practice, it was found necessary by F. I. de Waal[84], working with the author, to moisten soils prior to application of the cut-back, the optimum moisture content depending upon the type of soil and of cut-back used.

Tars have also been used as stabilisers in sand-mix construction, and have the advantage that they coat wet sand with greater ease than bitumens, and can be used in warmer weather than bitumens. Tar-sand mixes have been used in U.S.A., in South Africa, and to a limited extent in Britain, and trial lengths indicate that they should be as successful as bitumen-sand mixes. In common with all bituminous surfacings, the binder will deteriorate with age, the binder becoming brittle and lifeless on account of polymerisation, loss of volatile constituents, and possibly oxidation. From 2 to 5 years' service should be obtained before surface dressing becomes necessary.

Soil-Bituminous Roads have been described in some detail in U.S.A.[85] under the following classification :—

(*a*) Soil-bitumen, a waterproofed, cohesive soil system.

(*b*) Sand-bitumen—loose beach, dune, pit or river sand cemented by bitumen.

(*c*) Waterproofed mechanical stabilisation—a well-graded high density soil waterproofed by the uniform distribution of very small amounts of bitumen.

(*d*) Oiled earth—earth which has been made water and abrasion-resistant by the application of slow or medium-curing road oils.

It should be emphasised that all of the above methods are new in soil stabilisation, and are in course of development.

(*a*) *Soil-Bitumen Roads.* The binders used include (i) cut-back bitumens RC1, RC2, RC3, MC2, MC3, complying with one of the following specifications :—A.A.S.H.O. M81-42, M82-42, or with specifications of the Asphalt Institute, New York. (ii) Road oils, grades SC2, SC3. These are covered by Asphalt Institute specifications, and by the following A.S.T.M. tests: D4-42, D88-44, D92-33, D95-40, D113-44, D139-27 and D402-36. (iii) Emulsified bitumen, using A.S.T.M. specification D631-43 (test method D244-42). (iv) Road tars, grades RT3 to RT10, complying with A.A.S.H.O. Specification M52-42 or A.S.T.M. Specification D490-43T.

Soil. The soil used should have a maximum size not exceeding one-third the compacted thickness of pavement, more than 50 per cent. passing the No. 4 mesh sieve, 35 to 100 per cent. passing the No. 40 mesh sieve, not less than 10 per cent. and not more than 50 per cent. passing the No. 200 mesh sieve; the liquid limit should be below 40 and the plasticity index less than 18. The nature of the clay fraction affects the type and quantity of the bitumen required; obviously a kaolinitic clay fraction will be easier to stabilise than a montmorillonitic one. The soil must be moistened prior to the application of the binder.

(*b*) *Sand-Bitumen Base-courses.* The binders used include road tars RT6 to RT10, or cut-back bitumens RC1 to RC3, or emulsified bitumens as given above for soil-bitumen roads.

The sand used should be substantially free from vegetable matter, lumps or balls of clay, or adherent films of clay. In the case of wind-blown or dune sand, up to 25 per cent. passing 200 mesh may be allowed if the material passing the 40 mesh sieve has a field moisture equivalent not exceeding 20 and a lineal shrinkage of not more than 5.

The stability requirements in the case of sand and cut-back bitumens are that the sand, when mixed with $4\frac{1}{2}$ per cent. of cut-back bitumen, should give a Hubbard-Field stability value of at least 1,200 lb. at 77° F. In the case of sand and road tars, the sand used should have a Florida Bearing Value of at least 25 lb. per square inch. This value is obtained as follows :—The sample of sand is oven-dried to constant weight at 110° C. All lumps are broken up, after which 10·5 c.c. of water is added to 600 grams of sand. The water is mixed uniformly with the sand and the mix is compacted in a mould 3 inches in diameter and 3 inches deep under a load of 1,200 lb. A bearing plate or plunger with an area of 1 square inch is placed on the surface of the compacted material near the centre of the specimen. A uniformly increasing load is applied to the plate until the surface of the specimen is definitely disrupted, which usually occurs at $\frac{1}{8}$ inch penetration. The load producing rupture is termed the Florida Bearing Value of the soil.

In the case of sand and emulsified bitumens, the sand should be of such quality that when $1\frac{3}{4}$ per cent. water, expressed as a percentage of the dry weight of sand, is added, it will have a Florida Bearing Value of at least 30 lb. per square inch. It should also be of such quality that when mixed with the quantity of emulsion and mixing water as calculated from the formula below it will have a modified Florida Bearing Value of at least 150 lb. per square inch[86].

$$P = \frac{0{\cdot}43(0{\cdot}05A + 0{\cdot}10B + 0{\cdot}50C)}{\textit{Per cent. residue in emulsion}}$$

where A = percentage of sand retained on a No. 10 mesh sieve,

B = ,, ,, ,, passing a No. 10 and retained on a No. 200 mesh sieve,

C = ,, ,, ,, passing a No. 200 mesh sieve,

P = ,, ,, emulsion expressed as a percentage by weight of dry sand,

$\frac{P}{2}$ = ,, ,, mixing water based on weight of sand.

The emulsion used should comply with A.S.T.M. Specification D631-42.

(2). *Waterproofed Mechanical Stabilisation.*

This term applies to well-graded soils in which only a minimum of binder is necessary. It has been found that a rapid decrease in compressive strength occurs as the moisture content of a mechanically stabilised mixture is increased, due to the fact that the clay binder progressively softens and loses cementing power as its moisture content is increased. Thus, if high bearing capacity is to be preserved, the moisture content of such mixtures must be kept as low as possible, i.e., the road must be rendered as waterproof as possible.

Soil. Optimum gradings are as under :—

Percentage passing	*Maximum*	*Minimum*
$\frac{3}{4}$ inch	100	100
$\frac{1}{2}$,,	100	75
4 mesh	100	53
10 ,,	84	37
20 ,,	70	28
40 ,,	52	22
80 ,,	43	18
100 ,,	28	15
200 ,,	25	13

If complete waterproofing is not possible, the plasticity index should be less than 6; but if adequate waterproofing is possible, this figure may be between 10 and 15. The field compaction should be at least 95 per cent. of Proctor density.

Binder. Some form of RC binder is usually used, 1 per cent. being considered sufficient for dry and 2 per cent. for wetter climates. The viscosity of the binder should not exceed 100 seconds Saybolt-Furol at 122° F., or 26 degrees Engler at 50° C. The waterproofing material can be of any suitable seal coat type.

(3). *Oiled Earth Surfaces.*

These involve the use of silt and clay soils; granular soils are unsuitable for such treatment. The efficacy of the method depends almost entirely upon the surface waterproofing of a soil which contains at the time of treatment a moisture content at or near the optimum percentage for load-bearing capacity. Penetration of the oil into the soil surface will vary with temperature, the type of oil used, and the moisture content and state of compaction of the soil. Dust films on the upper surface of the soil tend to prevent proper penetration of the oil. The soil surface should therefore be freshly cut from a layer laid slightly thicker than the finished thickness in order to expose a surface layer to which the oil can be immediately applied. Penetration under the most favourable conditions will seldom exceed $\frac{3}{4}$ inch to 1 inch, but satisfactory results should be obtained with a penetration of between $\frac{1}{2}$ inch and $\frac{5}{8}$ inch. The binders used can be SC2 cut-back bitumen, though MC1 and MC2 grades have also been used. Tars should not have a higher viscosity than RT2.

(*ii*) *Portland Cement.*

M. D. Catton[87] has classified soils for soil-cement construction into four groups. The first three of these groups showed "very marked hardening," "marked hardening" and "substantial hardening" respectively on treatment with cement, while the fourth group comprised bad soils of limited occurrence not susceptible to stabilisation. *Group* 1 included predominantly sandy soils, *Group* 2 silty soils, and *Group* 3 clayey soils. Soils belonging to the first two groups were found to possess liquid limits below 50, plasticity indexes below 25 and clay contents below 35 per cent. This classification gives only a general guide to the suitability of soils for cement stabilisation.

Cement content is usually based on the results of durability tests (see Chapter 5, A.S.T.M. Designation D559-44); at the Road Research Laboratory, 3 inch test cubes are made up which are stored in damp sand for 7 days and then tested in compression, when suitable soils should give strengths of 250 lb. per square inch or more, although this is not necessarily a sufficient index of quality. Some soils which are rich in dissolved salts or organic content have been found by the author to give a delayed set and hardening due to the presence of such impurities, although the final strength at 28 days may finally prove adequate.

Although laboratory tests are necessary to determine the optimum cement content in any particular case, Catton has suggested the following percentages by volume as a guide for the preparation of trial mixes: Granular soils of the first group, 6 to 10 per cent. of cement; silty soils of the second group, 8 to 12 per cent.; soils of the third (clay) group, 10 to 14 per cent. These figures should be used with a great deal of caution, especially those relating to clay soils, since experience goes to show that soils containing clays of the expansive montmorillonite group (see Chapter 3) cannot be stabilised by any method at present known.

The following are suggested requirements for soil-cement mixtures for base-courses[88]:—

Soil. The best soils for this purpose fall within the following limits:—

Maximum size 3 inches.
Passing No. 4 mesh sieve, at least 50 per cent.
,, No. 40 ,, ,, 15 to 100 per cent.
,, No. 200 ,, ,, not more than 50 per cent.

The liquid limit should not exceed 40, and the plasticity index 18.

Tests on mixes should show the following results:—

1. Losses during 12 cycles of either the standard durability test (see page 136) or the freezing and thawing test should not exceed: for A2 and A3 soils, 14 per cent., for A4 and A5 soils, 10 per cent., for A6 and A7 soils, 7 per cent.

2. The maximum volume during either of the above tests should not exceed the original volume by more than 2 per cent.

3. The maximum moisture content at any time during either test should not exceed that of optimum moisture content.

Process of Soil-Cement Stabilisation.

The soil and cement are mixed when both materials are dry and powdery, just sufficient water being then added to give optimum moisture content for proper compaction. Unlike processes in which bituminous binders are employed, soil-cement stabilisation relies to a considerable extent on the strength conferred by the cementation of the soil grains by the set and hardened cement; the surfacing must, therefore, be completed the same day on which mixing is begun; curing with damp coverings is also necessary, as in ordinary concrete construction.

TABLE 30.—SOIL-CEMENT GROUPS (after M. D. Catton).

	Group 1	Group 2	Group 3	Group 4
Type of Soil	Sandy	Silty	Clayey	Peaty, highly organic; heavy clay
Liquid limit	Less than 50			
Plasticity index	Less than 25			
Clay content (less than 5μ)	Less than 35 per cent.			
Minimum compressive strength	250 lbs. per sq. in. at 7 days			
Permissible loss in durability test	14%	10%	7%	
Maximum permissible volume change	2 per cent.			
Normal cement content (dry weight basis)	6—10%	8—12%	10—14%	

Investigations carried out by the Road Research Laboratory on a number of soils showed that when deleterious organic matter is absent, and provided that the material is fully compacted, the strength of mixes made from a given soil depends upon the water-cement ratio in much the same way as in the case of concrete. Thus, maximum strength is obtained with the smallest water-cement ratio consistent with full compaction. In the field, the aim is always to obtain maximum compaction, and Catton has shown that reasonable agreement exists between the moisture contents at which maximum strength, maximum durability and maximum density occur, although with some soils, adequate durability is obtained at moisture contents slightly above that giving maximum density.

After a few months' drying out, it is advisable to apply a surfacing to the soil-cement construction to avoid abrasion of the surface by traffic, a light multi-seal armour-coat or other bituminous surfacing being suitable for the purpose.

It should be noted that soil-cement construction is liable to craze and to crack, while sandy soils tend to form horizontal shear planes under ordinary methods of compaction, vibration being the best method of avoiding this (see Chapter 7).

TABLE 31.—SOME ENGINEERING CHARACTERISTICS OF SOILS.

Soil Type	Properties			Possibility of stabilisation by means of :—			
	Settlement after Construction	Quicksand Phenomena	Frost-Heaving	Ground water lowering	Cement Grouting	Silicate and Bitumen Injection	Compressed air[3]
Gravel	None	Impossible	None	Possible	Possible	Unsuitable	Possible[4]
Coarse Sand	,,	,,	,,	Suitable	Possible only if very coarse	Suitable	Suitable
Medium Sand	,,	Unlikely	,,	,,	Impossible	,,	,,
Fine Sand	,,	Liable	,,	,,	,,	Impossible in very fine sands	,,
Silt	Occurs	Liable[1]	Occurs	Impossible[2]	,,	Impossible	,,
Clay	,,	Impossible	None	,,	Only in stiff fissured clay	,,	Used only for support

(1) This is true only of very coarse silts and silty sands.

(2) Ground-water lowering has been applied to silts by electric osmosis, which assists drainage.

(3) The use of compressed air is restricted to depths where the head of water is less than 100 feet. Freezing has been used for sinking shafts through greater depths of fine sand and silt.

(4) If compressed air is used in gravel, the loss of air is great and progress is slow ; in the case of a tunnel, the face must be sealed with clay.

(*c*) *Waste Products* such as resins, sulphite liquors, etc., have been tried during World War II as a means of stabilising soils, more especially in U.S.A., where resinous products have been studied[89]. The results of these studies were as follows : —

Treatment was considered satisfactory if (a) it limited the water absorption after 24 hours by capillary action in test specimens previously dried to 55 per cent. of optimum moisture content, to 75 per cent. optimum moisture content, (b) after full submersion for 24 hours similar specimens had an unconfined compressive strength equal to that of unsoaked test specimens of untreated soil. It was also concluded that : —

1. There is no simple laboratory test which will determine whether the use of a given water repellent will be satisfactory.

2. The use of the acidity value of a soil as a guide to determine whether a soil can be satisfactorily waterproofed is not recommended.

3. These repellents are not recommended for clean cohesionless sands and gravels.

4. They do not prevent dust formation.

5. They have great possibilities for the stabilisation of sand-clays, sand-clay gravels, and soils with a relatively high plasticity index. The author has tried the use of certain waste products such as sulphite liquors in this connection, but the results were not satisfactory.

CHAPTER TEN

THE SOIL MECHANICS OF CUTS, FILLS, RETAINING WALLS AND TRENCHES.

Cuts and Fills.

The principles of soil mechanics involved in the design of cuts and fills have been dealt with by L. A. Palmer and E. S. Barber[90], who consider the design of the subsoil below fills.

Case 1. Fill on Good Under Soil.

Fig. 95 shows the pressure diagram of the vertical cross-section of a fill. The weight of fill material on a square foot of the surface of the supporting soil is $wH = p$, where w = average weight of fill per cubic

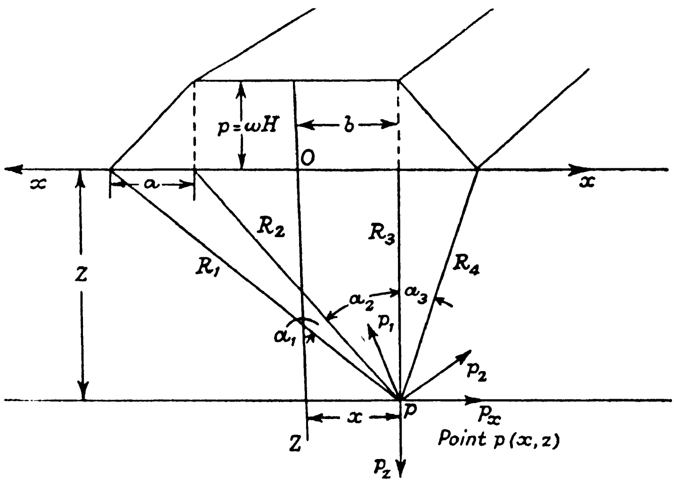


Fig. 95.—Pressure diagram of the vertical cross-section of a fill.

foot and H = height of fill. The major principal stress p_1 at point P below the fill bisects the angle a_2 and the minor principal stress p_2 is perpendicular to p_1. The vertical normal stress at this point is p_2 and the horizontal normal stress p_x, the shearing stress corresponding to p_z and p_x being s_{xz}. The maximum shearing stress at P is $\frac{p_1 - p_2}{2}$. According to the bulb of pressure theory (see Chapter 1), for a 45° slope, the greatest value of $s_{max} = 0{\cdot}31p$, distant $\frac{3a}{2}$ below point O, on OZ. If $a = 2b$, $s_{max} = 0{\cdot}3p$ and is $0{\cdot}96a$ below the base of the fill. For any type of fill, between $a = b$ and $a = 2b$, s_{max} = about $0{\cdot}3p$, and under-soils which have cohesions equal to or greater than $0{\cdot}3p$ furnish ample support for the fill.

If, however, $0{\cdot}3p$ exceeds the shearing strength of the soil, failure may still not occur, because the soil inside the $0{\cdot}3p$ danger zone of the bulb of pressure is surrounded and confined on all sides by soil under lower stress. Thus, the material in the plastic zone may yield to some extent and transmit to the adjoining soil that part of the load which it is unable to resist itself. But if a plastic zone develops beyond this point, then the theoretical stress diagram shown in Fig. 95 no longer applies.

Consider a fill 20 feet high, constructed of a soil with a cohesion value of 200 lb. per square foot, the angle of internal friction ϕ being 5°, unit weight of soil 90 lb. per cubic foot, which is to be built on under-soil for which the cohesion value is 800 lb. per square foot, the angle of internal friction $\phi = 15°$, and the unit weight of under-soil 110 lb. per cubic foot, then $p = wH = 90 \times 20 = 1{,}800$ lb. per square foot. The greatest shearing stress at any point in the under-soil is $0{\cdot}3p = 0{\cdot}3 \times 1{,}800 = 540$ lb. per square foot, which is less than 800 lb. per square foot, the cohesion of the under-soil. (This calculation disregards the term $p_n \tan 15°$, the additional factor in Coulomb's formula for the value of the shearing strength s.) The undersoil is therefore safe for the case given.

Case 2. Fill on Doubtful Under-soil.

The principles enunciated above can also be applied to the case of a fill constructed on doubtful under-soil (see Fig. 96). As an example, consider a case in which the physical properties of the fill material are $c = 800$ lb. per square foot, the angle $\phi = 15°$ and $w = 100$ lb. per cubic foot. If in Fig. 96 the supporting power of the undersoil is less than the unit load to which it is subjected, failure will take place along the surfaces MLK, AK, DK. The figure shows the application of H. Prandtl's theory of plastic equilibrium[91]. The section $EFIH$ has the same area as the section $ABCD$, and when failure of the under-soil takes place, Zone I moves down bodily, shearing at the planes AK, DK. Zone II undergoes a combination of rotation and sliding along the logarithmic spiral KL, while zone III moves outwards and upwards,

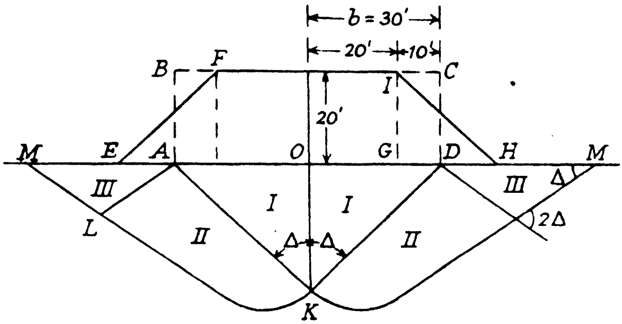


Fig. 96.—Zones of plastic flow below fill constructed on doubtful undersoil.

shearing along the plane LM. The radial line drawn from either of the points A or D to any point on the spiral LK makes a constant angle 2Δ with this curve, the angle α being equal to $45° - \frac{\phi}{2}$.

If ground-water level is at MM, w_1 = effective unit weight of soil = buoyed weight of a cubic foot of undersoil = 100 − 62·5 = 37·5 lb. per cubic foot.

The formula deduced by Prandtl in computing q, the bearing capacity of the undersoil is as follows :—

$$q = \left(c \cot \phi + w_1 b \cot \Delta \left[\frac{1 + \sin \phi}{1 - \sin \phi} e^{\pi \tan \phi} - 1\right]\right),$$ b being the half-width of the fill, whence in this case

$$q = \left(200 \times 11{\cdot}43 + 37{\cdot}5 \times 30 \times 1{\cdot}091 \left[\frac{1{\cdot}0 + 0{\cdot}087}{1{\cdot}0 - 0{\cdot}087} e^{0{\cdot}0875\pi} - 1\right]\right)$$

$$= 1990 \text{ lb. per square foot.}$$

The factor of safety is thus $\frac{1990}{2200} = 0{\cdot}9$, since the bearing load is $20 \times 110 = 2{,}200$ lb. per square foot. This factor of safety may be considered as too small, and it can be increased either by increasing the width of the fill or by decreasing its height.

Foundations Under Cohesive (" Rigid ") Fills.

The distribution of stress in homogeneous soils under cohesive (so-called " rigid ") fills has been deduced by S. D. Carothers, and working formulæ developed therefrom. Thus for side slopes of between 1 to 1 and 2 to 1 in the fill, the maximum shearing stress produced in the soil below the fill is approximately $0{\cdot}3P$, where P is the unit pressure at the point of maximum stress; P is the unit weight of soil times the height of the fill.

If a rigid stratum of rock occurs at a relatively short distance below the base of the fill, the supporting soil is not homogeneous, and the theory of plastic equilibrium should be applied. If the effect of internal friction is applied, it can be shown that the supporting soil is just stable if the total load $P = \frac{b}{h}.c$, where

b = width of fill at its base,
h = depth of rigid stratum below the surface of the supporting soil,
c = unit cohesion.

In this case P is the load on a strip of unit width at the point of maximum stress. This point is under the centre of the fill, and is due to the effect of load from the whole width of the fill. The stress distribution is triangular, and the load P can be shown to be equal to the weight of a column of soil whose height equals the height of a triangle having the same area and width of base as that of the cross-section of the fill. Fig. 97 shows this geometrical relationship. Then,

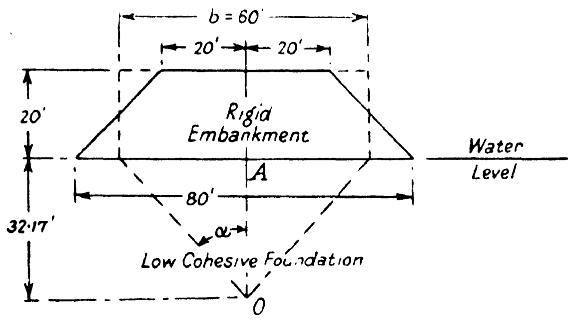


Fig. 97.—" Rigid " fill supported by undersoil of low cohesion.

if w = unit weight of soil in fill,
c = unit cohesion of soil in foundation,
q = supporting value of foundation $= \frac{b}{h}.\ c.$
$P = wH$, where area $ABCD$ = area AED in Fig. 98.

For a factor of safety of 1, $P = q$ and $wH_1 = \frac{b}{h}.c$

This condition of plastic equilibrium should not be assumed if h is less than $\frac{b}{4}$. As h decreases, the supporting value of the foundation increases, and for small values of h, q possesses very large values. For reasons of safety, q should be limited to a value not exceeding $4c$.

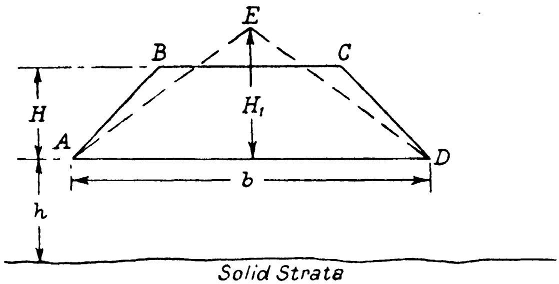


Fig. 98.—Trapezoidal fill replaced by equivalent triangular fill.

Thus, in the case of a fill carrying a roadway 40 feet wide (to the top of the side slopes), the height of the fill being 20 feet, and the side slopes $1\frac{1}{2}$ horizontal to 1 vertical, weight of fill soil 100 lb. per cubic foot, built on a cohesive foundation soil in which c is 800 lb. per square foot and ϕ is 6°, the supporting value of the foundation is obtained as follows :—

The unit load at the base of the fill is $20 \times 100 = 2{,}000$ lb. per square foot, and the maximum shearing stress in the foundation soil $= 600$ lb. per square foot. Since this is less than $c = 800$ lb. per square foot, the foundation to the fill is stable. Had the fill been 30 feet high, the maximum shearing stress would have been 900 lb. per square foot, and further investigation by Prandtl's method would have been necessary.

If a rigid stratum is assumed to exist at a depth of 25 feet below ground, it is seen that the cross-sectional area of the fill is $\frac{100+40}{2} \times 20 = 1{,}400$ square feet, the height of the equivalent triangle being 28 feet. The load P is then 2,800 lb. per square foot. The supporting value of the foundation soil is $q = \frac{b}{h} \cdot c = \frac{100}{25} \times 800 = 3{,}200$ lb. per square foot. Since P is less than q, the foundation soil is still stable. The apparent factor of safety here is $\frac{q}{P} = \frac{3{,}200}{2{,}800} = 1{\cdot}14$. Since the resistance obtained from the angle ϕ in Coulomb's formula for the shear value of a soil has not been taken into account, the true factor of safety will in fact be greater than $1{\cdot}14$.

Terzaghi's Method for Calculating the Stability of Fills.

Although it has been assumed by some workers that fills may act as rigid bodies, it has long been realised that a loose granular fill would probably not settle intact, but that the action would be more nearly as shown in Fig. 99, the central wedges below the fill remaining intact,

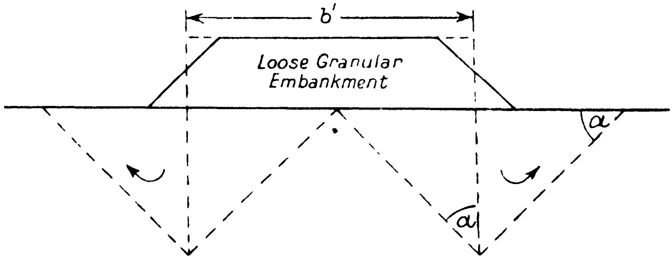


Fig. 99.—Probable movement of undersoil below loose granular fill. (Terzaghi.)

while the two side wedges move outwards and upwards as the fill divides near its centre. Terzaghi's equation for these conditions is as follows :—

$$q = \frac{2c}{\tan\alpha \sin^2\alpha} + \frac{wb'}{4\tan\alpha}\left[\frac{1}{\tan^4\alpha} - 1\right] + \frac{w'd'}{\tan\alpha}$$

where q = bearing capacity of undersoil, d' = depth of fill, b' = width of fill (average of top and base), w' = unit weight of soil in fill, w = unit weight of undersoil, c = cohesion and ϕ the angle of internal friction of undersoil, $\alpha = 45° - \frac{\phi}{2}$.

Neglecting at first the effect of the right-hand term $\frac{w'd'}{\tan^4\alpha}$ in the above equation, this term concerning only surcharge, the value of q can be found graphically by means of Figs. 100 and 101. Thus, in Fig. 100, the quantity $\frac{2c}{\tan\alpha \sin^2\alpha}$ is designated as M-value, while in Fig. 101, the quantity $\frac{wb'}{4\tan\alpha}\left[\frac{1}{\tan^4\alpha} - 1\right]$ is designated as N-value, the value of q (neglecting the third term in Terzaghi's equation) being the sum of these values.

Thus, in the case of a fill 40 feet wide at its top and 20 feet high, with side slopes of 1 to 1, the weight of its soil being 110 lb. per cubic foot, supported by an undersoil weighing 100 lb. per cubic foot with a cohesion value of 300 lb. per cubic foot and an angle of internal friction of 4°, the fill material being granular, the calculation is as follows :—

The fill is 80 feet wide at its base, this width giving a width of equivalent rectangular section of 60 feet = b', the factor wb', therefore, being equal to 100 × 60 = 6,000. From Fig. 100, the values c = 300

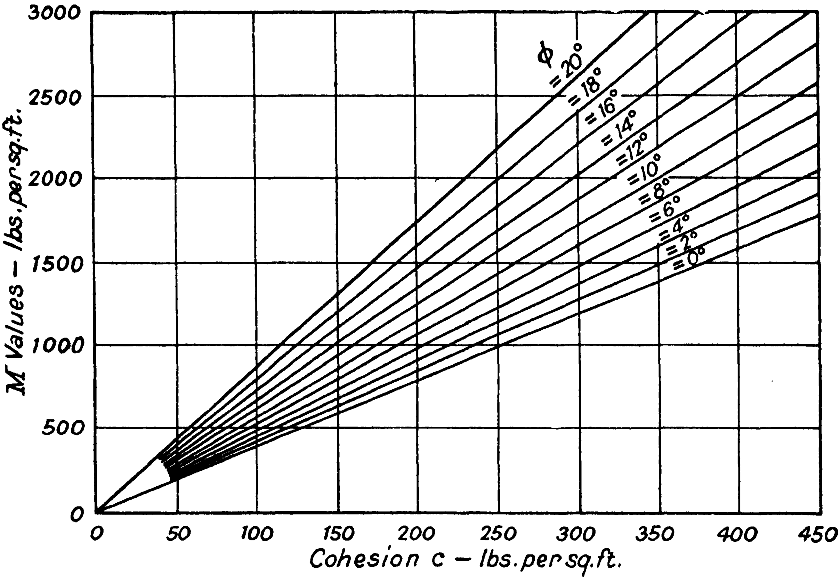


Fig. 100.—Chart for finding M-value for term $\frac{2c}{\tan a \sin^2 a}$ in Terzaghi's equation for the stability of fills.

and ϕ = 4° give an M-value of 1,375 lb. per square foot. From Fig. 101, when wb' = 6,000 and ϕ = 4°, the N-value is 520 lb. per square foot. The weight of the fill is 20 × 110 = 2,200 lb. per square foot, and the total value for q is 1,375 + 520 = 1,895 lb. per square foot, so that the undersoil is deficient in supporting power to an extent of 2,200 − 1,895 = 305 lb. per square foot.

This deficiency can be corrected by applying a surcharge at the toe of the slope of the fill ; the value of this surcharge is given by the right-hand term of the Terzaghi formula above, $\frac{w'd'}{\tan^4 a}$.

A graphical method of finding this term is given in Fig. 102 ; from this figure, it is seen that the deficiency of 305 lb. per square foot, divided by the unit weight w' of the surcharge material (assumed in this case to be equal to that of the fill), gives $\frac{305}{110}$ = 2·8, whence for ϕ = 4°, the required depth of surcharge for stability is 2·09 feet.

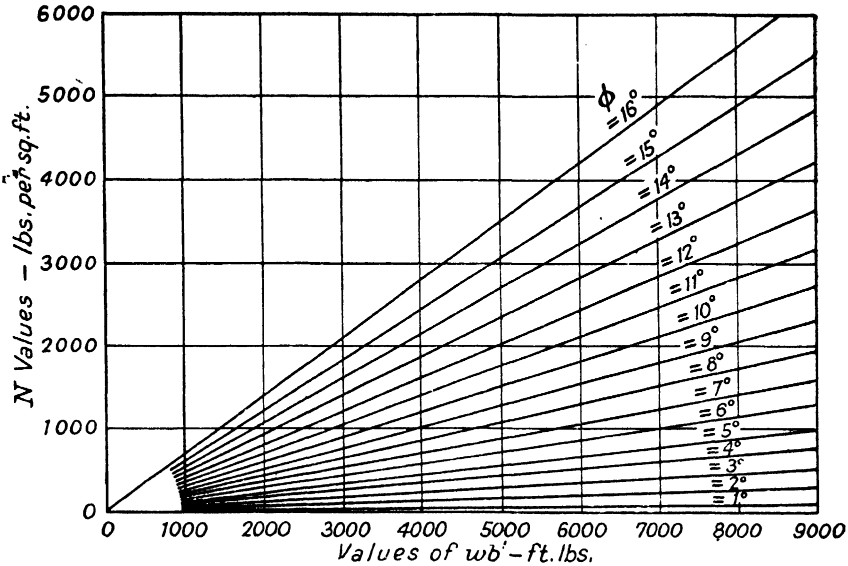


Fig. 101.—Chart for finding N-value for term $\frac{wb'}{4\tan a}\left[\frac{\quad}{\tan^4 a}-1\right]$ in Terzaghi's equation for the stability of fills.

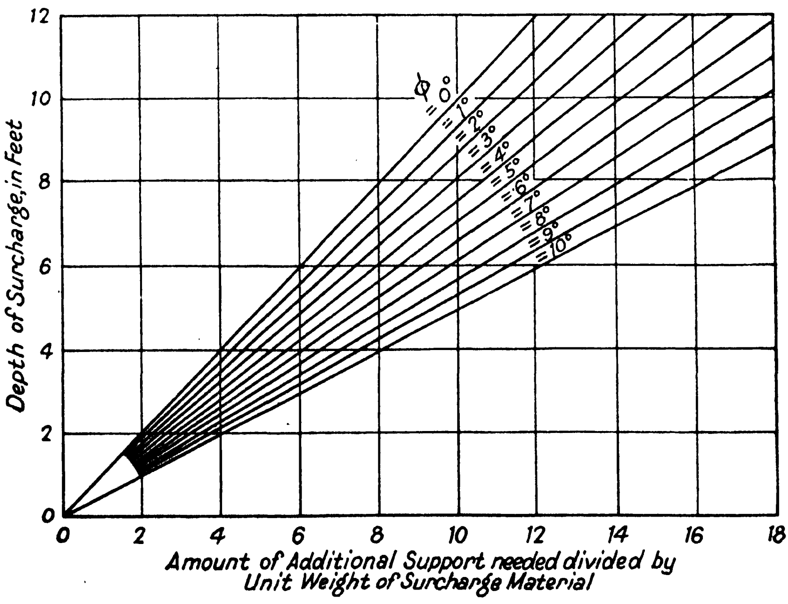


Fig. 102.—Chart for finding value of third term $\frac{w'd'}{\tan^4 a}$ in Terzaghi's equation for the stability of fills.

The width of the surcharge load depends upon the position of the point of intersection of the shear plane LK (see Fig. 96, page 218) with the surface of the supporting soil. The distance from the centre line of the fill to the point of intersection K of these surfaces is given by $A = \frac{b'}{2}(\cot^2\alpha + 1)$, where A is the distance from the centre line of the fill to the outside point of the surcharge load, and b' and α have the same meanings as before. Some values of the factor $(\cot^2\alpha + 1)$ are given in Table 32.

TABLE 32.—SHOWING VALUES OF FACTOR $(\cot^2\alpha + 1)$ FOR DIFFERENT VALUES OF THE ANGLE OF INTERNAL FRICTION ϕ.

Angle of Internal Friction ϕ	Value of factor $(\cot^2\alpha + 1)$
0°	2·000
1°	2·036
2°	2·072
3°	2·110
4°	2·150
5°	2·191
6°	2·233
7°	2·278
8°	2·323
9°	2·371
10°	2·420

In the above case, $b' = 60$ feet and $\phi = 4°$, so that $A = \frac{60}{2} \times 2{\cdot}150 = 64\frac{1}{2}$ feet. Since the width of the fill at its base is 80 feet, the extension should project $64{\cdot}5 - \frac{80}{2} = 24{\cdot}5$ feet beyond the toe of the slope.

If ground water is present at or near the base of the fill, the effect of buoyancy on the supporting soil must be taken into account. No factor of safety has been introduced into the above calculation ; a common value for such a factor would be, say, $1\frac{1}{2}$ times the load due to the weight of the fill.

It should be pointed out that the above methods are not mathematically precise ; they are intended only as a rough guide to assist engineers who have to deal with the design of fills.

The Stability of Earth Slopes.

Early textbooks and reference books give angles of repose for various types of soil regardless of the fact that every cohesive soil can, under certain moisture conditions, stand in vertical banks at a height which is dependent on the nature of the soil. W. Fellenius and J. Olsson, working in Sweden in 1922, produced the most complete analysis of landslides and of soils subject to slides published up to that date, and this Swedish method for finding the stability of slopes is based on the empirical fact that the profile of the surface of sliding always approaches the shape of a circular arc (see Fig. 103). The soil begins to fail by tension at c to a depth $\frac{2c}{w}\tan\left(45^\circ + \frac{\phi}{2}\right)$, where c denotes the cohesion and w the unit weight of the soil. Below that depth, sliding occurs

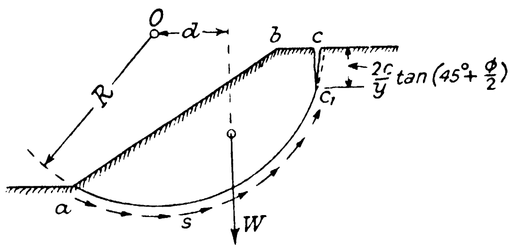


Fig. 103.—Showing profile of probable surface of sliding in cohesive soil forming a fill.

along the curved surface ac_1, after the shearing resistance of the soil is exceeded. By taking moments about O, the centre of the arc ac_1,

$$Wd = \frac{l\bar{s}R}{G_s}$$

$$G_s = \frac{l\bar{s}R}{Wd}$$

where W = total weight of sliding earth,
$\bar{s}$ = average shearing resistance per unit of area of the surface of sliding,
G_s = factor of safety against sliding.
l = length of arc ac_1.

The position of the surface of sliding for which the factor of safety G_s is a minimum, must be determined by trial and error, and this is one of the disadvantages of the method, since much time is involved in this process. It will be seen that if a soil slide takes place either before or after the fill is brought to its final height, then $G_s = 1$, and the above equation can be used for finding $\bar{s}$. When $\bar{s}$ is known, the section of the fill can be modified so as to satisfy any chosen value of G_s.

In the case of soil structures or natural banks consisting of fine-grained soils having very little cohesion, it is evident from the general principles of soil mechanics that the most dangerous conditions exist during rainstorms, while in the case of earth dams, the most critical conditions occur on the downstream side when the reservoir is full.

In the case of a mass of soil with very little cohesion and subject to seepage forces from the impounded water, the shearing resistance $s = (p - hy_0) \tan\phi$, where p is the total normal pressure of the weight of soil and water located above the potential sliding surface; the hydrostatic pressure hy_0 is found from a hydrostatic flow-net. ϕ can be found from shear tests, and as soon as the value of s is known for every point along the surface of sliding, the average shearing resistance $\bar{s}$ can be found by graphic integration. The formula given above is then used to find the factor of safety G_s.

Cohesive soils present a more difficult problem, but if the shear strength of the material *in situ* can be determined with some degree of accuracy, the principles evolved above apply.

The ϕ-Circle Diagram.

This construction, known as the ϕ-circle or friction-circle diagram, is shown is Fig. 104. The equation for equilibrium of the slope of the bank is :—

$R\Sigma T = R\Sigma N \tan\phi + RLc$, where R is the radius of the most dangerous sliding circle, T is the tangential stress for such a circle, N

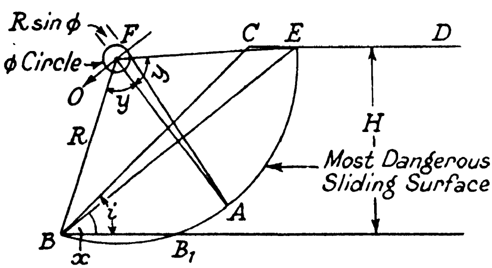


Fig. 104.—ϕ-Circle or friction circle diagram.

is the stress normal to the tangential direction, L is the length of the entire arc of the circle, and c is unit cohesion. $R\Sigma T$ is the shearing stress moment, the right-hand terms in the above equation being the resisting moment. The critical circle in Fig. 104 has a radius equal to $R \sin\phi$, and

$$c = \frac{T - \tan\phi\Sigma N}{L}.$$

The ϕ-circle or friction-circle construction shown in Fig. 104 has been further developed by D. W. Taylor[92], who gives details (see Table 33) of various common numerical values for the angles i, ϕ, x and y, and corresponding values for the stability number S (see page 227).

TABLE 33.—SHOWING TAYLOR'S STABILITY NUMBERS FOR COMMON ANGLES OCCURRING IN THE SLOPES OF BANKS (See Figure 105).

Angle i	Angle ϕ	Angle x	Angle y	Stability Number S
90°	5°	50°	14°	0·239
45°	0°	—	—	—
—	5°	31·2°	42·1°	0·136
—	10°	34°	39·7°	0·108
—	5°	36·1°	37·2°	0·083
30°	10°	25°	44°	0·075
15°	5°	11°	47·5°	0·070

From a fill section drawn to scale, and the data given in Table 33, the value of the chord BE (Fig. 104) can be found. Radial lines R and R are drawn from points B and E to intersect at an angle $2y$ at the centre O of the ϕ-circle, which lies on the perpendicular bisector of chord BE.

It is also seen from Fig. 104 that the circle BB_1E cuts into the undersoil below BB_1. The usual theory of the sliding circle assumes that the soil traversed by the most dangerous sliding circle is uniform throughout, and this simplifies an otherwise most complicated problem. It is also usual to extend this assumption to the undersoil, though actually the fill would tend to shear along B_1AE only, and the slide over the surface BB_1 would be in the nature of a detritus slide. The construction given in Fig. 104 is permissible when the most dangerous circle of sliding passes through the toe of the slope, which happens when $n = \frac{1}{2}(\cot.x - \cot.y - \cot.i + \sin\phi \operatorname{cosec}.x \operatorname{cosec}.y)$ is either negative or zero.

Taylor's[92] method of design of side slopes of fills has the object of eliminating the lengthy trial and error procedure of the graphical slip-circle construction mentioned above. A stability number $S = \dfrac{c}{FwH}$ is utilised for this purpose, where c = shear strength of soil, F = factor of safety, w = unit density of soil, and H = height of bank. The curves shown in Fig. 105 refer to various cases of uniform slopes in homogeneous materials. Four cases are considered :

(*i*) Slope in homogeneous material.

(*ii*) As (*i*) above, but slip-circle limited by hard layer of material below foot of slope (see (b) in Fig. 105).

(*iii*) As (*i*) above, but slip-circle constrained by underlying strata in such a way that it passes through the toe of the slope (see (a) in Fig. 105).

(*iv*) As (*iii*) above, but slip-circle limited by hard layer of material below foot of slope.

Case (*i*). The value of the stability number S for any given slope is given by the full lines of the curves shown in Fig. 105 for various

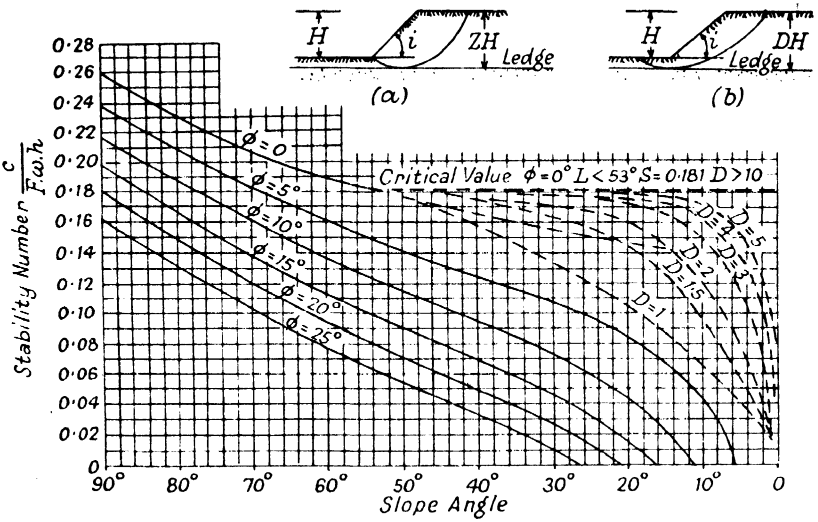


Fig. 105.—Stability of earth slopes. (D. W. Taylor.)

values of ϕ ; in the case of clays, $\phi = 0°$, and the value of S is constant for all values of the side slope i less than 53°, and is equal to 0·181.

Thus, in the case of a slope 20 feet high in clay, with a side slope i = 3 horizontal to 1 vertical ($\tan i = \frac{1}{3}$, and $i = 18{\cdot}5°$), the soil density being 110 lb. per cubic foot, if a value for the factor of safety F is taken as 1·2,

$\frac{c}{FwH} = 0{\cdot}181$, whence $c = 0{\cdot}181 \times 1{\cdot}2 \times 110 \times 20$ lb. per square foot = 458 lb. per square foot, and the soil must possess at least this value for c in order that the bank may be stable.

Case (*ii*). This is similar to Case (*i*) except that the slip surface is limited by a layer of hard rock at a depth DH below the top of the bank (see (b) in Fig. 105), D being known as the " depth-factor." The values of D for this case are shown by dotted curves in Fig. 105, but for the case of cohesive soils ($\phi = 0°$) only.

Thus, in the case of a clay bank 60 feet high with an average shear strength c of 1,000 lb. per square foot and a soil density of 120 lb. per cubic foot, the side slope of the bank is to be $2\frac{1}{2}$ horizontal to 1 vertical. It is found that there is hard rock at a depth of 30 feet below the supporting soil surface. The stability of the bank can be estimated by calculating the factor of safety F. The depth factor $D = \frac{30 + 60}{60} = 1{\cdot}5$. The side slope $i = 22°$, and from the curves shown in Fig. 100, $S = 0{\cdot}15$, whence $F = \frac{1{,}000}{0{\cdot}15 \times 120 \times 60} = 0{\cdot}926$, so that in this case the bank is unstable.

Case (*iii*). In this case, the slip-circle is constrained to a position which passes through the toe of the slope. For homogeneous soils with a value for $\phi = 0°$, this is the same case as that for a simple side slope as for Case (*i*).

Case (*iv*) is similar to Case (*iii*) except that a ledge of hard rock exists at a depth ZH below the top of the bank. One case only is given in Taylor's curves (Fig. 105), i.e., when $Z = 2$.

Thus, in the case of a road cutting 30 feet deep, with a side slope of 2 horizontal to 1 vertical, constructed in a clay soil whose density is 110 lb. per cubic, it is found that a ledge of rock exists 60 feet below the top of the cutting, and that the slope of the cutting has failed by sliding. In order to re-design the bank, it is necessary to calculate the shear strength of the failed soil. In this case, $i = \tan^{-1}0{\cdot}50 = 26{\cdot}5°$, and $Z = 2$. From Fig. 100, $S = 0{\cdot}152$, whence $c = 0{\cdot}152 \times 110 \times 30 = 502$ lb. per square foot.

Buoyancy of Fills.

The methods given above for finding the stability of fills assume a cohesive saturated soil with dry side slopes and without water seepage. If the fill is continuously submerged, as in the case of a fill across a lake, the buoyancy effect of the water must be taken into account, and this is done by subtracting unit weight of water from unit weight of soil, thus reducing the value of w used in the computation of the stability number S, and the values of c and of ϕ must be computed under similar conditions.

The worst case of this kind is probably that of a fill which is submerged long enough to become saturated and which is then subjected to a sudden fall in water level. In this case the weight of water in the fill adds to the load of the fill on the undersoil, while ϕ is decreased on account of the holding apart of the soil grains by the included water in the fill. A practical method of allowing for this possibility is to use a corrected value of ϕ, and this can be done by using an equation such as :—

$$\phi_w = \left[\frac{G - 1}{G + e}\right]\phi,$$

where ϕ_w = corrected value of ϕ,
ϕ_d = angle ϕ with desired factor of safety,
G = specific gravity of soil grains,
e = voids ratio of soil.

Taylor's charts are directly applicable to ordinary fills. In the case of cuts in side-long ground (i.e., ground with a cross-slope) the slope of the ground in which the cut is made may continue beyond the slope of the cut. Since the curve of failure breaks above the top of the slope, allowance should be made in the calculation for this additional height, the following equation having been used for the purpose :—

$$F_s = F + [0{\cdot}60 - \sin\phi]T$$

where F_s = increased factor of safety,
F = desired ,, ,, ,,
ϕ = angle of internal friction,
T = tangent of cross-slope of ground in which cut is made.

Typical Slopes in Cuttings.

Typical slopes in cuttings formed in various types of material are given in Table 34, but it should again be emphasised that the safe slopes of cohesive soils depend on their shear strength and on the height of the cut.

TABLE 34.—TYPICAL SLOPES IN CUTTINGS.

Type of ground	Typical safe slope in excavations
Igneous rocks in sound condition	Almost vertical
Slates ; schists ; hard shales ; hard chalk	½ horizontal to 1 vertical
Thinly bedded limestones, sandstones and mudstones	¾ ,, ,, 1 ,,
Clay shales ; friable sandstones	1½ ,, ,, 1 ,,
Gravel and sand	2 ,, ,, 1 ,,
Fine sands	2½ ,, ,, 1 ,,
Clays and silts	See text.

The slopes given in Table 34 apply only to strata with more or less horizontal bedding planes, and to rocks unlikely to be affected by exposure to weathering. The values given are approximate typical values only, and are intended merely as a rough guide in the absence of definite information based on laboratory examination of actual soils.

The Soil Mechanics of Retaining Walls.

This subject has been summarised by A. L. Little[93]; it should be pointed out that the present volume is not intended to deal with the design of structures other than soil structures, so that only the latter aspect of the subject will be covered.

The least satisfactory aspect of retaining wall design is the estimation of the load caused by the soil backing behind the wall, chiefly because it is only in recent years that laboratory methods have been developed for the determination of the shear strength of the impounded soil, and any method of design which does not take this factor into account cannot be regarded as fundamentally sound.

Fig. 106 shows the cross-section of a rigid retaining wall together with the forces acting in the soil mass behind the wall. The soil exerts

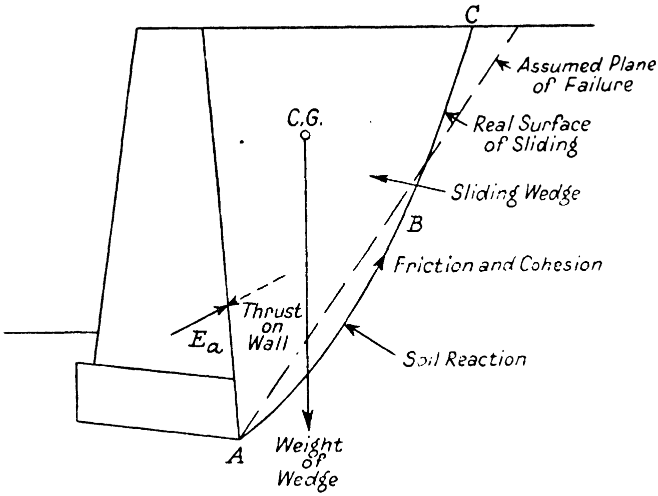


Fig. 106.—Mechanics of " Active Wedge " of soil behind a retaining wall.

a thrust E_a on the back of the wall. If as a consequence of this thrust the wall yields slightly, the soil will tend to move towards the back of the wall, and in so doing will mobilise its powers of cohesion and internal friction along the arc ABC, and this in turn will have the effect of reducing the thrust on the wall. The amount of movement of the soil necessary to mobilise its full cohesion and internal friction depends upon the type of soil, but if enough movement can take place, then the thrust E_a will reach a minimum value which is known as the " active pressure " just before breakdown of the soil along the arc ABC occurs. Conversely, if the wall is pushed against the soil, the cohesive and frictional properties of the soil will resist motion in a reverse direction, and just before breakdown of the soil takes place, the thrust E_p reaches a maximum known as the " passive pressure." The surface of sliding ABC is usually shown curved as in Fig. 106, but for the sake of simplicity a straight line surface of sliding (shown dotted in figure) can be assumed, and has been found to give results in the case of the active thrust which are within a few per cent. of the probable value.

Classical Earth-Pressure Theories.

In 1773, Coulomb developed a theory for the thrust caused by soil on a vertical wall with a horizontal backfill surface, using a straight line assumption for the surface of sliding. His expression is too involved for direct use, but Poncelet and Merriman later developed the theory further, arriving at the well-known formula

$$E_a = \tfrac{1}{2}wh^2 \frac{K_a}{\sin\alpha \cos\delta}$$

where E_a = active pressure, w = unit weight of soil, δ = the angle made by E_a with the horizontal (see Fig. 107), α = angle made by the

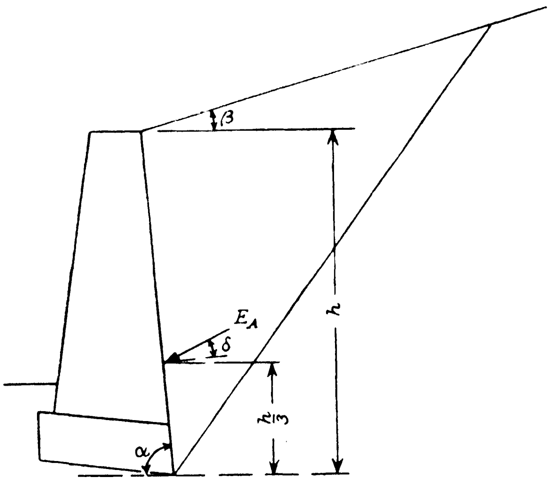


Fig. 107.—Coulomb or general wedge theory of retained soil.

back of the wall with the horizontal, β = angle of surcharge measured from the horizontal, and

$$K_a = \frac{\sin^2(\alpha + \phi)\cos\delta}{\sin\alpha\sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha - \delta)\sin(\alpha - \beta)}}\right]^2}$$

For the passive case, the corresponding formula is :—

$$E_p = \tfrac{1}{2}wh^2 \frac{K_p}{\sin\alpha \cos\delta}$$

$$\text{where } K_p = \frac{\sin^2(\alpha - \phi)\cos\delta}{\sin\alpha\sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi - \delta)\sin(\phi + \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}}\right]^2}$$

Useful tables are available for finding the value of $\dfrac{K_a}{\sin\alpha\cos\delta}$ and $\dfrac{K_p}{\sin\alpha\cos\delta}$, and are due to H. Krey[94].

In 1857, W. W. Rankine developed his widely used formulæ:

$$E_a = \tfrac{1}{2}wh^2\frac{1 - \sin\phi}{1 + \sin\phi} = \tfrac{1}{2}wh^2\tan^2\left(45^\circ - \frac{\phi}{2}\right)$$

$$E_p = \tfrac{1}{2}wh^2\frac{1 + \sin\phi}{1 - \sin\phi} = \tfrac{1}{2}wh^2\tan^2\left(45^\circ + \frac{\phi}{2}\right)$$

These formulæ assume a vertical back to the wall, and a horizontal backfill surface. All the foregoing formulæ apply only to clean, dry or submerged sand soils ; they assume that movement can take place in the backfill, and do not apply to cohesive soils.

Modern Earth-Pressure Theories.

In 1915, A. L. Bell[95] worked out extensions of Rankine's theory which included cohesion in soils, and his results have been applied most successfully to soft clay soils in which ϕ is zero. The equations used are then

$$p_a = wh - 2c$$

$$p_p = wh + 2c$$

where p_a and p_p are the active and passive pressure intensities respectively. It should be noted that both Rankine and Bell ignore any friction between the back of the wall and the face of the soil. It should also be noted that as shown on page 225, down to a depth $z = \dfrac{2c}{w}$ below the surface of the soil, the active pressure is negative, since in a cohesive soil, it is possible for tension cracks to remain open, so that no lateral pressure can be exerted over the distance $\dfrac{2c}{w}$ below the surface of the ground. Evidently, also, such a crack must have the effect of reducing the length of the effective sliding surface ABC (see Fig. 103, page 225).

Engesser's graphical construction (based on the assumption of failure along a plane surface) is useful for finding the thrust on the wall (see Fig. 108). Any plane surface of failure is assumed ; the forces acting on the wedge ab_3c are its weight W_3, the cohesive force c'' acting up the back of the wall, the soil reaction R_3 and the active thrust E_a. The directions of R_3 and E_a are known or assumed, and the direction and magnitude of the forces W_3 and c'' are known, so that a set of force polygons can be drawn for the different soil wedges cab_1, cab_2 and so on. The envelope of the reactions is drawn in, and the force E_a measured off from the right-hand diagram (Fig. 108). According to

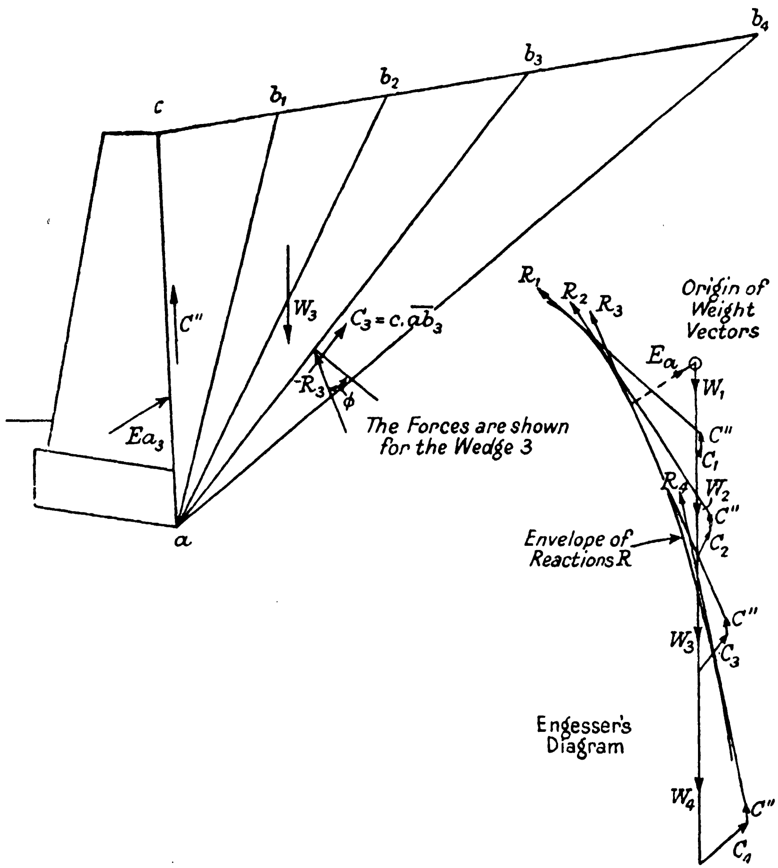


Fig. 108.—Engesser's graphical method for determination of active pressure behind a retaining wall.

Terzaghi, serious error may arise if the angle of wall friction is greater then $\frac{\phi}{3}$, and in this case Krey's construction (see Fig. 109) may be used.

In this case, it is assumed that the trace of the surface of failure is an arc of a circle ab, tangential to a straight line bc which is inclined at an angle $\left(45^\circ - \frac{\phi}{2}\right)$ to the horizontal. Failure is assumed to occur by displacement of the soil immediately in front of the toe of the wall, and the forces acting on the mass of soil *abde* are its own weight W, a

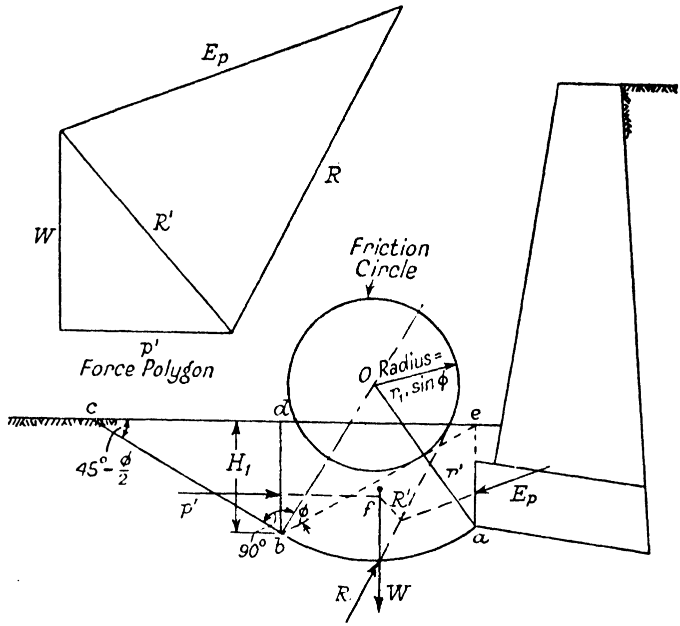


Fig. 109.—Determination of passive pressure behind retaining wall. (H. Krey.)

horizontal pressure P' due to the triangular wedge of soil *bcd*, the soil reaction R on the curved failure surface *ab*, and E_p, the thrust from the toe of the wall. The thrust P' can be obtained from the Rankine equation $P' = \frac{1}{2}wh^2.\tan^2\left(45^\circ + \frac{\phi}{2}\right)$, where h is the vertical height from the heel of the wall to the upper surface of the soil.

The weight W is easily calculated, and from the triangle of forces the resultant of forces P' and W is found ($= R'$ in left-hand diagram in Fig. 109). This resultant passes through f, the point of intersection of P' and W, and the soil mass *abde* is then in equilibrium under the action of the three forces, R, R' and E_p meeting in a point. The directions of R' and E_p are known or may be assumed, and R is assumed to be tangential to the so-called " friction-circle," the radius of which is $r_1 \sin\phi$, r_1 being drawn perpendicular to $cb = ob = oa$. R is drawn on the force polygon, whence E_p can be found. By choosing a number of arcs of failure *ab*, a minimum value for E_p can be found.

Fig. 110 illustrates the argument for the assumption that the soil reaction to a circular arc is tangential to the friction-circle. Consider the element of arc AB. If there were no friction around the arc AB,

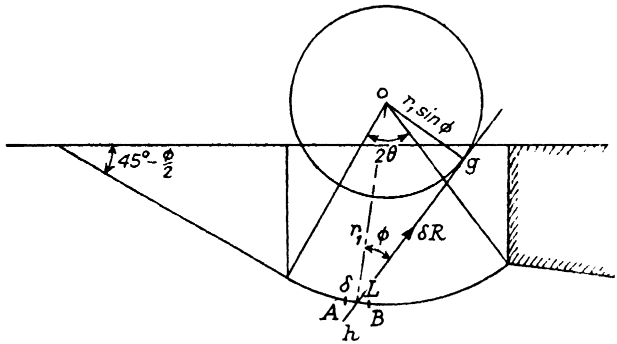


Fig. 110.—Relation between soil reaction at toe of retaining wall and the friction-circle.

the reaction δR would pass through point O, the centre of the friction-circle. When friction reaches its limiting value, the reaction δR departs from the normal by an angle ϕ, so that δR is a tangent to a circle of radius $r_1 \sin\phi$, since

$$\phi = \sin^{-1}\frac{og}{oh} = \frac{\sin^{-1} r_1 \sin\phi}{r_1}.$$

i.e., $og = r_1 \sin\phi$.

It is therefore assumed that the resultant R of all these elementary reactions will also be tangential to the friction circle, i.e., R will be a tangent to a circle whose radius depends on the central angle 2θ and the pressure distribution around the arc AB. If this pressure distribution is sinusoidal, then the radius of this circle is

$$r_1' = r_1 \sin\phi \left(\frac{1 - \frac{2\theta}{\pi}}{\cos\theta}\right)$$

It is considered by D. W. Taylor[92] that sinusoidal stress distribution is probable and that the ratio $\frac{r_1'}{r_1}$ is not likely to exceed 1·1. Thus the error involved in the original assumption is quite small and is on the safe side.

The above method applies strictly only to cohesionless soils, but has been modified by Terzaghi[96] to include soils possessing both cohesion and internal friction, although the method is somewhat involved.

Point of Application of Resultant Soil Thrust.

Most of the older theories of earth-pressure, excepting that of Bell, assumed that the pressure distribution of a soil behind a wall is hydrostatic, i.e., that it increases uniformly with increase in depth, or, in other words, that the resultant thrust acts at a point two-thirds of the distance down from the top of the wall. This is certainly not always the case in actual practice, and it is now more usual to assume that the resultant thrust acts at a point about half way down from the top of the wall. It should be noted, however, that the distribution of pressure has little effect on the magnitude of the total thrust, but only on its point of application.

Failure by Rotation of Soil Mass.

A possible method of failure is by rotation of the soil mass about a centre, as shown in Fig. 111. Fellenius assumed a surface of failure

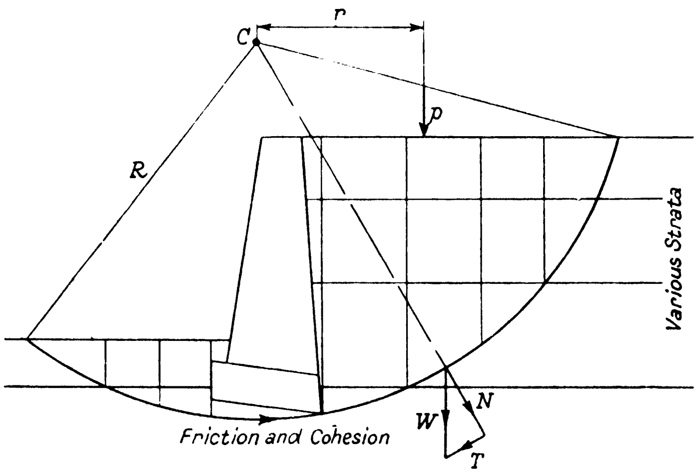


Fig. 111.—Slice method of consideration of stability of soil behind a retaining wall.

along a circular arc, and observation of many soil slips shows that this assumption is probably justified. In Fig. 111, the soil mass is considered as acting in horizontal slices, and the weight of each slice is calculated and resolved into radial components N and tangential components T. The total frictional force around the arc of slipping is then $\Sigma N \tan\phi$, and the total cohesive force is ΣLc, where L is the length of the arc over which cohesion acts. The forces tending to cause collapse are ΣT and isolated forces such as a surface point load P acting vertically downwards. Taking moments about the centre O of the circle

$R\Sigma N\tan\phi + R\Sigma L_c = F(R\Sigma T + Pr)$, where F is the factor of safety and r is the horizontal distance from the line of action of force P to centre O.

It should be noted that the above methods of consideration of possible failures of retained soil masses do not take into account failure of retaining walls themselves, caused by insufficient bearing capacity of the soil below the footings. These can be dealt with as already indicated previously on page 216 for undersoils for fills. It should also be realised the moisture conditions in the backfill are of paramount importance in soil behaviour, and that unless rubble filling immediately behind the face of the wall, with a suitably arranged sand filter and adequate and properly placed weep-holes through the wall are provided, moisture conditions may arise which may upset completely any calculations arrived at prior to the construction of the wall. Any retaining wall not properly drained is a natural barrier to and interference with the hydrostatic flow net operating in the impounded soil, and many failures of such walls are thought to be due to insufficient realisation of this fact.

This point has been brought out by experimental work done by Terzaghi[97], using a specially constructed bin 14 feet by 14 feet in plan, and 7 feet deep, closed in front by a moveable rigid metallic retaining wall 14 feet long and 7 feet high, and different kinds of backfill. Two cases were studied :—

(*a*) *Backfill of Dry Sand.* Uniform, angular dry sand of an average size of 0·54 mm. was used ; the value of the hydrostatic pressure ratio K (see Chapter 1) was 0·4 for loose and 0·6 to 0·7 for compacted sand. ϕ was 34°, whence $K = 0{\cdot}3$. Apparently the compaction of the fill caused deflections of the wall although probably these were very small in magnitude, and passive pressures were induced therefrom. The outward motion of the moving wall was accompanied by elastic recovery and considerable drop in soil pressure. After compaction of the soil, only a relatively small number of sand grains followed the moving wall, so that exceedingly low values of K, of the order of 0·1 to 0·2, were recorded.

The point of application of the pressure was above the lower third of the back of the wall at the start of motion of the wall, and close to it or even below it afterwards. The loose sands gave a point of application at the one-third point, i.e., they behaved similarly to impounded water.

During intermissions of motion of the wall, a decrease in the coefficient of wall friction (measured as the ratio of the vertical component of the pressure to its horizontal component), an increase in the hydrostatic pressure ratio K_a (the active hydrostatic pressure ratio), and in general a slight rise in the point of application of the lateral pressure.

(*b*) *Backfill of Saturated Sand.* The sand material described above was first submerged and then drained, and the horizontal pressure corresponding to these two conditions was determined. In the submerged condition, the moisture content was 25 per cent. of the dry weight of the sand, and in the drained condition, 2·5 per cent.

It was found that the presence of the sand behind the wall had no effect on the intensity of the water pressure, and that the values of ϕ and of the wall friction were also unaffected by the water. In the case of the drained but wet sand, it was found that drainage of the fill produced a slight decrease in the value of K_a and an increase in the value of the wall friction, but that during movement of the wall, K_a increased and the wall friction decreased. According to J. E. B. Jennings, of the National Building Research Institute, Pretoria, K is somewhere near $\frac{1}{2}$ for the consolidated condition if no movement takes place in the wall, falling off to 0·33 if movement takes place. Terzaghi[98] contends that the theory of stability of plain slopes may be applied with only slight modification to earth pressures against retaining walls. In the case of rather steep slopes of clay soil with the lower portion supported by a retaining wall, the curvature of the sliding surface is so appreciable that it affects the design of walls, but in the case of cohesionless backfills, with either a horizontal or a gently inclined upper surface, the curvature of the sliding surface may be neglected and a plane sliding surface assumed.

The intensity of earth pressure is thought to lie somewhere between the active and passive values, and according to Terzaghi, depends not on cohesion and internal friction, but solely on the elastic properties of the backfill, so that it cannot be computed in terms of the usual earth-pressure theories. The active and passive earth pressures themselves do, however, depend on cohesion and internal friction.

From a practical point of view, the effect of seasonal volume changes is of far greater importance than the factors just mentioned, especially in the case of clay soils. If the cohesion of these materials is alone considered, the earth-pressure exerted against low retaining walls should be almost zero, yet it is common knowledge that such walls do fail on occasion. The cause of such failures is undoubtedly the marked volume changes caused by alternate wetting and drying associated with such low permeability soils, i.e., the type of clay (see Chapter 3) is an all-important factor.

Earth Pressures in Trenches.

This problem, which is a special case of a retaining wall, has been studied by R. Glossop and H. Q. Golder[99], who point out that Coulomb's theory of earth pressure covers frictional and cohesive soils, and is correct for the conditions postulated for that theory, and that if the shear strength of a soil be known, it allows of the calculation of the active earth pressure on a retaining wall with a fair degree of accuracy. It gives only the total earth pressure, however, and does not give the distribution of such pressure. It is often assumed that this pressure is hydrostatic, i.e., that it increases uniformly with depth, and although this may be true in a few cases, it is hardly ever true when applied to the pressures on the timbering of a trench.

As shown previously in this chapter, the total earth pressure concerns the statics of a wedge of earth, but its distribution involves the deformation of the retained soil, and is statically indeterminate unless the amount of movement of the wall away from the soil is known or determinable. It is evident that when earth pressure is applied to a wall, the wall will yield in a manner which is dependent on the elastic properties of the wall and its supporting foundation soil. If movement of the wall about its toe occurs (see Fig. 112*a*), away from the retained

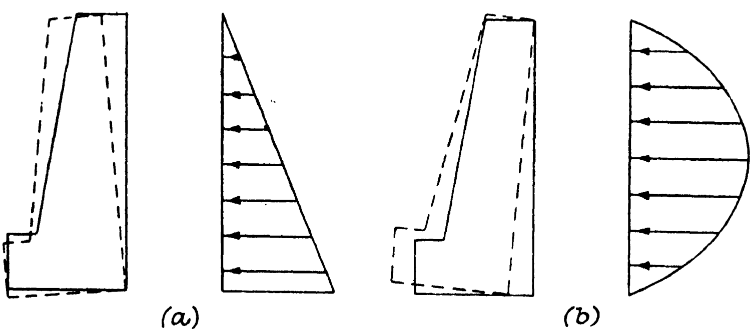


Fig. 112.—Movement of retaining wall. (*a*) about its toe. (*b*) about its top, with corresponding distributions of pressure.

soil, then the pressure distribution will probably be triangular, but if it moves about its top or towards the soil mass, then the pressure distribution will probably be approximately parabolic (see Fig. 112*b*). This difference of pressure distribution is due to the arching action of the soil from the top of the wall across to the earth which supports the slipping soil wedge, and arching of this kind transfers some of the pressure which would otherwise occur at the base of the wall to a point

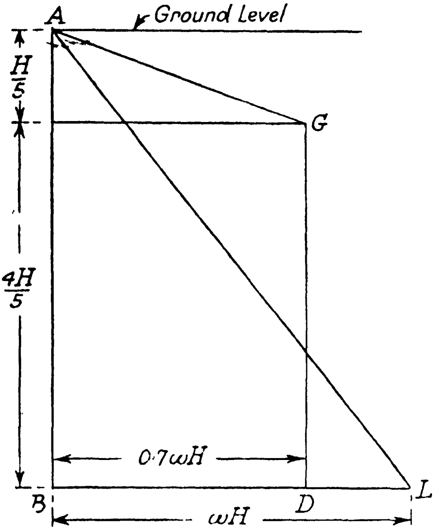


Fig. 113.—Illustrating Terzaghi's semi-empirical rule for distribution of earth pressure on a retaining wall.

higher up. In the case of trench timbering, field observations have shown that arrangement (*b*) in Fig. 112 is the more likely to apply.

Having found the total earth pressure by some method, such as the Engesser graphical construction, shown in Fig. 108, page 234, Terzaghi's semi-empirical rule (see Fig. 113) can be applied to obtain its distribution. The total pressure is taken as $\frac{1}{2}\omega H^2$, and is drawn as a triangular distribution ABL ; the pressure ωH at the base of the wall is reduced to 0·7 of its value ($= BD$ in Fig. 113), and a vertical DG is drawn up to a point G, $0{\cdot}2H$ below ground level. A line is then drawn from G to zero pressure at point A. The pressure diagram $ABDG$ is 26 per cent. greater than the calculated total pressure, but is the envelope of all likely parabolæ of pressure which may result from the usual method of excavation, and the timber or sheet piling can safely be designed to withstand this pressure.

Although not strictly analytical, the above method is a great advance on the usual guesswork methods employed in such problems, and if applied carefully, with due regard to all the factors involved, should result in a saving in construction costs, especially in the case of deep trenches.

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