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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE*

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WITH DISCUSSION BY MESSRS. CHARLES R. GOW, E. P. GOODRICH, VICTOR A. ENDERSBY, GEORGE PAASWELL, C. C. WILLIAMS, HERBERT CHATLEY, J. ALBERT HOLMES, JACOB FELD, W. S. HOUSEL, FRANK S. BAILEY, T. L. CONDRON, LAZARUS WHITE, R. D. N. SIMHAM, FRANK A. MARSTON, ARTHUR M. SHAW, D. P. KRYNINE, S. P. WING, M. L. ENGER, ALBERT JORGENSEN, AND CHARLES TERZAGHI.

SYNOPSIS

The paper reviews the present state of the science of foundations, its principal shortcomings, and the possibilities for its improvement.

The principal shortcomings were found to be: First, the practice of selecting the admissible soil pressure regardless of the area covered by the individual foundations and irrespective of the maximum differential settlement that the superstructure can stand without injury; second, the practice of computing the bearing capacity of the piles by the *Engineering News* formula regardless of the character of the soil; and, third, the practice of considering the bearing power of the individual piles as a sufficient guaranty that the bearing capacity of the entire foundation will be adequate.

In his discussion of these topics the writer tries to explain the reasons for the inconsistencies that are often experienced in attempts to interpret the results of loading tests for designing purposes. Concerning the pile-driving formulas the writer presents physical arguments why, for certain soils, no pile-driving formula can possibly furnish reliable information concerning the bearing capacity of the tested pile. For those soils for which the

* Presented at the meeting of the Structural Division, New York, N. Y., January 20, 1927.

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pile-driving formulas can be used to advantage, the *Engineering News* formula is found to furnish values which are by far too small, provided a drop-hammer is used and the penetration per blow amounts to less than ½ in.

Progress in the field of foundation engineering is handicapped essentially by the absence of reliable information concerning previous construction experience. Again, the lack of reliable information is due to inadequate description of the soils and to an interpretation of the observed facts which, in many cases, is inconsistent with the laws of physics and mechanics. The first step toward improving conditions should consist in establishing an adequate soil classification.

Thus far, related efforts have failed, because they represented attempts to classify the soils according to properties that have no bearing at all, or no well-defined bearing, on the behavior of the soil under load.

The writer presents a list of those soil properties that determine the character, the amount, and the speed of the settlements. He shows how the knowledge of these properties can be utilized for establishing a soil classification suited for its purpose and briefly discusses the difficulties associated with such an attempt.

The problems of foundation engineering are as inseparably associated with the problems of structural engineering, as is the shadow with the light, because one cannot conceive of a structure without a foundation. Therefore, they are among the most vital.

Effect of Type of Building on Admissible Settlement

Suppose it is desired to construct a reinforced concrete building on a lot selected and bought by the future owner. After inspecting the ground in the light of previous experience, it is decided to dispense with piles and put the building on individual spread footings. The mental operation leading to this decision is experienced almost daily, and is familiar to every engineer. This apparently simple decision may be analyzed by common sense. By spreading the weight of the building over the base of sufficiently large footings, the pressure can be reduced to a certain value, q, per unit of area. Under the influence of the pressure, q, the footings will settle through a distance, S. Perfectly uniform settlement will not hurt the building. However, because of the variations in the distribution of the live load, the pressure may vary between q and $0.8\,q$ (to name an arbitrary figure) which corresponds to a difference in settlement between S and $0.8\,S$. In addition, it may be that the compressibility of the ground varies to such an extent, that the settlement produced by a load, q, may range between S and $0.5\,S$ (which, again, is an arbitrarily selected figure, the actual value depending on circumstances). Hence, a difference, amounting to as much as $S - 0.5 \times 0.8\,S = 0.6\,S = H$, may be expected between the settlement of the individual footings. In the formula,

$$H = S - 0.5 \times 0.8\,S = 0.6\,S = n\,S \quad \text{.......... (1)}$$

the factor, n, represents the numerical effect of both the variations in live load and the variations in the compressibility of the soil. The value, H, will rule all further considerations. If the building is statically determinate a value of several inches for H would be permissible. On the other hand, if it is statically indeterminate, the allowable intensity of the secondary stresses must be selected first. Consider, for example, that an increase of 15% in certain maximum bending moments would be admissible. Then, the maximum value, H, for the difference in settlements is at once determined. It may range between 0.1 in. and 1.0 or 2.0 in., according to the character of the building, the distance between the columns, and the cross-section of the beams. Hence, the decision to admit an increase of 15% in the dangerous stresses, would require selecting the soil pressure, q, anywhere between limits as far apart as 1 and 10 or 20, according to the character of the building. Selecting the value, q, on the other hand, irrespective of the structural characteristics of the building, would mean either ultra-conservative wastefulness or danger, according to the circumstances. Since this is done in most of the cases dealt with in actual practice, one cannot help feeling the necessity of a thorough reform. Every foundation design ought to be started by roughly computing the maximum admissible value of H.

Relation Between Settlement, Size of Loaded Area, and Depth of Foundation

The second inconsistency that confronts the foundation engineer daily, concerns the relation between the settlement, S, and the diameter of the loaded area. In selecting the admissible unit soil pressure, should the width and the length of the loaded area be taken into consideration? Consider, as an example, that some one makes a bending test on a 10-in. by 10-ft. I-beam, freely supported at both ends, and finds that the beam breaks under the influence of a concentrated load of P-tons. He publishes the results of his test, and as a consequence, in future practice, the designers would consider a load of $\frac{1}{4}P$ as a safe load for the beam, irrespective of the span and mode of support. In the light of present knowledge, such procedure would be considered simply absurd. However, in the field of foundation engineering it corresponds precisely to what is actually done. The safety of the building depends on H $(= 0.6\,S)$ not exceeding a definite value. The value of S depends as essentially on the diameter of the loaded area and on the depth of the foundation as the load causing the rupture of the I-beam depends on the span. Nevertheless, all tables of admissible soil pressures, given in textbooks and building codes, contain the values without any indication as to what areas and to what depths of foundation they apply. They can well be compared with tables containing the loads under which I-beams of different cross-sections broke down, without mentioning the distance between supports.

The rather indifferent attitude of practising foundation engineers toward the important influence of these two items—the dimensions of the loaded area and the depth of foundation—is essentially due to the apparent inconsistencies of related experiences which seem to contradict each other, to such an extent that an attempt to digest them appears to be hopeless. However, by analyzing them and correlating them with present theoretical knowledge, one finds that they already represent a fund of most useful information, and that the apparent contradictions do not exist. The following facts may serve to illustrate these statements.

In 1916, the Foundation Committee of the Austrian Society of Engineers and Architects, in Vienna, made a series of loading tests for the purpose of investigating the influence of the shape and size of the loaded area on the amount of settlement in a typical cohesive Vienna soil (a variety of loess).* The tests were performed on the bottom of a trench, 10 ft. deep and 5.75 ft. wide, protected by a roof. The loaded soil consisted, according to a statement issued by the investigators, of 27% of sand with grains larger than 0.2 mm., 31% of sand with grains less than 0.2 mm., and 42% of very fine-grained constituents. The water content was said to range between 11.2 and 13.2 per cent. When moulded into cubes and dried, the soil developed a cube strength of about 35 tons per sq. ft. The tests included load settlement, time settlement, and rebound observations on square and on round plates with areas of 0.053, 0.110, 0.672, 2.700, and 6.100 sq. ft. Fig. 1 shows some of the results obtained. They include the settlements of plates with different diameters under loads of 2 and of 4 tons per sq. ft., and the rebound of the soil

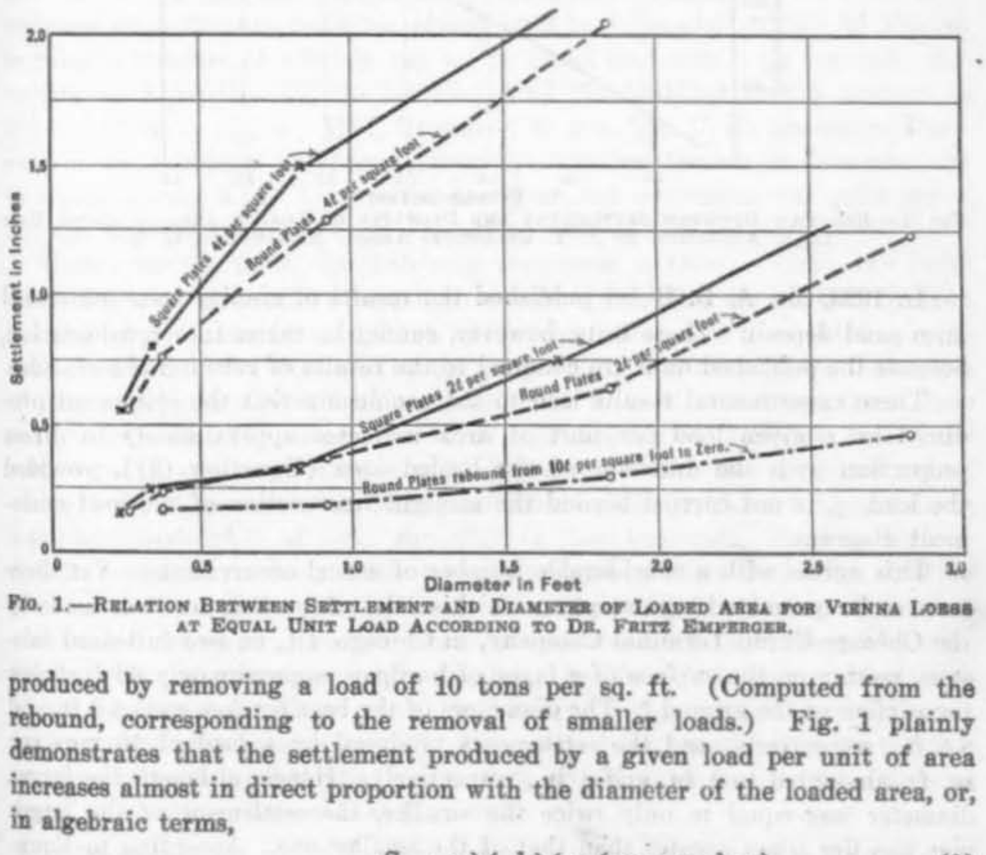


FIG. 1.—RELATION BETWEEN SETTLEMENT AND DIAMETER OF LOADED AREA FOR VIENNA LOESS AT EQUAL UNIT LOAD ACCORDING TO DR. FRITZ EMPERGER.

produced by removing a load of 10 tons per sq. ft. (Computed from the rebound, corresponding to the removal of smaller loads.) Fig. 1 plainly demonstrates that the settlement produced by a given load per unit of area increases almost in direct proportion with the diameter of the loaded area, or, in algebraic terms,

$$S = c \times d \times q \text{ (more or less)} \qquad (2)$$

* According to a manuscript, "Der Wiener Löss und seine zulässige Belastung," by Dr. Fritz Emperger, received in May, 1926. According to Dr. Emperger's letter of March 16, 1926, an extract of this paper has been published in *Die Bautechnik*, Berlin, *Le Genie Civil*, Paris, and *Gewappend Beton*, Amsterdam.

wherein, c is a constant of the soil. The shape of the upper set of curves is obviously due to the fact that the load-settlement curve for each one of the individual plates is, in itself, more or less parabolic.

A few years ago, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., carried on some research work for the U. S. Bureau of Public Roads, on artificial mixtures of sand and clay for the purpose of investigating the relation between the area of a bearing block and the settlement produced by a given unit load.* The area of the bearing blocks ranged from a few square inches to 9 sq. ft. The results of the tests made on plastic mixtures were of the type shown in Fig. 2. They indicate, as do the Austrian tests, that for cohesive soils the penetration produced by a given unit load increases in direct proportion with the diameter of the loaded area.

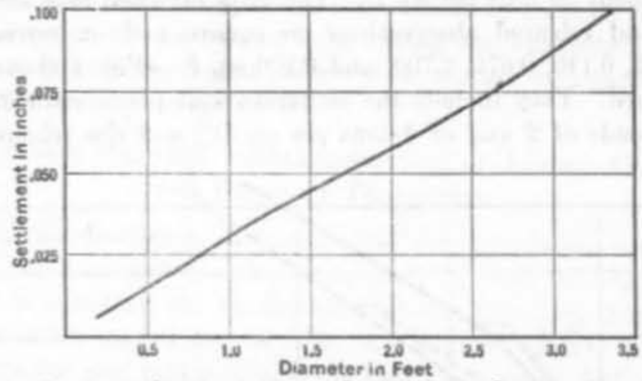


FIG. 2.—RELATION BETWEEN SETTLEMENT AND DIAMETER OF LOADED AREA AT EQUAL UNIT LOAD, ACCORDING TO A. T. GOLDBECK, ASSOC. M. AM. SOC. C. E.

In 1923, Mr. A. B. Bijls† published the results of similar tests performed on a sand deposit. These tests, however, cannot be taken into consideration, because the published data are confined to the results of rebound observations.

These experimental results lead to the conclusion that the settlement produced by a given load per unit of area increases approximately in direct proportion with the diameter of the loaded area (Equation (2)), provided the load, q, is not carried beyond the straight line section of the load-settlement diagram.

This agrees with a considerable number of actual observations. Yet there seem to be quite striking exceptions to this rule. A loading test was made by the Chicago Union Terminal Company, in Chicago, Ill., on two full-sized caissons, resting on the surface of a layer of hardpan, approximately 60 ft. below the surface of the ground.‡ The diameters of the bearing area were 4.3 ft. and 8.5 ft., respectively, and the settlements produced by a load of 10 tons per sq. ft. amounted to $\frac{1}{8}$ in. and $\frac{5}{8}$ in., respectively. Hence, although the larger diameter was equal to only twice the smaller, the settlement of the larger pier was five times greater than that of the smaller one. According to Equa-

* "Researches on the Structural Design of Highways by the United States Bureau of Public Roads," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

† *Le Génie Civil*, May 26, 1923, p. 490. Abstract of a report by M. Wolterbeck to the Ministère des Travaux Publics des Pays Bas, 1922.

‡ *Journal*, Western Soc. of Engrs., February, 1924.

tion (2), it should not have been more than twice as large. This fact seems to indicate that the formula represents an optimistic statement.

On the other hand, experience shows that, under certain circumstances, the settlement of the larger area may be very much smaller than the value obtained by the formula. Numerous loading tests made on the plane surface of a natural bed of well-compacted sand, have shown that the settlement of a single bearing plate, with an area of 1 sq. ft., loaded with 3 tons, is never less than about ⅛ in. Hence, according to Equation (2), a foundation 20 ft. square, resting on a firm sand deposit, should settle 2½ in. under a load of 3 tons per sq. ft. Every engineer who has had experience with foundations on firm sand can testify that the settlement will be very much less, certainly not exceeding ½ in. In 1922, the Society's Special Committee on Bearing Value of Soils for Foundations, etc., published the results of loading tests performed on a building lot in San Francisco, Calif.* A stratum of soft, clean, or sticky sand, with occasional layers of sand clay and yellow clay was encountered. It was tested at eight different points. Under a load of 4 800 lb. per sq. ft., the settlement of the bearing plate ranged between 0.04 and 0.17 in., averaging 0.10 in. For the past few years the lot has been occupied by a 22-story building, placed on a raft foundation, 218 by 152 ft., exerting a pressure of 4 800 lb. per sq. ft. (dead load only)† on the soil. According to Equation (2), the settlement of the building should amount to 152×0.10 in. $= 15.2$ in. H. J. Brunnier, M. Am. Soc. C. E., Designing Engineer of the building, has found‡ that the total settlement to date amounts to approximately 2 in. During the first year, the settlement was quite rapid, but the rate has been less each succeeding year.

Reviewing the facts, the following statement is clear: First, two independent sets of experiments have been quoted concerning the relation between the size of the loaded area and the settlement. Both sets have led to practically the same result, expressed by the simple Equation (2). Then, the results of these experiments have been compared with those of two observations made on a large scale. In one of these cases (Chicago Union Terminal tests), the settlement of the larger area was 2.5 times more than the computed value; while in the other case (San Francisco office building), the actual settlement was about one-eighth of what should have been expected. Suppose a foundation company sincerely tries to obtain some reliable basis for drawing conclusions from the results of its loading tests. The engineers of the firm make their observations year by year, accumulate the data in their files, and, finally, when they try to correlate the data, they face grotesque contradictions of the type quoted. They cannot be blamed if they give it up in disgust, loading tests, observations, and all. However, if they would examine the data in the light of applied mechanics, they would first realize the following facts, which the writer published§ in 1925:

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 529, Plate VII.

† "Continuous Mat Foundation for 22-Story Building," *Engineering News-Record*, July, 1922, pp. 73 *et seq.*

‡ By letter of December 4, 1925, to Maj. W. A. Danielson.

§ "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

(*a*) The relation between the diameter of the loaded area and the settlement produced by a given unit load depends essentially on the cohesion (actual shearing strength) of the soil. For soils with great cohesion the settlement produced by a given unit load increases in direct proportion with the diameter of the loaded area. On the other hand, for perfectly cohesionless soils, the size of the area has very little effect.

(*b*) With increasing depth of foundation, the settlement produced by a given unit load decreases. However, the ratio between the settlement, S_0, for a foundation depth of 0, and the corresponding settlement, S, for a depth, t, does not depend on the value of t alone, but on the ratio, $\frac{t}{d}$, between the depth of foundation and the diameter of the loaded area. Thus, if a foundation 5 ft. deep was found to reduce the settlement of a footing, 5 ft. square, by 50% of S_0, the effect of a footing 10 ft. square, and 5 ft. deep, in reducing settlement, will be very much less. In order that the potential settlement, S_0, of a footing 10 ft. square, and 0 ft. deep, may be reduced 50%, the depth would have to be increased to 10 ft., thus making the ratio, $\frac{t}{d}$, for both cases equal.

(*c*) The effect of the ratio, $\frac{t}{d}$, on the settlement is the less, the greater the cohesion. For perfectly cohesionless materials, a ratio of $\frac{t}{d} = 1$ (depth of foundation = diameter of loaded area) almost triples the bearing capacity and reduces the settlements to one-third of what they would be if the footing rested on the surface of the ground.

Knowing nothing more about soil mechanics than these three simple rules, the foundation engineer would at once discover that there are no contradictions in the data hereinbefore quoted. The experiments of the Austrian Committee and of Mr. Goldbeck were made on very cohesive soils. Hence, in strict agreement with theory, they showed that the settlement increases in direct proportion with the diameter of the loaded area. The Chicago tests were made on a cohesive soil, but the ratio between the depth of foundation and the diameter of the loaded area was twice as great for the larger area as for the smaller one. That fully accounts for the difference between the computed and the actual value of the settlement of the larger area. On the other hand, the soil of San Francisco belongs to the class of very slightly cohesive soils and, according to Rule (*a*), which is supported by experience, the size of the area should have comparatively little effect on the settlement.

Thus, if it combines actual observation with proper soil investigation and with a thorough consideration of the mechanical aspects of its problems, the same firm that gave up the attempt of systematic study in disgust, could gradually build up an experience covering the whole range of its activities and acquire a fairly reliable basis for the interpretation of the results of loading tests. However, under the present system of interpretation, loading tests are practically good for nothing.

Distribution of Soil Reactions Over Rigid, Loaded Slabs

The preceding discussion deals exclusively with the potential settlement of structures without questioning the stresses that may develop within the foundation. As a matter of fact, when referred to individual spread footings, these questions are of minor importance. However, the situation changes when it becomes necessary to connect the footings with each other, replacing the group of footings by a continuous mat. In order to keep the costs of such a mat within reasonable limits, the stresses must be computed as closely as possible. It is first necessary, however, to know the forces acting on the mat, and this is where difficulties arise. Suppose the mat is rectangular, and the dead load of the structure is distributed in such a manner that the resultant force passes through the center of gravity of the mat. Under such conditions, it is generally assumed, that the soil pressures are uniformly distributed over the entire area. However, according to the measurements carried out by M. L. Enger, M. Am. Soc. C. E., the distribution of the soil reactions over the base of a rigid slab is by no means uniform.* The pressures are equal to zero at the edge of the slab and greatest at the center, the pressure curve having a parabolic shape. In order to show the effect of the difference between uniform and parabolic stress distribution, the diagram, Fig. 3, has been plotted, showing the bending moments in a mat, loaded by four walls,

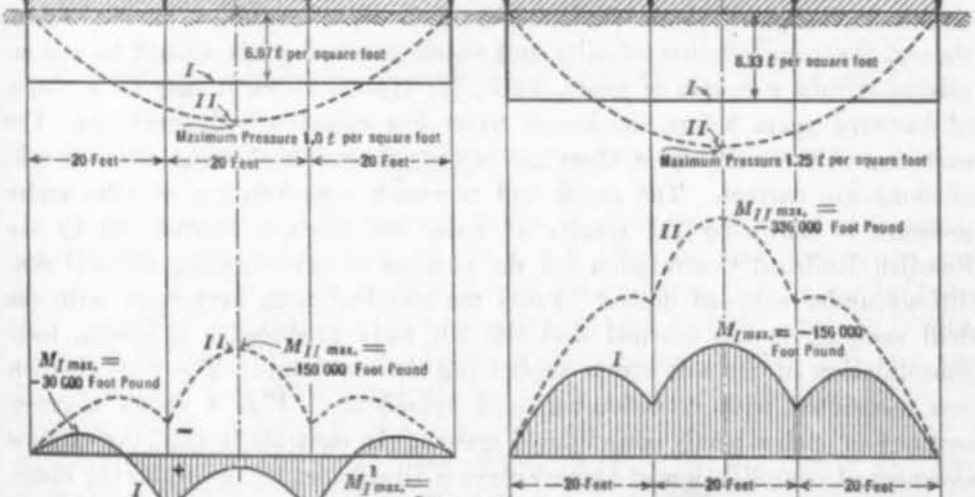


Fig. 3.—Bending Moments in Rectangular, Perfectly Rigid Slab for (I) Uniform and (II) Parabolic Distribution of the Soil Reaction.

and supported by a homogeneous soil. For a parabolic stress distribution the maximum bending moments will be about 100% greater than they are for a uniform reaction. Thus far, all attempts to deal theoretically with the problem of stress distribution have failed, and there is little hope for success within the near future. The data obtained by Professor Enger are confined

* "The Distribution of Pressure by Granular Material," *Engineering*, Vol. 101, 1916, p. 170.

strictly to perfectly cohesionless materials; hence, the information they furnish is not yet conclusive. Considering the importance of the possible error involved in assuming uniform stress distribution, the need of a more exhaustive experimental investigation of this phase of the foundation problem becomes obvious.

Effect of Degree of Permeability and of Time on Settlements

Another interesting aspect of the foundation problem concerns the changes that will take place in the soil under pressure. Since every soil, without exception, is compressible, the pressure tends to produce a decrease in volume. If the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface. On the other hand, if the voids are completely filled with water, which is usually the case with very fine-grained and very compressible soils (silts, mud, clay), a decrease in volume obviously involves a considerable decrease in the water content, and the compression cannot possibly proceed with a greater speed than the corresponding speed of squeezing out the excess water. The less permeable the soil, the more slowly the water escapes, and, as a consequence, the more slowly the volume of the compressed soil decreases. Hence, the settlement of the foundation will not occur at once. There will be a lag, depending on the degree of permeability of the soil.

The physical side of this gradual consolidation process has been thoroughly investigated, both theoretically and experimentally. According to theory, the consolidation of silts and coarse-grained muds should be accomplished within a couple of years, while, for typical clays, it may be a couple of hundred years before the excess water has completely drained out. For each one of these examples, there are records accessible showing that the conclusions are correct. The rapid and thorough consolidation of silts under pressure is shown by the results of many test borings carried out by the Swedish Railroad Commission for the purpose of investigating the soil conditions under railroad dams.* From the physical tests performed with the drill samples, it was learned that the fills have produced a thorough, local consolidation of the soft strata supporting the surcharge. The consolidation was associated with considerable local subsidence. Fig. 4 shows a cross-section† of such a partly consolidated system. In contrast to this, the extreme slowness of consolidation of typical clays is illustrated by the following observation. In 1915, a building of the Massachusetts Institute of Technology was erected on a 30-ft. layer of sand, silt, and fill, resting on a soft clay deposit. The pressure exerted by the building on the clay amounts to approximately 1 500 lb. per sq. ft. In 1926, a test boring was made next to the heaviest section of the building. Physical examination of the drill samples disclosed the fact that the consolidation of the clay deposit had hardly started. A similar observation was published in 1923 by Thaddeus Merriman, M. Am. Soc. C. E., concerning the behavior of a mass of puddled clay forming part

* Statens Järnvägars Geotekniska Kommission, 1914–22, Stockholm, May, 1922.

† Copied from Statens Järnvägars Geotekniska Kommission, 1914-22.

of an embankment on the Ashokan Reservoir, Catskill Water Supply.* The surface of the puddled clay deposit, confined between the core-wall and the up-stream slope of the core ditch, carries the weight of an embankment 85 ft. high. Nevertheless, nine years after the construction of the embankment, a test boring disclosed the fact that the clay core was practically in its original condition.

The study of these and similar facts has led to the recognition of two essentially different types of settlements:

(*a*) Those due mostly to lateral flow, with very little, or no, consolidation; and,

(*b*) Those due to consolidation and lateral flow combined.

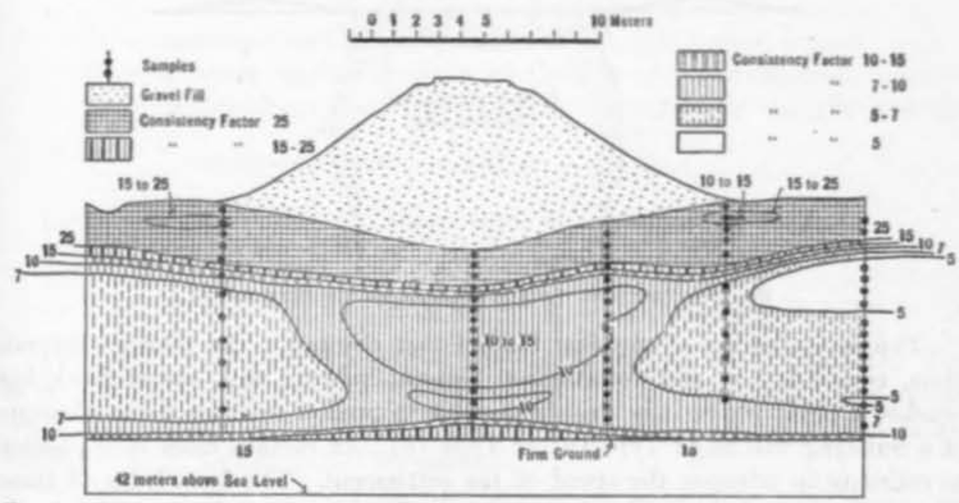


FIG. 4.—CONSISTENCY FACTORS OF SOIL DIRECTLY UNDER AND ON THE SIDE OF THE BANK AT SMEDSERÖD, SWEDEN.

In order to explain the characteristics of these two types of settlements, assume that two vertical reference lines, *a b* (Fig. 5), had been established in the ground, prior to the construction of the building. Under the influence of the weight of the building the soil tends to spread laterally, and the vertical lines, *a b*, become Curves *a c b*. The area included between the lines, *a b*, and the curves, *a b c*, is obviously equal to the area, $a\ a\ a_1\ a_1$, through which the building settles on account of lateral soil displacement. If, on account of a low permeability of the ground, the consolidation of the soil due to increased pressures, proceeds very slowly, the area, $a\ a\ a_1\ a_1$, represents practically the entire vertical displacement that the building will undergo, at least for one generation. The speed of the settlement will be governed exclusively by the laws of viscous flow in plastic materials, and if the flow is successfully stopped by driving deep sheet-piling, or by similar measures, the settlement of the building will be practically stopped. Those are the characteristics of Type (*a*) settlements.

On the other hand, if the material is permeable enough to release its excess water within a couple of years, the settlement of the building will consist of

* Discussion of the paper entitled "Design of Earth Dams," by Joel D. Justin, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 109.

two different parts, namely, the settlement through a space, $a\ a\ a_1\ a_1$, due to lateral bulging; and a settlement through an additional space, $a_1\ a_1\ a_2\ a_2$, due to compression of the soil beneath the loaded area. In this case the speed of the settlement will be governed by two essentially different laws: (1) Viscous flow in plastic materials; and (2) hydrodynamic stress compensation; which the writer formulated several years ago.* By artificially preventing the lateral flow of the material, one could somewhat reduce the settlements of the building, but one could not possibly prevent them. Those are the characteristics of Type (*b*) settlements.

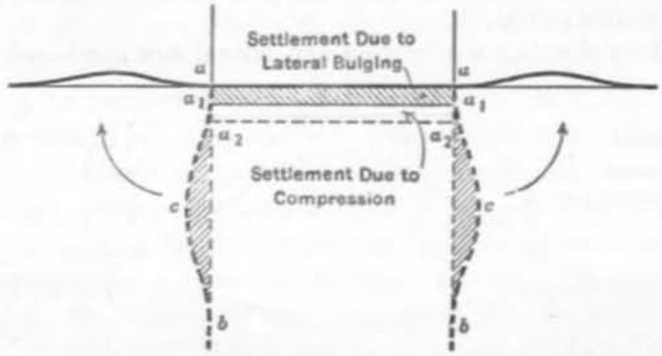


FIG. 5.—THE TWO PRINCIPAL SOURCES OF SETTLEMENT.

The investigation of physical factors that determine the relation between time, consolidation, and deformation (lateral bulging and lateral flow), has reached a point where it is already possible to predict whether the settlements of a building will be of Type (*a*) or Type (*b*). In certain cases it is possible to estimate in advance the speed of the settlement. The knowledge of these factors is particularly useful in all cases where the engineer faces the problem of reducing or stopping the settlement of an existing building.

BEARING CAPACITY OF INDIVIDUAL PILES

At the beginning of this paper it was assumed, that the engineer trusted with the design of a building, decided to erect it on spread footings. Let it now be assumed that his decision was premature. Suppose that test borings made after the first inspection of the building lot revealed the fact that the solid top layer of soil rested on a deposit of soft silt of such a thickness that there was no chance to carry the foundations down to solid ground. Then the engineer decided to put the foundations on piles. In order to find out how many piles he needed, he was obliged to drive a couple of test piles and determine the amount of load one individual pile could stand (unless he had previously tested some piles driven into similar deposits). To follow the traditional procedure, one must observe the average penetration of the pile under the last ten blows; compute the bearing capacity of the pile by the *Engineering News* formula; and divide the total weight of the building by the bearing capacity of the individual pile. This practice rests on two assumptions: (1) That the bearing capacity of a pile can be computed from

* "Die Theorie der Hydrodynamischen Spannungserscheinungen und ihr erdbautechnisches Anwendungsgebiet." *Proceedings*, International Congress for Applied Mechanics, Delft, Holland, 1924.

the effect of the blow; and (2) that the bearing capacity of the complete foundation is equal to the sum of the bearing capacities of the individual piles. To examine Assumption (1), let,

R = weight of the hammer.
G = weight of the pile.
L = length of the pile.
F = area of the cross-section of the pile.
E = modulus of elasticity of the pile material.
h = distance the hammer drops.
s = penetration produced by one blow.
m = coefficient of elasticity of the impact; $m = 0$ for perfectly non-elastic impact; and $m = 1$ for perfectly elastic impact.
Q_d = resistance against penetration of the pile, under impact.
Q = ultimate bearing capacity of the pile under static load.
C = empirical constant depending on the nature of the pile and the resistance against penetration $= \frac{Q_d L}{2 F E}$

The theory of semi-elastic impact leads to the following equation:

$$Q_d = \frac{F}{L} E \left[-s \pm \sqrt{s^2 + \frac{2 R h}{E} \frac{R + m^2 G}{R + G} \frac{L}{F}} \right] \qquad (3)$$

The value, m, is usually assumed equal to 0.5 (semi-elastic impact). For $m = 0$, the equation becomes Redtenbacher's formula, which is quite extensively used in Europe.* On the other hand, if perfect elastic impact is assumed, $m = 1$, Equation (3) becomes,

$$R h = Q_d s + \frac{1}{2} \frac{Q_d^2}{F} \frac{L}{E} = Q_d \left(s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E} \right) \qquad (4)$$

or,

$$Q_d = \frac{R h}{s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E}} \qquad (5)$$

If the fact that the term, $\frac{1}{2} \frac{Q_d}{F} \frac{L}{E}$, depending on both the nature of the pile and the resistance against penetration, is disregarded, and if this variable term is replaced by an empirical constant, C, independent of all these factors,

$$Q_d = \frac{R h}{s + C} \qquad (6)$$

which is none other than the well-known *Engineering News* formula.

From a mechanical point of view, the assumptions on which these formulas are based, are sound, and there is not much doubt about their giving a fairly accurate conception of the resistance one has to overcome while driving the pile by a succession of impacts. Therefore, if experience shows that in certain cases the values furnished by the pile-driving formulas have practically

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf. Leipzig, 1906.

nothing in common with those determined by loading tests, being either far too small or far too large, the cause cannot be a defect in the formulas, but must be due to the fact that in these particular instances the forces, Q_d, resisting the penetration of the pile under impact, are fundamentally different from the forces, Q, which resist the penetration of the pile under static load.

This is precisely the situation a theoretical study of the pile-driving phenomenon has disclosed. It is known that the bearing capacity of piles depends on two different factors, namely, the frictional resistance acting along the sides of the piles, and the point resistance, or the resistance of the soil against being compressed and displaced by the pile. If these two resistances were dependent only on the character of the ground and on nothing else there could be no question about the resistance, Q_d, against driving the pile, and the bearing capacity, Q, of the pile, being identical. However, it is easy to prove that either one of them may be very different according to whether the pile is slowly forced down or driven by impact.

If a friction test is made on a layer of sand, by loading it and then measuring its resistance against shear, it is found that the shearing resistance of the loaded layer, immediately after the application of the load, is practically the same as it is three days later. If, however, precisely the same test is performed with a layer of clay immersed in water, it is found that immediately after application of the load the frictional resistance is very small, so small indeed that one has the impression that the material is lubricated. The full frictional resistance does not develop for a couple of days. If a laterally confined mass of sand is compressed, the speed with which the compression is performed has very little influence on the amount of work required to compress the material. On the other hand, the amount of work required for rapidly reducing the volume of 1 cu. ft. of laterally confined clay by 2 cu. in. may be 100 times as great as the amount of work required for producing the same volume change slowly. The physical causes of these phenomena are clearly understood. The writer has called them the hydrodynamic stress phenomena. They inevitably develop as a result of rapid application of loads or pressures on water-soaked materials with a low degree of permeability.* The theory has been repeatedly checked by experiment.

Applied to the mechanics of pile-driving, knowledge of the hydrodynamic stress phenomena has led to classifying soils in two main types. In certain materials (particularly in sand, gravel, and permeable artificial fills), the resistances acting while the pile is being driven, are practically identical with those acting on the pile under static load. Under such conditions the pile-driving formulas can be expected to furnish results of sufficient accuracy.

In other materials (very fine-grained silts, soft clays, etc.), the friction acting on the pile during the driving (hydrodynamic pile friction) is very much less than that which develops after a couple of days' rest (static pile friction), while the resistance of the point of the pile under impact (dynamic point resistance) is very much greater than its resistance under static load (static point resistance). Due to these facts the total resistance against penetration of the pile into such materials is:

* "Erdbaumechanik," by Charles Terzaghi, Wien, 1925.

(1) Dynamic resistance (resistance, Q_d, against penetration under impact), which is the sum of a very small frictional resistance (dynamic pile friction) and a very considerable point resistance (dynamic point resistance).

(2) Static resistance (ultimate bearing capacity, Q, under static load), which is the sum of a full frictional resistance (static pile friction) and a very small point resistance (static point resistance).

Since the pile-driving formulas furnish the value, Q_d, there is no assurance whatsoever that for this class of materials the value, Q, may be of the same order of magnitude. The value, Q_d, may, by chance, be equal to Q, the deficiency in static friction being compensated by an excess in point resistance; but this condition is by no means necessary. It could as well be very much greater or very much less, depending on the material. The following analogy demonstrates the error committed when applying the pile-driving formulas to resistance against the penetration of piles in this second class of materials. Suppose that a body slides on a rail under water. In order to produce a slow forward movement of this body, the only resistance there is to overcome is the static friction between the body and the rail, the resistance of the water being negligible. On the other hand, in order to keep the body sliding rapidly along the rail, the only resistance that counts is the resistance of the water, which, in this case, may be very considerable, while the frictional resistance along the rail will practically be eliminated because of a film of water trapped between the rail and the sliding body. Application of a pile-driving formula to materials of the second class is no more logical than an attempt to identify the static resistance of the sliding body against being set in motion with its dynamic resistance acting while moving rapidly through a viscous medium.

The best way to distinguish whether a material belongs to the first or to the second class is that of comparing the penetration per blow immediately before and after a period of rest of at least 24 hours. If these two penetrations are identical, one can be quite sure that the material belongs in the first class, and that the pile-driving formulas can be expected to furnish fairly reliable results. However, for this case, experience seems to show that the formulas based on the theory of impact (Equation (3)), furnish far better results than the *Engineering News* formula.

In 1925, the writer investigated the bearing capacity of an artificial fill, consisting of residual soil (stones, sand, and earth mixed), at Pasha Liman, on the Asiatic shore of the Bosphorus. When an attempt to make test borings with a normal outfit failed, several test piles were driven, one of which was loaded to the limit of its bearing capacity. The pile-driver was of the drop-hammer type with a weight of 0.575 metric tons. Fig. 6 (*a*) shows the results of the loading test; Fig. 6(*b*) shows the resistance against penetration under impact computed by the *Engineering News* formula (dotted lines) (Equation (6)) and by the theory of semi-elastic impact (thin full drawn line) (Equation (3)). The two sets of values are not very different, because the penetrations were rather important (ranging between 1.0 and 1.4 in.), although the values obtained by the impact theory are nearer the ultimate bearing capacity determined by the loading tests.

However, with decreasing values of the penetration produced by the last blows, the error involved in the *Engineering News* formula rapidly increases. A rather striking example can be quoted from pile-driving in Germany. As a result of many years of experience, the foundation engineers of Berlin, Germany, have developed the following empirical rule for the sandy soil of that city: For a pile that penetrates less than 0.4 in. under the impact of a 1-ton hammer, dropping from an elevation of 3.3 ft., the safe bearing capacity ranges between 20 and 25 tons. If the *Engineering News* formula is applied to this case, the ultimate bearing capacity is found to be 28.2 tons, corresponding to an allowable load of $\frac{1}{6} \times 28.2$ tons $= 4.7$ tons! For the ultimate bearing capacity of the same pile, Redtenbacher's formula* furnishes a value ranging between 52.8 to 63.6 tons, which is very close to the actual ultimate bearing capacity of these piles. On the other hand, by driving a 90-ft. wooden pile to refusal with a 5 000-lb. steam hammer (2.8-ft. drop), the safe bearing value of the pile, according to the *Engineering News* formula, should be equal to 127 tons. This is even more than the ultimate bearing capacity

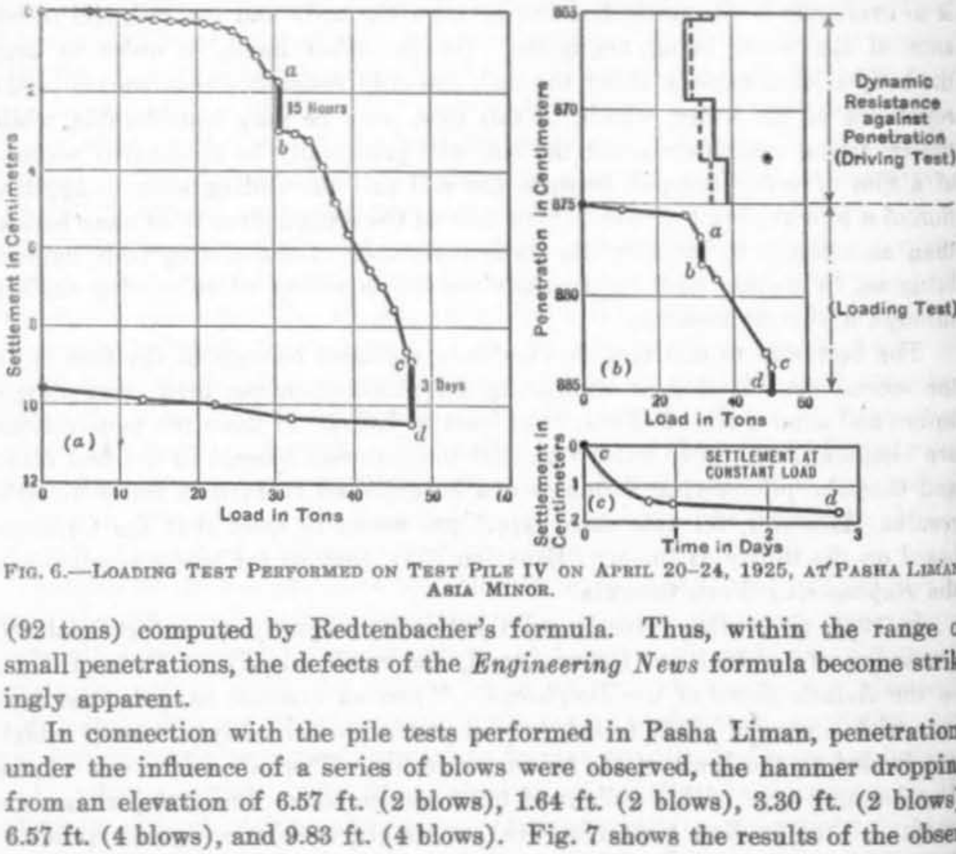


FIG. 6.—LOADING TEST PERFORMED ON TEST PILE IV ON APRIL 20–24, 1925, AT PASHA LIMAN, ASIA MINOR.

(92 tons) computed by Redtenbacher's formula. Thus, within the range of small penetrations, the defects of the *Engineering News* formula become strikingly apparent.

In connection with the pile tests performed in Pasha Liman, penetrations under the influence of a series of blows were observed, the hammer dropping from an elevation of 6.57 ft. (2 blows), 1.64 ft. (2 blows), 3.30 ft. (2 blows), 6.57 ft. (4 blows), and 9.83 ft. (4 blows). Fig. 7 shows the results of the observations. The points that correspond to a drop of the hammer of 6.57 ft. (200 cm.), 1.64 ft. (50 cm.), and 3.30 ft. (100 cm.), are located on a straight line. According to the theory of pile-driving by impact, this fact indicates that the resistance of the ground remained unchanged during the driving period.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf. Leipzig, 1906.

Hence, for this period, a correct pile-driving formula ought to furnish identical values, irrespective of the height from which the hammer was dropped. Table 1 shows the values obtained by the *Engineering News* formula and the theory of semi-elastic impact, respectively. It demonstrates that for penetrations of less than 1 in., the *Engineering News* formula furnishes values that are by far too small, the error rapidly increasing with decreasing depth of penetration.

The obvious reason for this deficiency of the *Engineering News* formula is that the variable item, $\frac{1}{2}\frac{Q_a}{F}\frac{L}{E}$, (Equation (5)), forthcoming as a result of the theory of impact, has been replaced by a constant, C.

All these facts and data refer to typical materials of the first class, that is, to materials for which the pile-driving formula can be expected to furnish fairly reliable values. This conclusion was reached by observing, among other things, that the penetration produced by impact of a given intensity was practically the same before and after a period of rest.

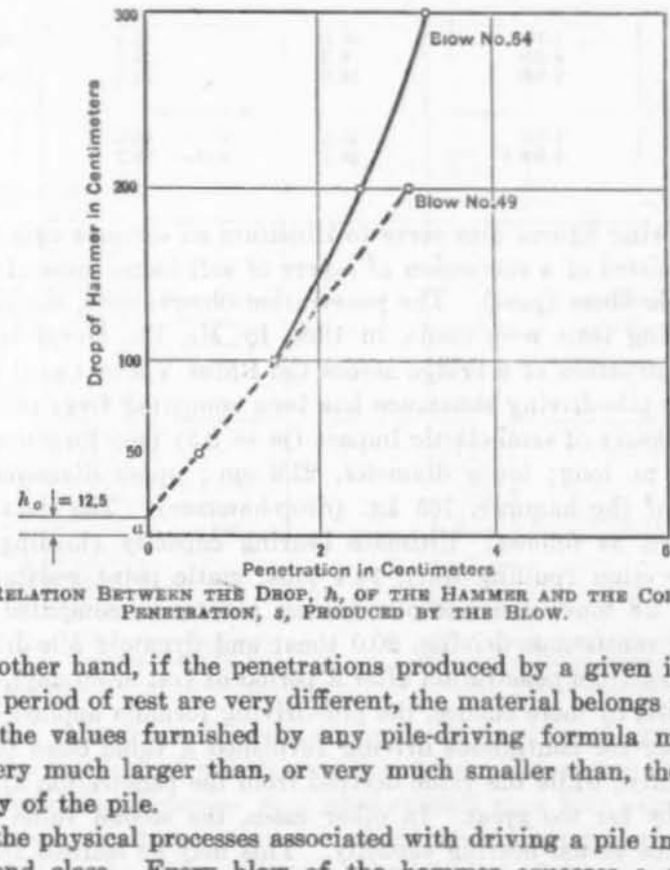


FIG. 7.—RELATION BETWEEN THE DROP, h, OF THE HAMMER AND THE CORRESPONDING PENETRATION, s, PRODUCED BY THE BLOW.

On the other hand, if the penetrations produced by a given impact before and after a period of rest are very different, the material belongs in the second class, and the values furnished by any pile-driving formula may be either equal to, very much larger than, or very much smaller than, the static bearing capacity of the pile.

Notice the physical processes associated with driving a pile into a material of the second class. Every blow of the hammer squeezes a certain quantity of water out of the soil beneath the point of the pile. The water escapes toward the surface through the space between the pile and the ground, and forms a film acting as a lubricant, just as any liquid does if trapped between two surfaces. Due to the presence of this film, the friction acting along the surface of the pile is very much less, whereas the force required to squeeze the water rapidly out of the soil beneath the point of the pile is greater than that required to compress the same soil slowly. During a period of rest, the film of water is gradually absorbed by the soil and the full static pile friction develops. When pile-driving is resumed, one has to overcome both the static friction and the dynamic point resistance. However, during the same period of rest the annular space, serving as an outlet for the water squeezed out of the soil by impact, has closed. Hence, in certain kinds of soils, the dynamic point resistance acting after a period of rest may be much more than such resistance acting before the rest.

TABLE 1.—COMPARISON OF THE BEARING VALUE OF PILES.

Drop of hammer, in feet.	Penetration, in inches.	Ultimate Bearing Capacity, in Tons, by:		Remarks.
		Engineering News formula.	Semi-elastic impact theory.	
6.57	1.160	21.0	25.4	Should be equal
1.64	0.236	9.2	26.5	
3.30	0.590	14.2	24.6	
6.57	1.160	23.0	30.3	....
9.83	1.280	29.7	35.2	....

The following figures may serve to illustrate an extreme case of this type. The soil consisted of a succession of layers of soft loam, some of them mixed with vegetable fibers (peat). The penetration observations, the loading tests, and the pulling tests were made, in 1904, by Mr. Ph. Krapf in connection with the construction of a bridge across the Rhine Valley Canal in Austria.* The dynamic pile-driving resistance has been computed from the penetration data by the theory of semi-elastic impact ($m = 0.5$) (see Equation (3)). The pile was 8.4 m. long; lower diameter, 23.6 cm.; upper diameter, 33.3 cm.; and weight of the hammer, 765 kg. (drop-hammer). The data concerning this pile were, as follows: Ultimate bearing capacity (loading test), 17.2 tons; skin friction (pulling test), 14.4 tons; static point resistance, 17.2 − 14.4 tons = 2.8 tons; dynamic pile-driving resistance, computed from penetration after continuous driving, 20.0 tons; and dynamic pile-driving resistance, computed from penetration after a period of rest of 30 days, 80.0 tons.

In this case, by mere chance, the pile-driving formula applied to the effect of the hammer for continuous driving furnished a value close to the actual bearing capacity; while the value derived from the penetration after a period of rest was by far too great. In other cases, the second value is found to be closer to the actual bearing capacity. This may be learned from the customary practice of introducing into the *Engineering News* formula the values obtained after a period of rest.

Based on what is known about the physics of the penetration of a pile in the second class of material, the aforementioned data can be interpreted.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf, Leipzig, 1906.

While continuously driving the pile into the ground, the skin friction is practically eliminated. Hence, the dynamic pile-driving resistance is practically equal to the dynamic point resistance; or, before the period of rest, the dynamic point resistance is 20.0 tons, as compared with 2.8 tons static point resistance.

After a 30-day period of rest, the hammer had to overcome the full static friction (17.2 tons) plus the dynamic point resistance (62.8 tons); or a total of 80 tons.

Fig. 8 represents this interpretation graphically. The shaded areas correspond to the pile friction acting during the pile-driving process (*a*); under impact after a period of rest (*b*); and under static load (*c*).

This and similar examples show that the application of any pile-driving formula to the bearing capacity of piles driven into the second class of materials is a gamble, trusting that the deficiency in skin friction associated with the driving of the pile may, by chance, be compensated by the corresponding excess in point resistance.

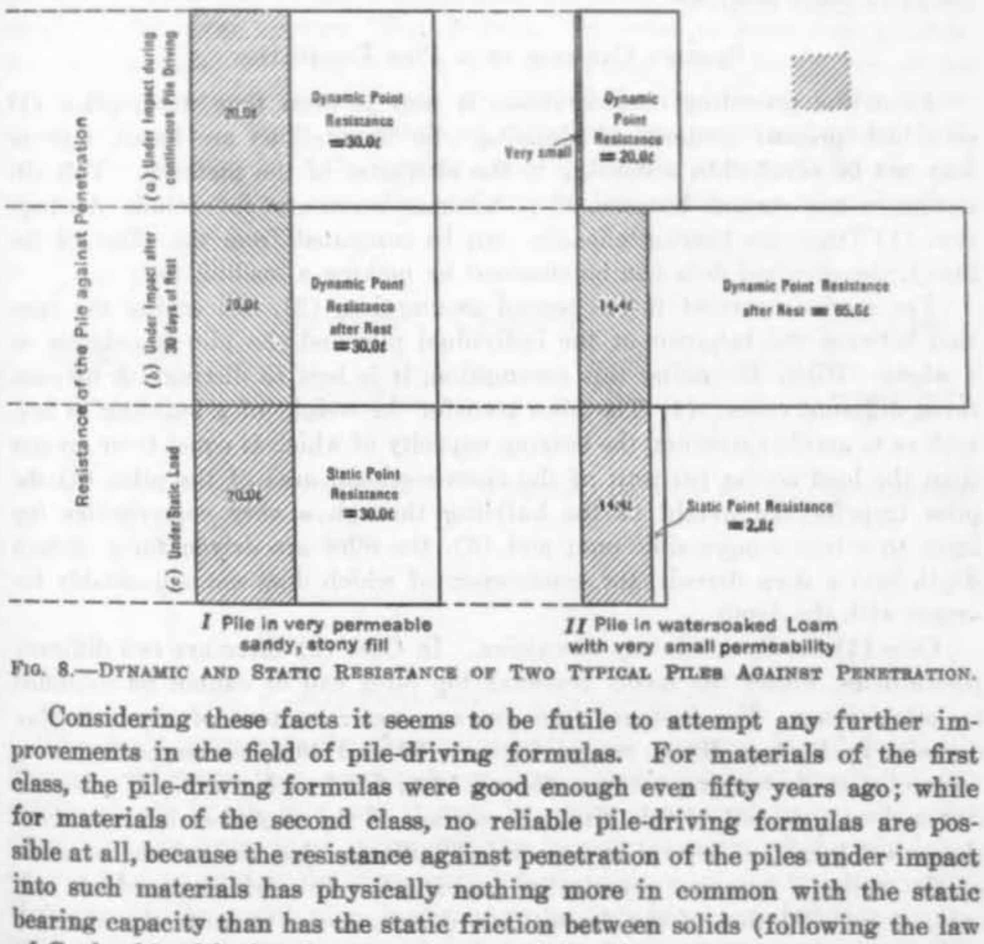


FIG. 8.—DYNAMIC AND STATIC RESISTANCE OF TWO TYPICAL PILES AGAINST PENETRATION.

Considering these facts it seems to be futile to attempt any further improvements in the field of pile-driving formulas. For materials of the first class, the pile-driving formulas were good enough even fifty years ago; while for materials of the second class, no reliable pile-driving formulas are possible at all, because the resistance against penetration of the piles under impact into such materials has physically nothing more in common with the static bearing capacity than has the static friction between solids (following the law of Coulomb) with the viscous resistance of liquids against rapid deformation (following the law of Newton). In the future, the penetration curve of piles, and particularly the difference between the rate of penetration before and after a period of rest, may give more reliable information about the character of a ground of the second class than a test boring, but it never will give any reliable information about the bearing capacity of the piles.

Therefore, future investigations in the field of bearing capacity of piles should be directed toward studying the effect of the shape, thickness, and length on the bearing capacity of piles driven into different kinds of soils. In this connection attention should be called to the exhaustive investigations by the engineers of the Whangpoo Conservancy Board, in Shanghai, China. The tests included more than forty complete driving, loading, and pulling tests in soils the nature of which was previously investigated by test borings. The soil consisted principally of clay and silt, apparently belonging to the second class of materials. The tests furnished valuable information concerning the effect of the shape and length of the piles on the ultimate bearing capacity. Characteristically enough, the published reports do not contain any information on the penetration observations, for the reason that "the application of general pile-driving formulas to the local conditions has resulted in great errors".*

Bearing Capacity of a Pile Foundation

From the preceding considerations it may be seen that Assumption (1) on which present methods of planning pile foundations are based, may or may not be admissible, according to the character of the material. This situation is not serious because, if it becomes necessary to exclude Assumption (1) (that the bearing capacity can be computed from the effect of the blow), the required data can be obtained by making a loading test.

Far more important is the second assumption, (2), concerning the relation between the behavior of the individual pile and the pile foundation as a whole. When discussing this assumption, it is best to distinguish between three different cases: (1) The piles transfer the weight of a building to bedrock or to another stratum, the bearing capacity of which is equal to or greater than the load acting per unit of the cross-sectional area of the pile; (2) the piles transfer the weight of the building through a very compressible top layer to a less compressible one; and (3), the piles are driven for a certain depth into a deep deposit, the consistency of which does not appreciably increase with the depth.

Case (1) hardly needs any discussion. In Case (2) there are two different possibilities; either the feebly resistant top layer can or cannot be solidified by pile-driving. For instance, the average water content of the soft clay deposits in Boston, Mass., ranges between 30 and 40%, corresponding to a volume of voids ranging between 45 and 52%, filled with water. If piles are driven into a deposit of this kind, the surface of the clay rises between them through a height of several inches, which indicates that the volume of voids of the material remains practically unchanged. Pile-driving produces not only no consolidation of the deposit, but it even seems to cause the material to become softer. This last conclusion has been drawn from the observation

* Whangpoo Conservancy Board, S. H. T. Series 1, No. 7. Various reports to the Engineer-in-Chief on special investigation, Shanghai, 1921; report to the Engineer-in-Chief on pile tests.

that the compressive strength of an undisturbed sample of clay is always considerably greater than that of the same sample, with the same water content, after the sample has been moulded by external pressure. In striking contrast to this behavior of the deposits of blue clay stands the behavior of other very fine-grained deposits. Quite recently, when piles were driven into a deposit of exceedingly fine-grained saturated quicksand, the surface of the deposit subsided between the piles through a distance of almost 1 ft. Judging from previous experience, the original volume of voids of the material was certainly not greater than 45%, corresponding to the volume of voids of a clay with a water content of 30 per cent. Nevertheless, based upon the subsidence of the surface of the deposit, the pile-driving must have reduced the volume of voids by several per cent., which, in turn, involves a considerable increase in the bearing power and a decrease of the compressibility of the deposit.

In these two cases, the piles must serve two very different purposes if they are to be utilized to full advantage. In the first case, they should transfer the load to the more resistant sub-stratum, thus diverting the pressure from the upper to the lower stratum (Fig. 9 (*a*)). In order to serve this purpose, the bearing capacity of the section, *b c*, of the pile below the bottom of the upper layer must be great enough to support the load without any appreciable settlement. Since the result of a loading test performed on an individual pile includes both the bearing capacity of this section of the pile and the skin friction acting along the section, *a b*, this result may be misleading.

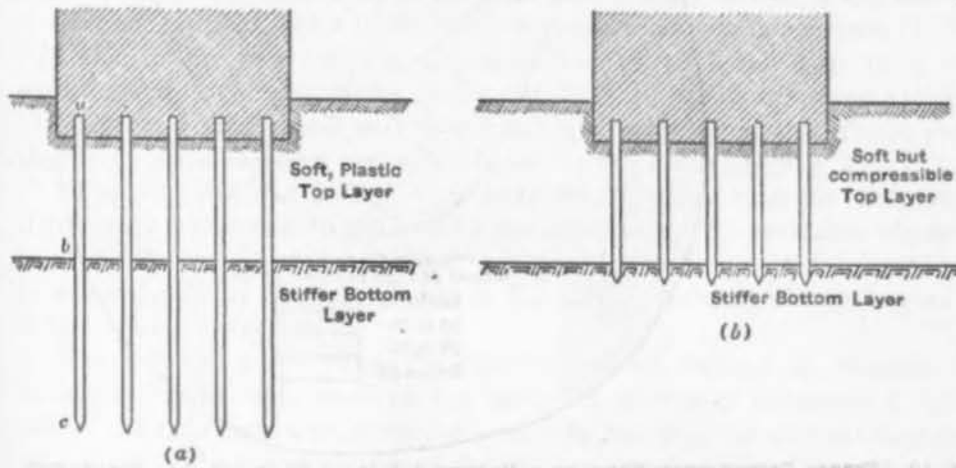


FIG. 9.—PILE FOUNDATION ON A TWO-LAYER SYSTEM.

Fig. 9 (*b*) represents the same combination of strata as Fig. 9 (*a*), with the only difference that the top layer is apt to be solidified (artificial fill, quicksand, etc.). In this case it may be more economical to use the piles merely for the purpose of increasing the bearing capacity of the top layer by pile-driving, letting the piles extend to the bottom of the top layer only. Here, again, the result of a loading test performed on an individual pile, prior to driving the others, would fail, by far, to furnish any reliable information concerning the number of piles required. What is needed is to learn about the effect of driving on the density of the surrounding soil. Deposits that can be consolidated by pile-driving belong almost exclusively to those classes of material for which the pile-driving formulas are valid. The proper procedure would be to start with piles far apart and to drive intermediate piles until the bearing capacity, computed from the penetration, indicates the resistance of a well-consolidated ground.

In the third case (piles driven into the upper part of a very deep deposit with a fairly uniform consistency), the value of the piles may or may not be problematical, depending on the ratio between the width of the foundation and the length of the piles. Suppose that the pressure diagrams, Fig. 10 (*a*) and Fig. 10 (*b*), have been plotted. The left-hand sections of these diagrams show the distribution of soil pressures beneath two raft foundations with different widths, computed approximately by Boussinesq's theory. The right-hand sections of the same diagrams show the change in the stress distribution due to the presence of 20-ft. piles in the ground, computed by the same theory.

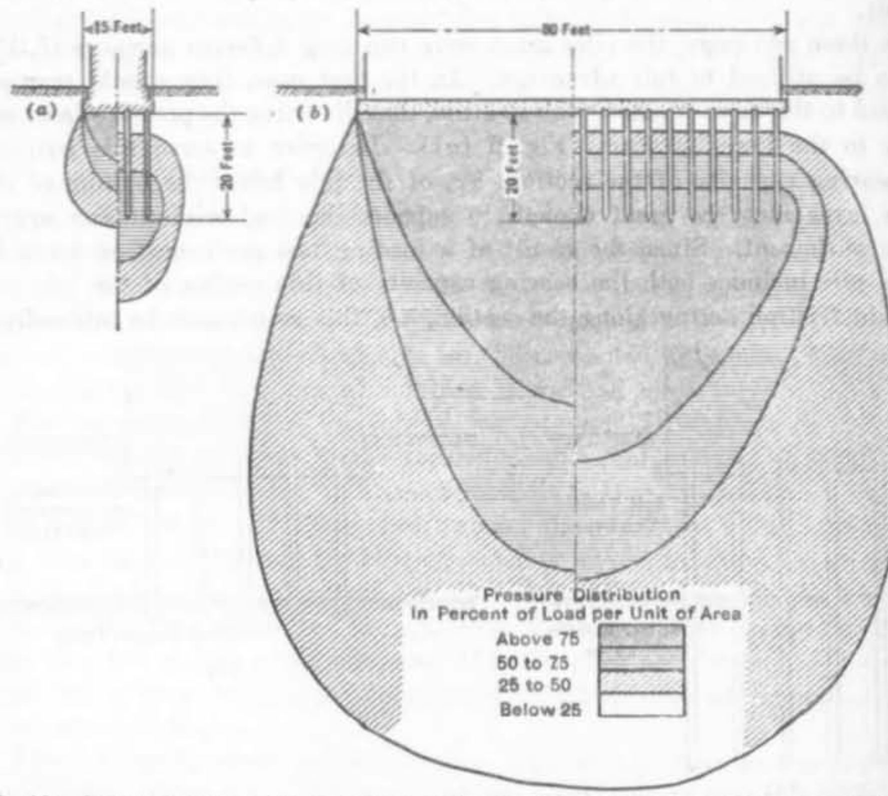


FIG. 10.—STRESS DISTRIBUTION BENEATH A NARROW (*a*) AND A WIDE (*b*) PILE FOUNDATION.

If the length of the piles is at least equal to the width of the base, their effect is obviously very beneficial (Fig. 10 (*a*)). The piles reduce the intensity of the maximum pressure acting on the ground, and, in addition, they shift the zone of maximum stress from the surface to the level where their lower ends are located. Since the effect of the depth of foundation on the bearing capacity of the ground depends, as previously noted, not on the depth, *t*, but on the ratio between the depth of foundation and its width, the effect of transferring the pressure to a deeper level, in the case of Fig. 10 (*a*), may be quite important. On the other hand, if the width of the foundation is considerably greater than the length of the piles (Fig. 10 (*b*)), the pressure-reducing effect of the piles is very small and the ratio between the depth of foundation and the width of the structure is also very much smaller than in the case of Fig. 10 (*a*). Hence, the beneficial effect of the piles may be negligible and the money invested in them may represent an unwarranted expenditure. The reason piles are so generously used beneath raft foundations, in spite of the facts represented in Fig. 10, is that many engineers confound the bearing capacity of the individual piles with the bearing capacity of a score of them. Yet one can hardly conceive of a more obvious fallacy. The individual pile spreads the load over a wide area, thus reducing the specific soil pressure to a negligible item, while beneath a wide pile foundation, the soil pressure is just as great as it is under a simple raft foundation of equal width. In structural engineering such a procedure would correspond to considering that, since a bridge has stood the load of a single car without measurable deflection, it will stand the load of 200 cars, acting simultaneously.

Some years ago a company intended to construct a power house with a raft foundation supported by 500 reinforced concrete piles, 25 ft. long, because a similar building resting without piles on the same mud deposit had suffered a subsidence of more than 12 in. A loading test performed on an individual pile showed that a weight of 4 tons did not produce a measurable sinking of the pile. As a consequence, it was believed that, if a load of not more than 2 tons were assigned to each pile, no settlements would occur. After the mud deposit was investigated, the company was advised to sell the piles and construct a simple raft foundation, because conditions were similar to those shown in Fig. 10(*b*). However, as some of the piles were already driven, it was decided to drive all of them. Fig. 11(*a*) shows the settlements of the corners (I, IV, VII, X), of the new building (*a*) supported by piles and Fig. 11(*b*) the simultaneous settlements of the corners (A, B, E, F) of the adjoining building (*b*), supported by a simple raft foundation and exerting on the ground practically the same pressure per unit of area as the building (Fig. 11(*a*)).

By keeping the load, acting on the individual pile, less than the "safe load", it is merely transferred to the level of the pile points. No assurance whatever is obtained as to how the ground below the pile points will behave, and in cases of piles of the type shown in Fig. 10(*b*), the ground may behave as if they were non-existent.

Hence, in all cases where the piles do not act strictly as columns, the knowledge of the "safe load" of the individual piles only represents a minor part of the information required for predicting the behavior of the foundation as a whole. The essential part of the problem consists of studying the effect of the load on the ground located around the piles and beneath the points. Yet building codes are satisfied with specifying the load per pile, disregarding the true cause of future complications.

Effect of Freezing on Foundations

During recent years valuable contributions have been made to current knowledge of the effect of frost in the soil and on the structures supported by it. Most important among them are the investigations of Professor Stephen Taber,* of the University of South Carolina, concerning the capacity of

* "The Growth of Crystals Under External Pressure," *American Journal of Science*, 4th Series, Vol. 41, pp. 532–556; "Pressure Phenomena Accompanying the Growth of Crystals," *Proceedings*, National Academy of Sciences, Vol. 3, No. 4, pp. 297–302.

freezing veins of water to absorb additional water out of the surrounding clay soil, or through the clay soil from the ground-water, and to develop from thin seams into layers with a considerable thickness. The space required for the growth is produced by the pressure of crystallization, forcing the soil out of the way.

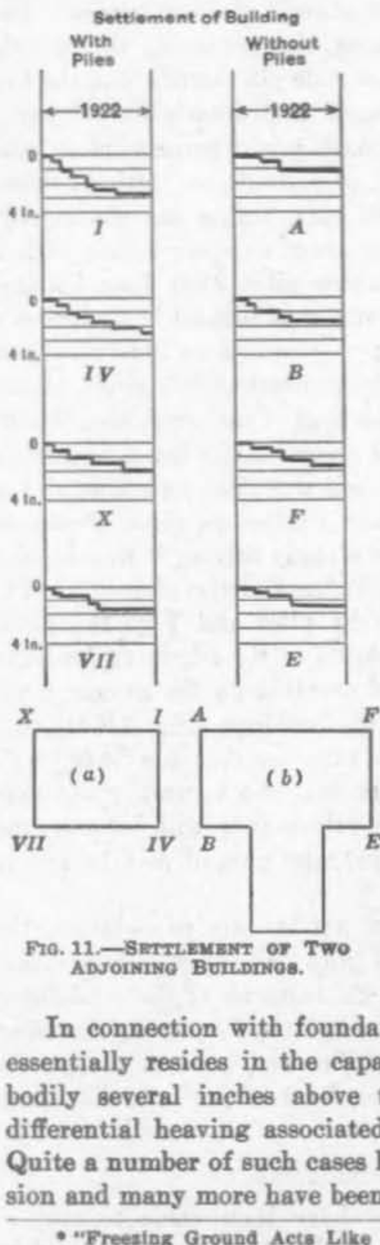


FIG. 11.—SETTLEMENT OF TWO ADJOINING BUILDINGS.

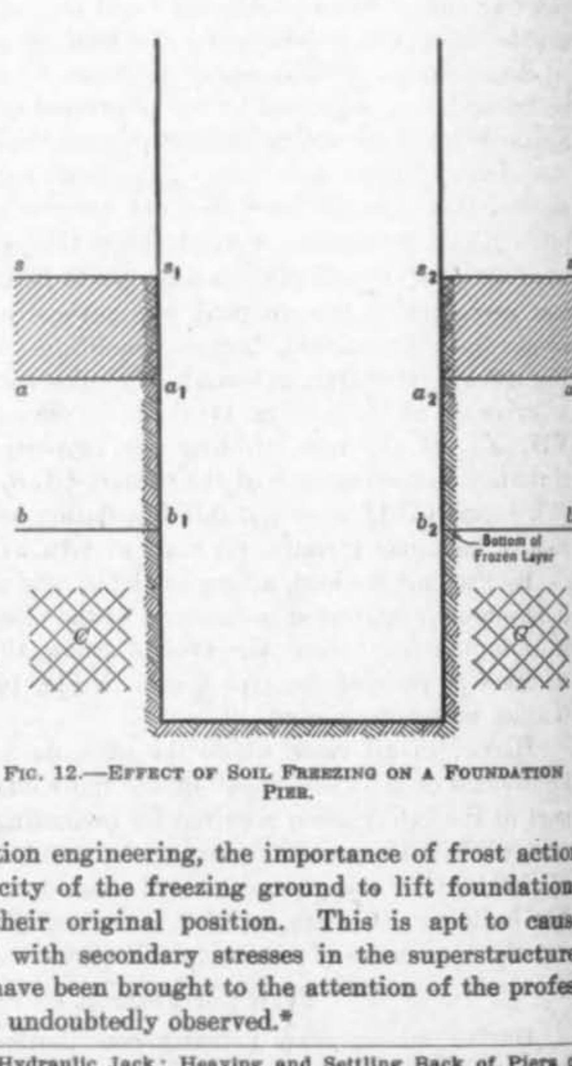


FIG. 12.—EFFECT OF SOIL FREEZING ON A FOUNDATION PIER.

In connection with foundation engineering, the importance of frost action essentially resides in the capacity of the freezing ground to lift foundations bodily several inches above their original position. This is apt to cause differential heaving associated with secondary stresses in the superstructure. Quite a number of such cases have been brought to the attention of the profession and many more have been undoubtedly observed.*

* "Freezing Ground Acts Like Hydraulic Jack: Heaving and Settling Back of Piers of Railroad Bridge Over City Street, Lead to Interesting Deduction," by H. J. Gilkey, *Engineering News-Record*, Vol. 79 (1917), pp. 360–361; "Some Observations on Effect of Frost in Raising Weight," by L. B. Wyckoff, *Engineering News-Record*, Vol. 80 (March 28, 1918), pp. 627–628; "Water Expansion in Ground Cause of Heaving in Winter," by C. D. Norton, *Engineering News-Record*, Vol. 80 (May 30, 1918), p. 1058; "Action of Frost in Heaving Concrete Piers," by L. DeB. McCready, *Engineering News-Record*, Vol. 91 (August 30, 1923), p. 360.

However, thus far, apparently no attempt has been made to go beyond the observed phenomena and to analyze the conditions required for producing the lifting effect. The following conclusions represent the results of a preliminary survey of these conditions.

The freezing starts at the surface of the ground and gradually proceeds toward the interior. At the same time the soil freezes to the pier, the adhesion being practically equal to the shearing strength of the frozen soil. Suppose that at some intermediate state, freezing has proceeded from the surface, *S-S* (Fig. 12), to the level, *aa*, the frozen soil being cemented to the pier along the faces, $a_1\ s_1$ and $a_2\ s_2$. If the freezing process continues still farther, causing the freezing and the expansion of the soil located within the space, *a a b b*, there are then three possibilities:

(*a*) The ground beneath the freezing layer is feebly compressible, and the adhesion between the faces, $a_1 s_1, a_2 s_2$, and the soil may be greater than the weight of the pier. In this case the pier should be lifted bodily, as freezing proceeds, leaving an empty space beneath it.

(*b*) The ground is compressible, but the adhesion along the faces, $a_1\ s_1$ and $a_2\ s_2$, is greater than the weight of the pier. In this case, with the pier as a reaction, the freezing soil will exert a downward pressure causing a small, but permanent, consolidation of the ground within the spaces, *C-C*.

Every frost season will add some additional consolidation, until, after several seasons, the soil located within *C-C* will be so compact, that finally a frost will succeed in lifting the pier, the consolidated material, *C-C*, acting as a reaction.

(*c*) The ground is feebly compressible, but the weight of the pier is greater than the adhesion. In this case the expansion of the soil, *a a b b*, will cause a slow upward creep of the soil, *s s a a*, along the outside of the pier, associated with an upward lift equal to the adhesion. The result should be a fatigue effect, similar to repeated application and removal of a live load, causing slight additional settlements of the pier.

From these remarks it may be learned that the lifting action of the expanding ground should depend on several factors, namely, the compressibility of the unfrozen foundation soil, the compressive strength of the frozen soil, the depth of freezing, and, finally, on the ratio between the weight of the pier and the total adhesion between the pier and the frozen ground. As experiences increase, other factors may have to be added to this list. Considering the scarcity of available information, any observations made in this field may increase the engineer's capacity for predicting the effect of freezing on proposed foundations.

Soil Classification Based on Elastic Constants of Soils

The preceding parts of this paper have brought out the following facts: (1) The settlement produced by a given unit load may increase either in direct proportion with the diameter of the loaded area, or at a very much smaller rate, depending on the character of the soil; (2) the settlement of a building may be due to volume change combined with lateral flow, or to lateral flow alone, depending on the character of the soil; (3) the pile-driving formulas may furnish fairly reliable or utterly inconsistent results, depending on the character of the soil; and (4) the driving of piles into a soft top layer may cause a softening of the soil associated with a rise of the surface, or consolidation associated with subsidence, depending on the character of the soil.

The fundamental requirement for bringing the manifold foundation experience into a rational working system consists of establishing a system for the classification of soils based essentially on those characteristics that are of engineering importance.

Thus far, attempts to classify soils (including the revised soil classification* scheme of the Special Committee on the Bearing Value of Soils for Foundations, etc., of the Society) have been based essentially on such properties of soils as: (*a*) mineral composition; (*b*) volume of voids; (*c*) grain composition (result of mechanical analysis); (*d*) water content; and (*e*) percentage of colloidal material present in the soil. A study of these factors, covering a period of several years, has disclosed that:

(*a*) The mineral composition of very fine-grained soils cannot be determined except by elaborate optical or chemical methods, the cost of which is forbidding. Even if it were possible to make such a determination, the benefit would be doubtful.

(*b*) The volume of voids is so complicated a function of the shape and uniformity of the grains that it is impossible to correlate it with any definite properties of the soil, even if both the uniformity and the effective size of the material are known.

In 1926, the writer investigated the permeability of two sets of samples; one coming from a deposit of modified glacial drift near Westfield, Mass., and the other from a glacial lake deposit near Springfield, Mass. The effective size of both materials ranged between 0.01 mm. and more than 1 mm. In order to simplify the laboratory work, an effort was made to determine for each set the relation that exists between the uniformity coefficient (according to the well known definition of Allen Hazen, M. Am. Soc. C. E.) and the volume of voids. Fig. 13 shows the results of this investigation. Although there was a slight difference only between the shape of the grains and the mica content of the various samples, the relation between the volume of voids and the uniformity coefficient was found to be very erratic. To quote an example: At a voids' ratio of 0.8 (volume of voids of 45%), Soil No. 7, from Westfield, is as compact as it could possibly be and, if loaded, it would have a considerable bearing capacity. At the same voids ratio of 0.8 (volume of voids of 45%), Soil No. 34, from the same locality, would be very compressible, although its uniformity coefficient is practically equal to that of Soil No. 7.

(*c*) A mechanical analysis, according to the revised size grades of the Society's Special Committee on the Bearing Value of Soils for Foundations, etc., involves more than two weeks of work per sample, and the costs are forbidding. Nevertheless, the writer performed more than one hundred complete

* *Proceedings*, Am. Soc. C. E., February, 1921, Papers and Discussions, p. 17.

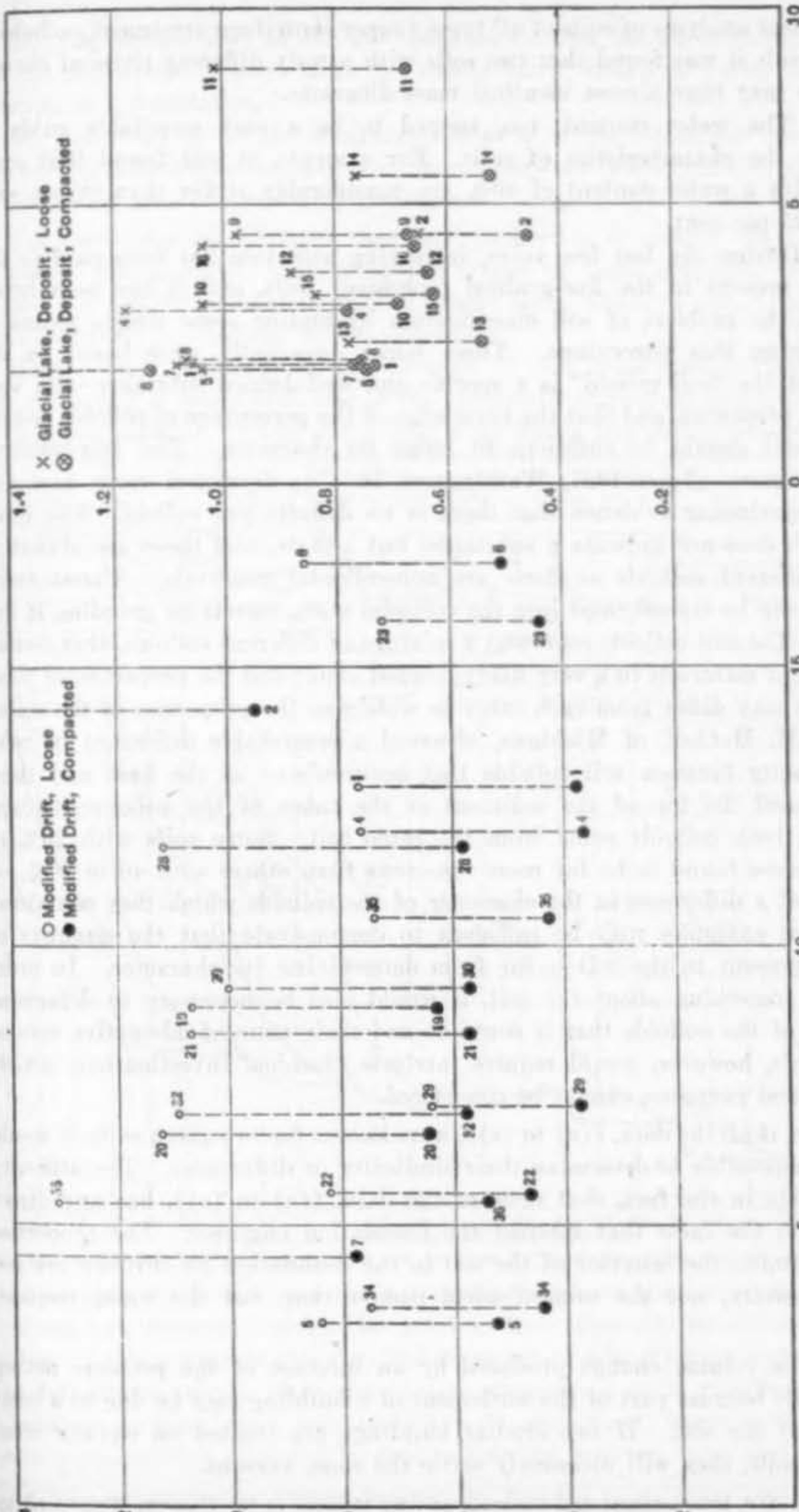


FIG. 13.—RELATION BETWEEN VOIDS RATIO AND UNIFORMITY.

mechanical analyses of soils of all types (super-centrifuge treatment excluded). As a result it was found that two soils with utterly different physical characteristics may have almost identical mass diagrams.

(*d*) The water content, too, seemed to be a very unreliable guide in judging the characteristics of soils. For example, it was found that some clays with a water content of 40% are considerably stiffer than others with one of 25 per cent.

(*e*) During the last few years, increasing attention has been paid to the colloids present in the fine-grained (cohesive) soils, and it has been hoped to solve the problem of soil classification by finding some simple means of determining this percentage. These hopes apparently were based on the idea that the "soil colloid" is a specific and well-defined substance with very unusual properties, and that the knowledge of the percentage of colloids present in the soil should be sufficient to judge its character. The International Soil Congress (June 1927, Washington, D. C.), developed some new and rather convincing evidence that there is no definite soil colloid. The term, "colloid", does not indicate a substance, but a state, and there are almost as many different colloids as there are non-colloidal materials. Almost every mineral can be transformed into the colloidal state, merely by grinding it fine enough. The soil colloids represent a mixture of different colloids, that means, of different materials in a very finely divided state; and the properties of these mixtures may differ from each other as widely as the properties of the soils.* Mr. M. M. McCool, of Michigan, observed a remarkable difference in color and tenacity between soil colloids that accumulated at the base and those that formed the top of the sediment in the tubes of the super-centrifuge, although both colloids came from the same soil. Some soils with 20% of colloids were found to be far more tenacious than others with 40 to 70%, on account of a difference in the character of the colloids which they contained. These few examples may be sufficient to demonstrate that the quantity of colloids present in the soil is far from determining its character. In order to learn something about the soil, it would also be necessary to determine the type of the colloids that it contains and their state of adsorptive saturation. This, however, would require intricate chemical investigations which, for practical purposes, cannot be considered.

Hence, if all the data, ((*a*) to (*e*)), were known for two given soils, it would still be impossible to determine their similarity or differences. The difficulty is obviously in the fact, that none of the data, ((*a*) to (*e*)), has any direct bearing on the facts that interest the foundation engineer. The properties that determine the behavior of the soil in the foundation pit directly are not the uniformity, nor the mineralogical composition, nor the water content. They are:

(1) The volume change produced by an increase of the pressure acting on the soil; because part of the settlement of a building may be due to a compression of the soil. If two similar buildings are erected on equally compressible soils, they will ultimately settle the same amount.

* "The First International Soil Congress and Its Message to the Highway Engineer," by C. Terzaghi, *Public Roads*, Vol. 8, No. 5, pp. 89–94.

(2) The permeability of the soil; because the less the permeability of the soil the more time it takes until the excess water drains out after the construction of a foundation.

(3) The cohesion or the shearing resistance of the soil under zero load; because the cohesion determines the relation between settlement and diameter of the loaded area at equal unit pressures.*

The determination of these three soil properties is enough for practical purposes. The question concerning the causes of these properties—whether or not they are determined by colloid constituents, effective size, or mineralogical composition—belong in the laboratory.

Hence, when approaching the problem of soil classification, the writer first attempted to study each one of the practically important properties, ((1) to (3)), individually, to find out within what limits they could possibly vary.

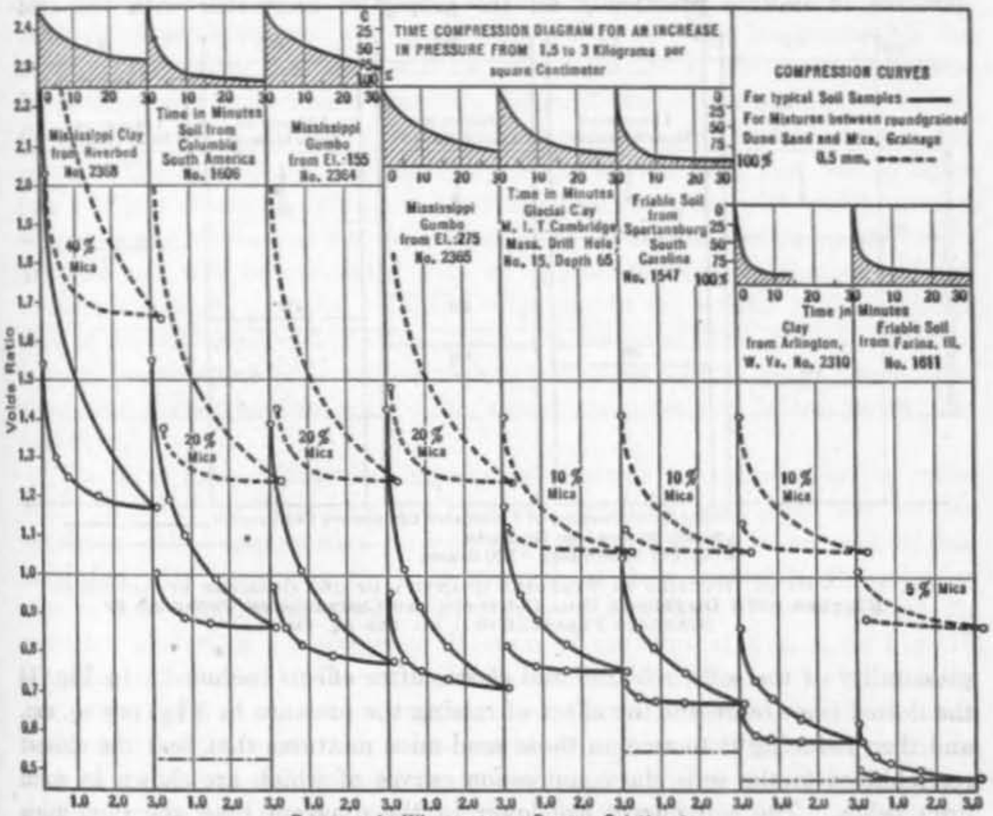


FIG. 14.—PRESSURE VOLUME DIAGRAMS FOR SAND-MICA MIXTURES AND FOR TYPICAL SOILS.

A careful study disclosed the surprising fact that the compressibility of soils may vary between limits as far apart as the compressibility of concrete and rubber. To demonstrate this fact the diagrams of Fig. 14 have been plotted. The full drawn curves show the volume changes of soils, ranging between Mississippi gumbo (most compressible soil) and clean, round-grained sand (least compressible), produced by first raising the pressure from zero to 3 tons

* "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

per sq. ft. (3 kg. per sq. cm.), and then gradually reducing the pressure to zero. At the outset the water content of each one of the samples was equal to the water content of the material after slow sedimentation in quiet water (Atterberg's lower liquid limit). From Fig. 14 it may be learned that the compressibility of the sand amounts to only a small fraction of the compressibility of the Mississippi gumbo. Even a year ago (1925) the cause of the tremendous difference in the compressibility of different soils was not yet quite clear to the writer. Suspecting that it might be due to the greater or smaller abundance of scale-like particles in the soil, he induced Glennon Gilboy, Jun. Am. Soc. C. E., of the Massachusetts Institute of Technology, to investigate the elastic properties of differently proportioned mixtures of sand and mica, both with a grain size of 0.5 mm.* The results of these investigations certainly were striking. By properly selecting the mica content of the sand, it was possible to imitate practically all the properties associated with the com-

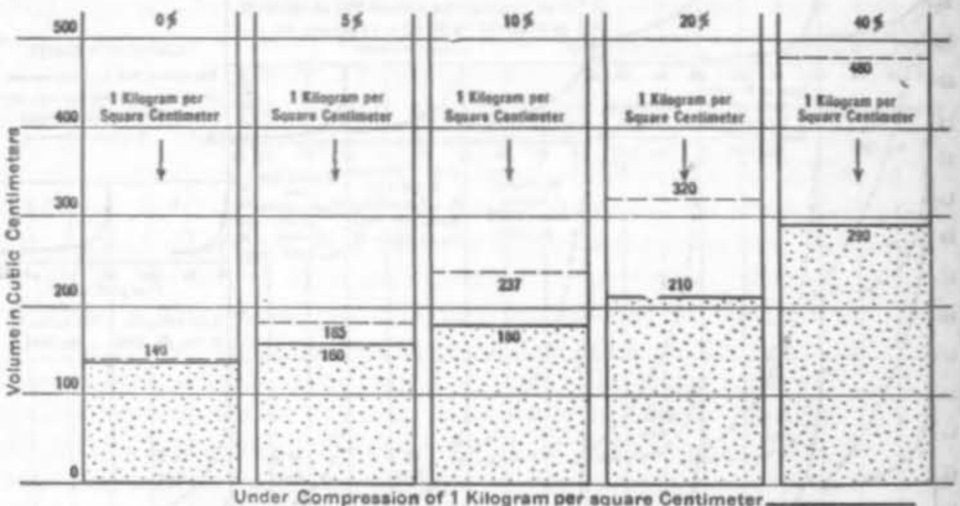


FIG. 15.—VOLUME OCCUPIED BY STANDARD QUANTITY OF 200 GRAMMES OF SAND-MICA MIXTURE WITH DIFFERENT MICA CONTENTS AND COMPRESSION PRODUCED BY STANDARD PRESSURE OF 1 KG. PER SQ. CM.

pressibility of the soils, rebound and elastic after effects included. In Fig. 14 the dotted lines represent the effect of raising the pressure to 3 kg. per sq. cm. and then reducing it to zero on those sand-mica mixtures that bear the closest resemblance to the soils the compression curves of which are shown in solid lines below. The soil curves are lower in the diagram than the sand-mica curves merely because of a difference in uniformity of the two materials. In order to make these facts clearer, the diagrams, Fig. 15, have been prepared. The upper row of numbers shows the space occupied by a standard quantity of 200 grammes of sand-mica mixtures with a mica content of 0%, 5%, 10%, 20%, and 40%, respectively. The lower row shows the space occupied after compression produced by a standard pressure of 1 ton per sq. ft.

Thus, it becomes evident that one of the most important properties of the soils—compressibility—has nothing to do with the effective size, the uni-

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 555.

formity, or the colloid content. It is merely the mechanical effect of a greater or smaller abundance of scale-like particles.

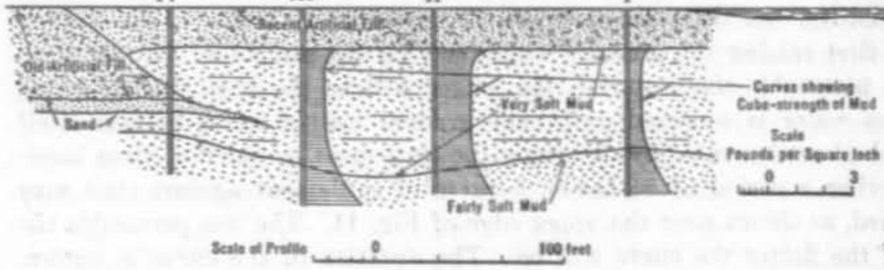
The property next in importance concerns the permeability of soils. The first tests that the writer made for determining the permeability of very fine-grained materials, such as clays, lasted several months.* However, a method has been devised, based on the theory of hydrodynamic stresses, by which the coefficient of permeability can be determined from not more than one dozen readings. By using the same sample that serves for investigating the compressibility, the determination can be made in 24 hours.† The test is made by first raising the pressure which acts on the sample by 100 per cent. The less permeable the material, the longer will be the time during which the excess water is squeezed out. The gradual loss of water betrays itself by a gradual settlement. By observing the settlement in specified time intervals covering a period of 24 hours, a curve of settlement against time may be obtained, as shown near the upper edge of Fig. 14. The less permeable the material, the flatter the curve will be. The equation of the curve is known. It contains only one variable quantity, the coefficient of permeability. Therefore, it is merely a matter of arithmetic to compute this coefficient from the curves. The coefficient of permeability is the second item that has to enter into a system of soil classification. It determines the speed with which the soil will settle and whether or not the settlement of a building resting on very compressible soil will be associated with an appreciable volume change. If the coefficient is high, the total volume change due to the weight of the building may be accomplished within a few years. On the other hand, if this coefficient is very low, no appreciable volume change may occur within one or two generations, and the settlements will be due almost exclusively to lateral flow of the loaded soil.

The third item of practical importance concerns the cohesion of the material. In order to determine the cohesion, methods have been devised for obtaining drill samples with their original water content in an undisturbed condition. By measuring the cube strength of these materials, the cohesion can be determined with a sufficient degree of accuracy. The results of the tests are plotted in a consistency diagram of the type of Fig. 4, or Fig. 16. By using this method of investigation it was found that there are clay deposits that are still in an undrained condition, which means that their consolidation under the influence of their own weight is still proceeding. As examples may be mentioned certain mud deposits along the shores of the Golden Horn in Constantinople; parts of the blue clay deposit which underlies Boston and Cambridge, Mass.; and a clay deposit in Detroit, Mich. There is no doubt that many others of a similar kind exist. The surface of such deposits would gradually subside even if no buildings were erected on them, and the construction of buildings accelerates the process, involving considerable settlements regardless of the type of foundation.

* "Die physikalischen Grundlagen des Technisch-geologischen Gutachtens," by Charles Terzaghi, M. Am. Soc. C. E., *Zeitschrift* des Oesterreichischen Ingenieur- and Architekten Vereins, September, 1921.

† "Die Berechnung der Durchlässigkeitsziffer der Tone aus dem Verlauf der hydrodynamischen Spannungserscheinungen." *Sitzber.* der Akad. der Wiss. in Wien, Math. Natur., Abt. IIa, 1923.

The first two items, compressibility and permeability, determine the type of soil with a greater precision than the concrete term, "1 : 2 : 4", describes the property of artificial stones. A third item, cohesion (shearing resistance) for clays, and density for sands and quicksands, determines the state in which the soil occurs. The methods for measuring the density (firmness) of sandy materials have not yet been developed, but there seems to be no insurmountable obstacle to the solution of the problems involved.



Roman Numerals indicate Drill Holes

Fig. 16.—Consistency Diagram for a New Mud Deposit at Aïwan Seraï, near Constantinople, Turkey.

With these three items as a basis, a soil classification could be devised which would serve to bring soils into as rational a system as that which covers artificial construction materials, because soils with equal compressibility, equal permeability, and occurring in the same state, will behave, under load, in an identical manner, regardless of what the ultimate causes of their properties may be.

However, in practice, there are two serious difficulties associated with any attempt to classify soils according to the method proposed in the preceding discussion. The first is the difficulty of obtaining undisturbed samples of certain soils. According to the results obtained by Swedish investigators, disturbing a soil without changing its volume of voids and its water content may cause the cohesion (consistency) of the soil to be reduced by an amount ranging between 75 and 96% of its original value. Hence, the conclusions derived from testing the consistency of a drill sample may be very misleading unless one has succeeded in securing the sample in an undisturbed state.

The second difficulty is due to the fact that the character of the natural ground varies to a greater or less extent from foot to foot, in a horizontal and in a vertical direction, the variations depending on how the soil deposit was formed. In order to get an accurate conception of the average character of the soil, a considerable number of samples ought to be tested. The tests required for determining the constants mentioned in the preceding discussion are too expensive, in time and money, to be performed on every individual sample. Hence, efforts are made to work out certain simple routine tests, the results of which would furnish the approximate information desired at a reasonable amount of time and labor.

Conclusions

Considering the present state of knowledge in the field of soil mechanics, the prospects concerning the future of foundation engineering as an applied science are decidedly encouraging, the principal obstacles against progress in this field having been removed. The elastic properties of the most troublesome soils, clays included, are now at least as thoroughly known as those of concrete or steel. The methods of soil classification and soil identification have reached a point where it can be clearly seen what is needed for identifying soil materials. The relations between the size of the loaded area, depth of foundation, character of the soil, and intensity of the load are well analyzed, at least in principle, and the physics of the time effects are clearly understood. Yet, stress must be laid on the fact that the knowledge thus obtained merely serves as a means for uprooting certain persistent prejudices (for instance, concerning the value of pile-driving formulas, or the interpretation of the results of loading tests) and establishing a more accurate interpretation of actual construction experience. The bulk of the work—the systematic accumulation of empirical data—remains to be done.

Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results. By systematically accumulating such knowledge, the empirical data being well defined by the results of adequate soil investigations, foundation engineering could be developed into a semi-empirical science, comparable in its character to certain branches of medicine. For the first time a large-scale effort of this kind was made by the Swedish Geotechnical Committee, in its epochal investigation of Swedish landslides, 1914-22.* In connection with its work, the Committee has tested a vast number of soil samples, extracted from more than 10 000 drill holes, at different depths below the surface. On account of the time (five years) that has elapsed since the Committee finished its work, some of the experimental methods have been superseded. Nevertheless, the work of the Committee could serve as a noteworthy example for the way in which a similar enterprise should be organized in the field of plain foundation engineering.

At present (1927), the data derived from previous experience in foundation engineering are of as little value as were those derived from Swedish landslide experiences prior to the work of the Geotechnical Committee. The information concerning soil character is inadequate, and the interpretation of the observed facts is very often arbitrary and inconsistent with the laws of physics and mechanics. Hence, the first requirement for improving conditions consists in standardizing the methods of soil classification, and, the second, in consistently applying present knowledge of soil mechanics to observations in the field.

* Statens Järnvägars Geotekniska Kommission, 1914-22, Stockholm, May, 1922.

DISCUSSION

Charles R. Gow,* M. Am. Soc. C. E.—The theories advanced in explanation of the behavior of soils under various conditions of use open up many new and interesting avenues of approach in the solution of foundation problems involving the predetermining of soil behavior under specific applications of loads. Undoubtedly, from the standpoint of definite and accepted standards of practice, the general status of foundation engineering is at present in the most unsatisfactory condition of any branch of the science with which engineers are called upon to deal.

There is no recognized authoritative source of information available to the foundation engineer by means of which he may act with the reasonable assurance that the requirements of his problem will be properly and efficiently met. Reliance must be had, in general, on observed experience with regard to previous construction undertakings in the same locality, and where such histories are lacking the designer must often proceed on a basis of uncertain assumptions, which may or may not ultimately prove to have been justified.

Heretofore, soils have usually been classified according to their visible physical properties, and engineers have acted on the supposition that these easily recognized characteristics furnished them with a sufficiently satisfactory guide by which to differentiate between soils and identify those possessing similar properties and behaviors.

The results now made available to the profession as a consequence of the author's research investigations throw much new light on the subject. The conclusions thus far would indicate that the essential factors affecting the behavior of soils under pressure are rather more obscure and complex in their nature than has hitherto been generally assumed.

Obviously, such factors as voids ratio, permeability, compressibility, and elasticity, cannot be determined by any superficial examination of soil samples. Intricate and somewhat elaborate laboratory experiments are necessary for securing this information. Yet, assuming the validity of the author's reasoning, these elements must be known before it is possible to predict, with any degree of accuracy, the capacity of a given soil to support a specified load.

The most encouraging feature of this paper is the evidence that the subject of soil mechanics is at last being studied in a scientific manner and that, as a consequence, there are now some positive experimental data which serve to explain many well recognized but heretofore little understood phenomena.

It is known, for example, that sands of similar appearance as to shape and size of grain may behave quite differently under equal conditions of loading. Likewise, certain clays appear to be highly compressible, whereas, other soils of identical appearance will sustain considerable loads without appreciable consolidation. Again, the driving of piles into a given soil may result in a pronounced upheaval at the surface level, while another apparently identical set of conditions produces little or no such action.

It is not suggested that the author's analysis presents a practical method of forecasting these variations in soil behavior; but it indicates the direc-

* Pres., The Gow Co., Inc., Boston, Mass.

tion of inquiry that should be followed in order to discover a satisfactory answer for apparent inconsistencies.

It is true that a determination of the essential factors that would make soil identification positive would probably involve more elaborate and expensive procedure than the ordinary engineer could be expected to command. Nevertheless, once the correctness of the author's theories have been established by a sufficient number of laboratory experiments on soils of known behavior, it should be possible, by means of further research, to calibrate these results against more simple rule-of-thumb tests that could be applied in the field. If a satisfactory method can be developed for accomplishing this result, engineers will have made the greatest single step in foundation engineering that has so far been taken.

The author's analysis of the relation between settlement and foundation area is extremely interesting and more or less in agreement with accepted beliefs; yet it probably will be unwise to assume too readily that such settlements can be reduced to common formulas until at least there has been more practical corroboration of the theory by actual experience.

The author's observations respecting the application of the customary pile-driving formulas are probably justified by the facts, since there are few engineers, having occasion to drive piles under varying conditions, who have not been impressed by the variation between actual and computed capacities. Fortunately, the so-called *Engineering News* formula has usually given results that are at least on the safe side, and until engineers become more enlightened respecting the factors that produce the observed departures from theory, they will probably do well to stick to that formula as the most dependable guide available.

The author does well to emphasize once again the fallacy of relying too strongly on the results of individual pile tests in determining the average pile load of the cluster. His analysis of the distribution of soil stresses, as well as his explanation of the variation between dynamic and static resistance, is especially interesting and enlightening.

Of unusual significance is the author's presentation of the compression and rebound curves (Fig. 14) developed by laboratory experiments on varying types of soil, particularly the comparison presented by similar experiments made on synthetic clays produced by mixing different percentages of powdered mica with fine sand. To those who have been seeking an explanation of the characteristic behavior of clays, as compared with sands, this presentation is especially interesting.

On the whole, it may be said that the author has, by means of this paper, added another important contribution to the clarification of the engineer's understanding respecting many of the most troublesome features of foundation engineering.

E. P. Goodrich,* M. Am. Soc. C. E.—About 1900, the speaker abandoned the use of the *Engineering News* formula. A demonstration of its ineffectiveness was had in connection with some work that was being supervised by a

* Cons. Engr., New York, N. Y.

certain governmental group. Certain requirements were laid down as to the supporting power of the piles, and the *Engineering News* formula was to be used. Those responsible were asked if they would be satisfied with a certain weight of hammer, dropping a certain number of feet. They said they would. The speaker then told them to apply the formula and they found that the result was a negative quantity.

Some data with reference to one of the questions that Professor Terzaghi has propounded, that is, a test of the distribution of stress under a square foundation under which the soil was rather cohesive, are as follows: Some years ago the speaker took a timber platform 4 ft. square, penetrated it by plungers located along a diameter and a diagonal and found that the diagonal stresses were larger than the diametrical ones. In other words, the corners of the square foundation supported more in proportion than any other part, either the center or the sides.

VICTOR A. ENDERSBY,* ASSOC. M. AM. SOC. C. E. (by letter).—The profession should feel indebted to Professor Terzaghi for the clear line he draws between soils to which the *Engineering News* formula is applicable, and those to which it is not. Among field engineers directly in charge of pile-driving, there are a great number who regard this formula as applicable, without modification or question, to every circumstance. There is an equal number who —perhaps more logically—refuse to believe in any formula whatsoever, and who regulate their pile-driving by guess, by the penetration they can force a contractor to attempt, or by the approach of five o'clock. Every practical engineer finds, sooner or later, that there is a great difference in the reliability of formulas in different soils, but there are not many who have any very clear idea as to general distinctions.

It seems very true that the most that can be hoped in regard to the science of foundations is that it become a reasonably reliable empirical system. One is especially impressed with this after such an experience as the writer had on one structure. Certain piles came to refusal in sand at a penetration which he considered unsatisfactory. A jet not being available at the time, he acted on a highly irrational "hunch" and suggested that the piles be left over night and an attempt be made to drive them next day. When this was done, the piles continued to drive with surprising ease, application of the *Engineering News* formula showing that as they stood they had lost 40% of their theoretical bearing power in a few hours. The excavation in which they were being driven was at the time 25 ft. below sea level, and 27 ft. below permanent water line, so that there could have been no question of a change in the texture of material due to subterranean water flows. There was evidently a decompression of some kind, but the occurrence is still a mystery to the writer, which he would like very much to have definitely explained. According to Professor Terzaghi's thesis, this might occur in some soils, but not in sand. The writer has heard a rumor of a similar occurrance on the Gila River in Arizona.

He has had occasion also to test the driving in adjacent and uniform material under three different hammers, a Union No. 2, double-acting, and Vul-

* Constr. Engr. of Bridges, Southern Section, State Div. of Highways, Los Angeles, Calif.

can Nos. 1 and 2, single-acting. The theoretical bearing power for piles of uniform size at uniform penetration with these three hammers was practically the same; the result was a good test for consistency of the formula as applied to various types of hammers.

George Paaswell,* M. Am. Soc. C. E. (by letter).—The paper, being synoptical in character, the author, naturally, omitted many vital points which bear on foundation practice as well as on the type of laboratory and field tests which supplied the data for the theory developed. The paper must be a great disappointment to engineers who look for a simple set of rule-of-thumb notes to embrace all foundation practice. It is no mean accomplishment of the author that he has demonstrated the fact that foundation design and construction is as exacting in analysis and as worthy of study as the design of the structures resting on these foundations.

The essential difference between superstructure and sub-surface design is that one may fit the materials to the design of the former; one must fit the design to the materials of the latter. The engineer cannot fit a foundation to a rigid set of formulas. The material must be studied in the site and the foundation must be fitted exactly to it. A code set of bearing values is more than worthless; it is dangerous. The author has illustrated this point very well. Settlement in a foundation has been viewed as an unfortunate and usually unavoidable concomitant of foundation construction; yet such settlement is as inevitable as the deflections and distortions of the supported structure. Provision must be made for them. That makes it necessary to formulate means to determine foundation distortions. The author has briefly indicated how such studies may be made.

The study of soils has indicated that two broad types exist: (1) Those in which the usual stresses of elastic solids are found, modified, of course, by proper elastic coefficients; and (2), those in which hydro-mechanical stresses are set up. Both types can be uniquely identified by sample tests for composition and loading, but neither can be safely described by grain size or moisture contents alone. The author's striking experiment on sand and mica mixtures has demonstrated fully the futility of mere sieve analysis as a basis of soil classification. He has put to rest also the colloid theory as the basis of coherent action in soils.

In his paper entitled "Transmission of Pressure Through Solids and Soils and the Related Engineering Phenomena,"† the writer attempted an analysis of stress distribution through soils and found that certain characteristic failure surfaces existed, similar in type to those found in elastic solids. It was also demonstrated that a uniform distribution of pressure under a load was a fiction of design. Numerous experiments preceding and following the paper have amply demonstrated the truth of this statement.

The huge subway construction program of Greater New York is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems and the writer hopes to see, in the course of the next few years, the crystallization of a definite soil mechanics and the complete abandonment of bearing value codification as a foundation formulation.

* Engr., Corson Constr. Corporation, Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

The author well points out that, given a complete soil identification, there is sufficient mathematical and physical science available to complete the practical investigation, of the foundation. It remains to codify soil identification.

The use of piles has been a blind foundation expedient, save where such piles were driven to absolute refusal. The anticipated capacity of a pile whether determined by formula or by actual field test has been of so indeterminate a character that most foundation failures may be laid to miscalculated pile-bearing values. It has been frequently emphasized by the writer that no adequate pile theory could be formulated until a rational soil mechanics had been developed, for the author has well demonstrated the fact that there is no separate theory for piles and for soil behavior. The development of the pile for underpinning buildings has evolved a better understanding of soil behavior than any other single phenomenon in engineering foundation practice. It is not a question of bearing capacity alone, but of actual load settlement behavior which determines the ultimate safety of the foundation underpinned. The study of the loaded soil and its distortions under the pile toe brought out the reasons for progressive settlement even if a pile had already been tested for a safe load, and there was evolved the ingenious "pre-test" pile, a scientific application of soil mechanics.

The writer is heartily in favor of the abandonment of the *Engineering News* formula for piles, and the adoption of one similar in nature to that suggested by the author, although he would prefer to disregard pile formulas entirely and make a separate study for each foundation type. That would eliminate the dangerous conditions resulting from group application to single pile tests or formulas.

There remains the extremely interesting subject of temporary earthwork supports, such as sheeting and bracing. In this work modern soil mechanics, has been given ample demonstration of its validity. The failure of earth banks is not a simple application of the Rankine or Coulomb special soil-type theory, but is one that can be easily formulated by a study of the elastic properties of the soil. The absolute limits of ground movement are predictable, and that gives a rational method for determining the maximum pressures that may be induced in the sheeting and timbering. This, of course, leads to a far safer and more economical layout of such bracing; but, on the other hand, it requires more study and more design than is usually given to such structures. The use of large excavating machinery, which requires large clearances because of working room, calls for a very careful investigation of load distribution through wales and braces. It must be borne in mind that a movement in the sheeting that permits movement of the earth behind it, also permits the grain re-arrangement of the soil and consequent increase in pressures as is well illustrated by the famous "dilatancy" experiment of Osborne-Reynolds.

C. C. Williams,* M. Am. Soc. C. E. (by letter).—The principles of soil mechanics presented in this paper should serve as a rational basis for correlating observed facts concerning foundation tests and behavior, and should thus contribute to making the design of foundations more scientific.

* Dean, Coll. of Eng., Univ. of Iowa, Iowa City, Iowa.

That the supporting capacity of soil is not, in general, directly proportional to the area of the foundation, has long been recognized, but the true relationship is not even yet entirely understood. In experiments on alluvial soil made in 1851-52 at New Orleans, La.,* in which areas varying from 0.25 sq. in. to 576 sq. in. were used, the penetration under a constant unit load increased practically with the area. Observations made in France† on alluvial soil indicated similar results. From these and other data,‡ the late Randell Hunt, M. Am. Soc. C. E., drew the conclusion: "Large areas of soft soil will not support as much weight per unit of surface as more limited areas of the same soil." Reflection on the manner of distribution of a local load through an indefinitely extensive solid or semi-solid, would indicate such to be the case.

The relation between the plastic and the elastic deformations of soil should first be recalled. The former is due to displacement of soil particles, whereas the latter is due to the resilient deformation of the particles, without slipping, relative to each other. Contrary to the behavior of steel, the plastic flow precedes the elastic, the latter constituting a small proportion of the total compression. Fig. 17, taken from a test made under the writer's direction by Mr. H. T. Heald, a graduate student, illustrates this relationship for loam and for sand. The experiment consisted of applying and releasing a load which was increased with each application and never entirely removed, the compression and the recovery being recorded at each application.

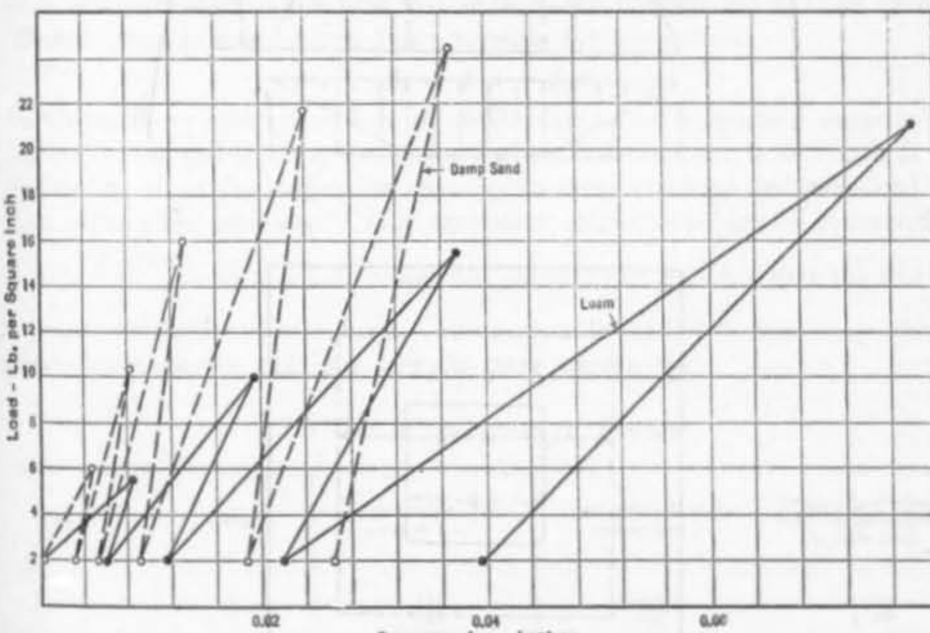


FIG. 17.—COMPARISON OF LOAD COMPRESSION CURVES.

If a load is applied over an area, A, at the surface, the inter-reactions of soil particles along the perimeter of this area are such as to spread the load laterally by virtue of shear over an ever-increasing area of bearing planes at

* *Van Nostrand's Magazine*, Vol. 27.

† *Annales des Ponts et Chaussées*, 1864.

‡ *Proceedings*, Assoc. Eng. Soc., 1888.

successive depths. The rate of this spread depends on the characteristics of the soil, being greatest for coarse fragmental materials, less for granular, and least for silt. Moreover, the spread is only approximately linear and the surface of uniform pressure is curved rather than plane, but the variation from the conception indicated in Fig. 18 is not sufficient to affect seriously the conclusions based thereon. Although the rate of spread is dependent on the soil characteristics, the depth of soil affected, and hence the total spread, depends on the depression of the soil under the load. This deformation increases until the spread distributes the load over an area sufficient to bring the intensity within the elastic resistance of the soil. When a homogeneous soil is assumed, an expression for the total deformation may be obtained as follows:

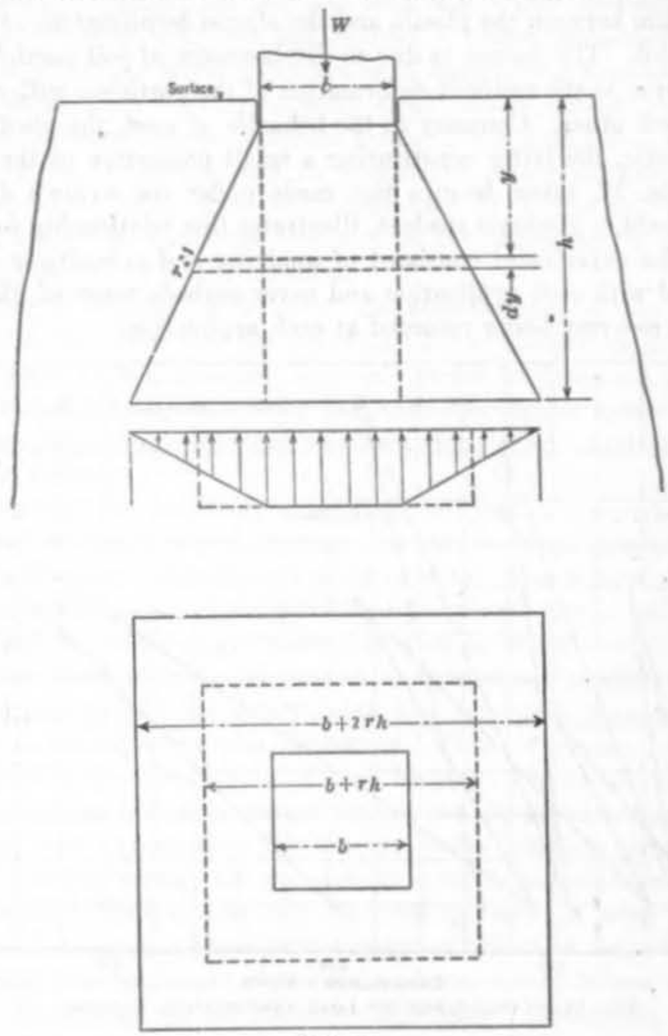


FIG. 18.

Let W be the load applied; h, the total depth of soil compressed; y, the depth to any plane considered; r, the tangent of the angle of spread; and C, the modulus of compressibility. The distribution of pressure on any horizontal plane may be assumed equivalent to a uniform intensity over a width of $b + r y$. Then, for a square surface with side, b, the deformation of an elementary horizontal prism bounded by the sides of the frustum of the pyramid is $\frac{W\,d\,y}{C\,(b + r\,y)^2}$, and the total deformation in a depth, h, is $\frac{W}{C}\int_0^h \frac{d\,y}{(b + r\,y)^2}$, which when integrated gives $\frac{W\,h}{C\,(b^2 + b\,r\,h)}$. This expression shows that the deformation varies inversely with the perimeter, b, as well as with the area, b^2.

If w is the pressure intensity at the surface and p the intensity that the soil will sustain elastically without deformation (at a given depression), then, $w\,b^2 = p\,(b + r\,h)^2$, whence $h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$, neglecting the weight of the soil itself. If, as for sand under certain conditions, $r = \frac{1}{2}$, and $h = 2$, the deformation would be $\frac{2\,W}{C\,(b^2 + b)}$. In this case the denominator contains simply the sum of the bearing area and the perimeter multiplied by a strip $\frac{1}{2}$ ft. wide.

Thus, an elementary analysis indicates that the supporting capacity of soil depends on two factors: (1) The bearing area; and (2) the perimeter of the bearing surface. However, on account of the variableness of soils, it is impossible to assign values that will be universally applicable to these factors. The relationship may be stated as a formula for convenience,

$$W = A\,p + L\,f$$

in which, W = load; A = area of application; p = supporting capacity per square foot of soil in compression for a given deformation; L = length of the perimeter; and f the supporting capacity in shear (friction and cohesion) per foot around the perimeter. This expression might be otherwise conceived as $\left(A + \frac{L\,h\,r}{2}\right)p$, since p is in reality the intensity of pressure that the soil will sustain within its elastic limit. However, although this form may clarify the conception, the previous form is more convenient.

TABLE 2.—Supporting Factors.

Sand.	Clay.	p, in pounds per square foot.	f, in pounds per linear foot.	Total superimposed load, in pounds per square foot.
0	1	110	400	1 710
1	1	120	280	1 240
5	1	500	160	1 140

These supporting factors, p and f, vary with the character of the soil (mineral and granulometric composition, compactness, water content, etc.), and with the depth to which the load penetrates the soil. The two terms are indeterminate, their relative values depending on which represents the more rigid element in the total support. For the more rigid soils, the direct compressive resistance becomes the greater, for any considerable compression. Thus, the tests made by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., quoted by the author, give, approximately, the results shown in Table 2 for 0.1-in. penetration and for the various mixes of sand and clay.

At 0.05-in. penetration and less, the marginal shear, in these experiments, carried nearly all the load. Inasmuch as Mr. Goldbeck worked with small compressions, these facts explain the apparent importance of the diameter, $(\sqrt{A})$.

Tests (see Fig. 19) made on a uniform clay soil containing 7% sand in undisturbed condition, using bearing areas of 1.0, 2.25, and 4.0 sq. ft., showed for 0.3-in. depression, $p = 3\,500$ and $f = 1\,500$. For depressions of less than 0.1 in., the perimeter factor carried nearly all the load. In still another test in clay soil, $p = 3\,400$ and $f = 2\,300$ for a settlement of $\frac{3}{8}$ in. While the Austrian experiments quoted by the author are rather fragmentary, the corresponding values at 1.0-in. settlement are approximately, $p = 2\,800$, and $f = 700$.

Of these two elements, p varies almost directly with the settlement in sand and dry clay soils, while, on the other hand, f remains fairly constant, varying, if at all, with some low power of the depth of settlement, in some cases about as the fourth root.

If these elements were determined, the bearing capacity of a larger area could be predicted from a test on a small area. Thus, based on the test on 7% sand soil mentioned previously, using the data from the 0.3-in. penetration, $p = 3\,600$ and $f = 1\,500$, a footing 8 ft. square might be expected to sustain (with 0.5-in. penetration),

$$3\,600 \times \frac{5}{3} \times 64 + 1\,500 \sqrt[4]{\frac{5}{3}} \times 32 = 432\,000 \text{ lb.}$$

or, approximately 5 400 lb. per sq. ft. of superimposed load. The broken-line curves of Fig. 19 show calculated bearings assuming p to vary as the depth, and f as the fourth root.

The correlation between calculated and observed data is high within the ultimate bearing capacity of the soil. The writer has applied this theory to the other test data available, with similar results.

The most reliable practical basis for comparing the supporting capacity of two areas on the same soil, therefore, is the factor, $A + K \times L$, in which, K is a soil coefficient. If, as for a certain sand, K is $\frac{1}{4}$, then a 36-sq. ft. foundation can be expected to sustain on this soil, $\frac{36 + 6}{1 + 1}$, or 21 times the test load on a post 1 ft. square with the same settlement, instead of 36 times as much according to the usual assumption, or instead of 6 times as much, which would be the case if the supporting capacity varied with the diameter for a given penetration.

While for small settlements and for small test areas, observed data, such as those of Mr. Goldbeck's experiments, indicate settlement, under a constant

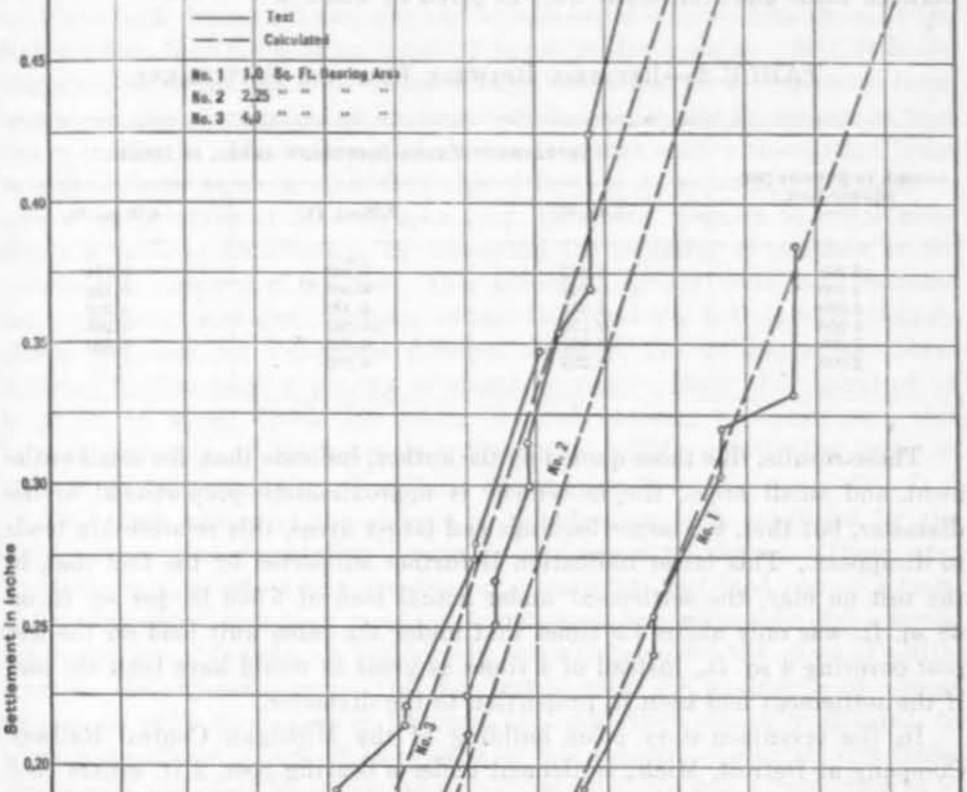

Test
Calculated
No. 1 1.0 Sq. Ft. Bearing Area
No. 2 2.25 " " " "
No. 3 4.0 " " " "
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
Settlement in Inches
No. 2
No. 1
No. 3
2
3
4
5
6
7
8
9
10
11
12
Load in Thousand Pounds per Sq. Ft.

FIG. 19.

unit loading, to vary with the diameter of the bearing area, the writer believes that a general conclusion to this effect is unwarranted, even for soils having great cohesion. In the tests on the 7% sand-clay soil mentioned, the relations between loads and settlement were as given in Table 3.

TABLE 3.—RELATION BETWEEN LOAD AND SETTLEMENT.

Load, in pounds per square foot.	SETTLEMENT UNDER DIFFERENT AREAS, IN INCHES.		
	1 sq. ft.	2.25 sq. ft.	4.00 sq. ft.
3 000	0.032	0.042	0.112
4 000	0.054	0.082	0.155
5 000	0.075	0.132	0.210
6 000	0.110	0.205	0.265
7 000	0.152	0.325	0.340
8 000	0.225	0.385	.....

These results, like those quoted by the author, indicate that, for small settlement and small areas, the movement is approximately proportional to the diameter, but that, for larger loadings and larger areas, this relationship tends to disappear. This latter indication is further supported by the fact that, in the test on clay, the settlement under actual load of 5 500 lb. per sq. ft. on 68 sq. ft. was only about 1.5 times that under the same unit load on the test post covering 4 sq. ft., instead of 4 times as great as would have been the case if the settlement had been in proportion to the diameter.

In the seventeen-story office building of the Michigan Central Railway Company at Detroit, Mich., settlement under a bearing post, 2 ft. square (not back-filled) was 0.02 ft. for 5 000 lb. per sq. ft., but under the mat foundation of the building loaded to 4 000 lb. per sq. ft. there was no appreciable settlement.* Indeed, numerous instances might be cited where an actual load on large footings amounts to one-half or even a larger fraction of a test load on a small area, and yet the settlement is not noticeably greater than that with the test load.

Where care is exercised to load the soil around the test footing, the discrepancy between the settlement under test and actual load on the structure is reduced. Unfortunately, too few correlated observations have been made on the settlement of the completed structure, and that of the test load at similar intensities of pressure. It is the writer's opinion, however, based on the rather meager data available, both theoretical and observational, that, for areas of practical size, the settlement does not vary greatly for comparable sized footings carrying a uniform intensity of pressure on the same soil.

If, as seems to be the case from this discussion, the bearing capacity varies with both the area and the perimeter of the footing, then, for larger foundations, the bearing for constant settlement varies nearly with the area. For instance, with two footings on sand, as previously assumed, one 10 ft. square

* *Engineering News*, September 3, and October 1, 1914.

and one 20 ft. square, the respective bearing capacities would be about as $\frac{100 + 10}{400 + 20} = 1:3.8$, instead of $1:4$ when based on the area directly, or $1:2$ if based on the diameter relationship.

While both theoretical analyses and experimental observations show, as the author states, that the bearing intensity is not uniform under a pier, it is the writer's opinion that the error in the moment calculated for a continuous beam or mat, on the assumption of uniform distribution, is not as serious as that shown in Fig. 3. The author neglects the fact that such a foundation beam or slab deflects upward, and that this deflection is generally sufficient to relieve the pressure at the mid-span, and, therefore, operates to effect more nearly a uniform distribution, by increasing the intensity of pressure at the (downward) supports of the slab. This deflection upward operates to decrease the pressure at mid-span, because of the fact that the soil is only slightly elastic and does not follow the deflected member, the loading of the beam differing therein from a gravity or constant-pressure load. For spans of 16 to 20 ft., or more, which the writer has had occasion to investigate, this upward deflection of the beam is sufficient to nullify greatly the error in moment on the assumption of uniform distribution.

Two other phases of the science of foundations, both suggested by the author, enter into foundation design to an important degree. These are: First, the relative effect on settlement of a constant dead load and of a transitory conjectural live load to which foundations are subjected; and, second, the prediction of foundation behavior in order to calculate the effect of settlement on the superstructure. One has but to visit the basements of almost any of the older structures in Chicago, Ill., and cities similarly situated, to witness the effects of lack of knowledge on this subject. Eccentric live loads, due to wind and other elements, are usually unavoidable, and a prediction as to the behavior of the foundation under such conditions and the consequent effect on the superstructure is necessary in order to prevent distortion, if not failure, of the structure. To secure uniform minimum settlement under a structure may be desirable, just as rock foundations may be desirable; but, since such foundations are not always obtainable, the problem then resolves itself into designing the structure for actual foundation conditions. Therefore, studies of soil behavior should include more detailed information than the usual observations accompanying test loading.

HERBERT CHATLEY,* ESQ. (by letter).—Whether or not some of the finer points of Professor Terzaghi's hypotheses will stand prolonged investigation is a matter for the future to decide, but no one can question that his masterly exposition of the manner in which the flow of water through fine-grained soils is related to their bearing capacity has thrown a brilliant light on what had previously been a most misunderstood subject.

The conditions in Shanghai, China, are those of a saturated soft alluvium of indefinite depth, and the major conclusions drawn by Professor Terzaghi as to the behavior of surface loads and piling under such conditions are of the greatest interest and value to engineers there. Through the initiative

* Acting Engr.-in-Chf., Whangpoo Conservancy Board, Shanghai, China.

of A. V. H. von Heidenstam, M. Am. Soc. C. E., pile tests on rather a large scale have been made for some years in Shanghai by the staff of the Whangpoo Conservancy Board. Professor Terzaghi has been good enough to speak very appreciatively of these tests. Broadly speaking, they have confirmed his views as to piling in soft soils.

The slowness with which fine alluvium expels its contained water is well shown in Shanghai by the behavior of hydraulic fills formed by the pumping up of dredge spoil. Where these fills are above the ground-water level they do become dry, but below the top 2 or 3 ft. some years may elapse before the deeper material becomes fully consolidated. This is true in spite of the fact that the hydraulic filling process tends to remove an appreciable part of the colloid content.

As to the use of piling formulas, the cogency of the reasoning as to the variation of the hydro-dynamic resistance with the duration of the applied force is complete. The new "derivation" of the *Engineering News* formula is most interesting and calls attention to the fact that the assumed constancy of the second term in the denominator of that formula is even more arbitrary than is generally supposed.

In the work on the internal friction of granular masses the author has seen fit to adopt the writer's conclusion* as to the importance of single molecular bonds in uniting particles the weight of which is of the same order as those bonds. The writer has since gone much further into the question, and these studies fully sustain his earlier conclusions as to the magnitude and importance of these forces in granular masses.

In regard to the classification of soils the new data as to the effect of mica are most important. There is one point to which attention might very well be directed and that is the possibilities of change of grain size by the permanent aggregation of particles. Putting it very crudely, one might imagine that, under certain circumstances, a fine clay could gradually be transformed into a loam by the adhesion of the particles.

Engineers would be very glad if the author would give some practical rules for the design of sheet-piling, which acts as a retaining wall.

J. Albert Holmes,† M. Am. Soc. C. E.—Although this paper relates particularly to foundations, it has, together with the author's previous investigations, a bearing on another engineering activity, the investigation, design, and construction of earth dams.

While earth dams of moderate height, built 2 000 years ago and more, are still impounding water and have continued to do so for the greater part of their existence, it is only within the memory of most of the older engineers and within the lifetime of many of the younger ones, that modern engineering thought has been directed toward the problems involved. One of these problems, and the principal one, is the selection and proper placing of materials in a structure. Theory and experience have demonstrated that pressures, tending to disruption, exist in the materials being placed in an earthen dam.

* "On Cohesion", *Philosophical Magazine*, Vol. XL, August, 1920, and "Silt", *Minutes of Proceedings*, Inst. C. E., Vol. CCXII, p. 400.

† Hydr. Engr., Pearse, Greeley & Hansen, Chicago, Ill.

To control this tendency a thorough knowledge of the medium used is as essential as in other engineering works. In investigating and studying the character of materials, permeability or ability to resist the flow of water through them, must be determined.

Several factors enter into the phenomena of flow through soils, as actual size of grain, relative sizes of grain, compactness, and colloidal content. A measure of relative size is indicated by the uniformity coefficient, which means a range of sizes such as will produce a dense mass resulting in small passageways and low percentage of voids. In their natural condition (and it is in this condition that use is made of them), soils have a tremendous range in the size of particles. Writers on the subject of colloids have maintained until quite recently that soils contain not more than 2% of material in a colloidal state. To account for the capacity of soils to absorb dyes and gases, it was assumed that they contained absorptive minerals known as zeolites. Upon investigation no zeolite minerals could be identified in soils. Instead it was found that soils contain from 6 to 70% of colloidal material, in which lies their entire absorptive ability. The inorganic colloidal material in soils is chiefly made up of the products of chemical weathering and decomposition of soil-forming minerals, together with organic matter and perhaps some soil minerals of colloidal size.

An average diameter of 1 micron is arbitrarily fixed by the U. S. Department of Agriculture as the upper limit of colloidal size. At and below this dimension the particles cease to be affected by gravity and are subject to Brownian movement when in suspension in a liquid. Also, at the dividing line of 1 micron between colloidal and non-colloidal particles, microscopical control is good, while, with less diameter, points of emanating light might be mistaken for the particle itself. In solution, those particles that diffuse slowly and, after being deposited, are evaporated and separate into a shapeless jelly, are of one class or group. Those that diffuse rapidly and, under the same process, crystallize, are of a second group. Because of their resemblance to glue the substances of the first group are called colloids, from the Greek word, "Kolla", and for the same reason the second group is called crystalloid. Colloidal material occurs in soils in the gel condition.

Professor W. D. Bancroft, of Cornell University, states that colloidal chemistry is the chemistry of every-day life. It certainly touches engineering frequently and at many points: Pavements; water supply; sewage disposal; foundations; concrete and water-proofing of concrete; earth dams; and agriculture.

The U. S. Department of Agriculture, in its Bureau of Soils* and Bureau of Roads, at Washington, has been investigating the subject of colloids for some time, as has the Society's Special Committee on the Bearing Value of Soils for Foundations, etc. The Department of Agriculture is interested in the subject because it bears on the fertility of soils. The retention of moisture and the chemical action of fertilizers are dependent on the quantity and character of colloidal matter present in soils. The colloidal condition of soils enters very materially into the problem of dam construction.

* *Bulletins 1122, 1193, 1311, 1452,* U. S. Dept. of Agriculture.

One of the recent accomplishments of the U. S. Department of Agriculture in its Bureau of Soils was to determine the chemical composition of forty-five samples of soils and subsoils, collected from widely distributed points in the United States and representing many types. Peats and mucks were not included. The colloidal material was extracted from these samples and the chemical composition of both soils and colloids determined. It was found that soils and extracted colloids alike contained the following simple chemical compounds and elements, but in varying quantities:

Silica	SiO_2	Soda	Na_2O
Titanium	TiO_2	Phosphoric acid	P_2O_5
Alumina	Al_2O_3	Sulfur	SO_3
Ferric oxide	Fe_2O_3	Chlorine	Cl
Manganese	MnO	Water	H_2O
Lime	CaO	Nitrogen	N
Magnesia	MgO	Organic matter	....
Potash	K_2O		

Carbon dioxide, CO_2, was obtained from three soil samples and one sample of colloid.

Silica is the principal constituent of soils, and it was found that the colloidal matter is composed mainly of silica, alumina, iron oxide, and combined water, the percentage varying in the samples as follows:

	In Soils.	In Extracted Colloids.
Silica	51.32 to 93.66	31.84 to 55.44
Alumina	2.57 to 22.92	16.42 to 38.28
Iron oxide	0.93 to 13.82	4.66 to 16.67
Combined water	....	3.33 to 16.56

The colloids in the soils ranged from 6.2 to 57.8 per cent.

The colloidal material differs chemically from the larger soil particles, but separate and distinct compounds were not identified. The colloidal mixtures are very intimate and are without tendency to break up into known compounds. Table 4 illustrates the differences in composition of colloidal matter and whole soil and the coarser particles of soil.

TABLE 4.—AVERAGE COMPOSITIONS OF SOILS, COLLOIDS, AND COARSER MINERALS COMPARED AS PERCENTAGES.

	SiO_2.	Al_2O_3.	Fe_2O_3.	CaO.	MgO.	K_2O.	Na_2O.	Combined H_2O.
Colloidal matter from 45 samples	43.34	26.84	10.41	1.04	1.72	1.43	0.38	9.93
Coarser mineral particles from 35 of the 45 samples	87.2	6.0	1.9	0.5	0.5	1.5	0.9	0.6
Mineral particles in fine sands and silt of a different series of 26 soils	83.3	7.3	1.5	2.0	1.7	2.9	0.9	0.4

The nature of soil colloids is destroyed by heat. Two samples, one of colloidal or ultra-clay, the other of clay soil, made into a number of test pellets, were subjected to carefully controlled heat. At desired temperatures a part was removed and the temperature was stepped up on the remainder. As the pellets were removed from the furnace, ammonia absorption determinations were made, with the following results:

Colloid Sample:

Temperature, in degrees centigrade	110	265	374	559	754	1130
Cubic centimeter of NH_3 absorbed per cubic centimeter of colloid.......	110.3	100.8	80.0	74.1	57.5	2.2

Soil Sample:

Temperature, in degrees centigrade	110	190	265	374	522	673	844	1130
Cubic centimeter of NH_3 absorbed per cubic centimeter of soil........	27.7	25.3	24.8	19.7	14.9	13.6	7.4	1.4

The colloids were progressively destroyed, and under the high temperatures quite completely destroyed.

Extracting the colloids from a soil to determine the amount of colloidal material is a long and laborious process. A more rapid method, checked by the extraction process, is the use of dyes, gases, and water vapor.

It was found that practically all the colloids in a soil are associated with the finer particles, and very few or none with the coarser parts. Certain ratios were found between component parts of the colloids which gave a correlation. This correlation, associated with rainfall, temperature, and color, has been presented as an index of the soils for agricultural purposes.

The speaker studied the data presented by the experiments with the object of finding a relation that might be used in the selection of soils for dam building. The most satisfactory is that existing between the colloidal content and loss by ignition. The process of incinerating samples of soil to determine their "organic" content is not a new idea, but a knowledge of the colloidal content, carefully determined, of a series of soil samples that have been incinerated, is new.

The relation between colloidal content and ignition loss is shown in Fig. 20, which indicates the determinations from thirty-five samples. The diagram is offered as a means of determining approximately the amount of colloidal material in soils proposed to be used in dam construction. The incineration of a soil can be accurately and quickly done. The extraction of colloidal material from the thirty-five soil samples was almost complete. It may be seen by the diagram that practically all the samples range between 2 to 8% ignition loss and 5 to 35% colloidal content.

The process of sluicing earth by the semi-hydraulic or the hydraulic method, would probably separate less than one-half the amount of colloid that is removed from a soil by laboratory methods; that is, an ignition loss of 5% would not mean that 25% of the colloids could be separated by sluicing, although approximately that amount is in the soil. All of it would be sluiced, with the finer particles, into the core where, by its gel-like condition and by swelling, it would fill the voids between the particles. It is as undesirable to have too little colloidal material in a dam as too much. Both these extremes should be avoided by the selection of a material containing the right amount of colloids. The colloidal content of core material being placed in a dam may be ascertained by incineration with the assurance that the result is reasonably correct.

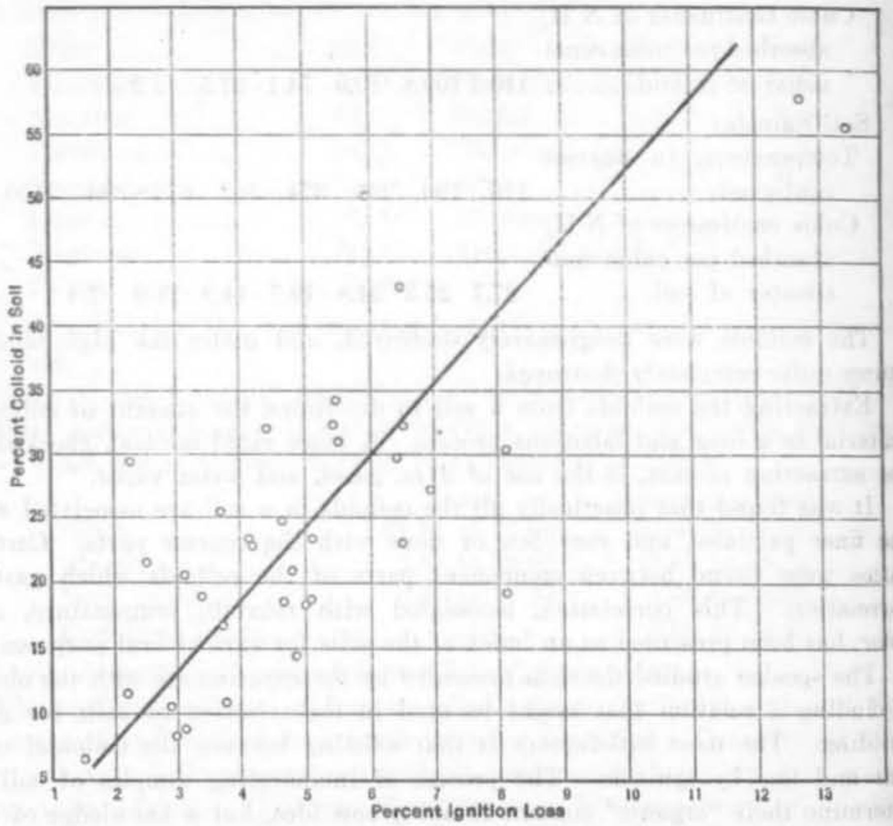


FIG. 20.—RELATION BETWEEN IGNITION LOSS AND COLLOIDAL CONTENT OF SOILS.

No permeability tests were made by the U. S. Department of Agriculture on the soils investigated for colloidal content and chemical composition. The speaker has compiled data, from various sources, on the relation between ignition loss and permeability. These data are given in Table 5, in Columns (5) and (6) of which it may be seen that the percentage of ignition loss increases in reasonable regular order as the permeability rate decreases.

Having this knowledge and having determined the colloidal content of a soil as indicated by Fig. 20, or by a more precise method, a solution of the problem of selecting a proper material for an earth dam is made more possible. Effective size and the uniformity coefficient of a fine-grained soil have no relation to its permeability rate, although a soil having a high uniformity coefficient insures small void spaces and low rates of flow.

A gramme of colloid, containing particles having an average diameter of 91 millimicrons, has a surface area of particles amounting to 24.2 sq. m. Because of this enormous area the surface forces developed are sufficient to make the colloid a stronger binding agent than Portland cement. This is true only when the material is dry. The experiment cited was not carried beyond a 10% content of the binding agent, although it was sufficient to show that colloidal material is the principal binding agent of soils, giving them cohesiveness according to the moisture content.

TABLE 5.—DATA SHOWING THE RELATION BETWEEN THE COLLOIDAL CONTENT (IGNITION LOSS), AND MAXIMUM RATE OF SEEPAGE THROUGH SOILS.

No.	Material.	Effective size,* in millimeters.	Uniformity coefficient.†	Ignition loss, percentage.	Maximum rate.‡
(1)	(2)	(3)	(4)	(5)	(6)
1*a*	Very fine sand	......	......	0.00	514 000
1*b*	Very fine sand	......	......	0.00	387 000
2	Very fine sand	0.023	2.4	1.36	375 000
3	Very fine sand	0.02	2.0	1.54	256 100
4	Upland soil	0.054	5.1	4.37	26 840
5	Subsoil	......	......	3.32	21 350
6	Fine soil, sandy, roots	0.026	9.3	4.52	21 200
7*a*	Top-soil and subsoil	......	......	3.67	15 300
8	Sandy soil	0.091	2.9	3.97	15 000
9	Fine sand	......	......	4.12	11 700
10	Top-soil, roots in	......	......	5.74	10 800
11	Top-soil, roots out	......	......	4.80	10 000
7*b*	Top-soil and subsoil	......	......	3.67	8 600
12	Gravelly subsoil	......	......	3.01	6 300
13	Fine sandy soil	0.04	5.9	6.79	3 550
14	Subsoil and sand	......	......	3.46	3 200
15	Top-soil and subsoil	......	......	5.21	2 200
16	Top-soil and subsoil	......	......	7.00	1 500

* "Effective size" is the size of grain in materials than which one-tenth of the sample is finer and nine-tenths coarser. The finer one-tenth controls the seepage rate.

† "Uniformity coefficient" is the relation, expressed as a ratio, between the grain size which has 60% of the sample finer than itself and that which has 10% finer than itself.

‡ "Maximum rate" is the quantity of water, in gallons per acre per day, that will seep through a material, with the loss of head equal to the depth of the material.

In a homogeneous structure, either compacted in layers or consolidated by irrigation, the function of water is to soften the colloids in the dry, hard soil, breaking it up and permitting its particles to re-arrange themselves in a more compact form and by the continued presence of water, in order to keep the colloidal material expanded in the voids of the soil. Colloids have capacities for swelling of from 40 to 150% and a water-holding capacity of 50 to 140 per cent.

Investigation of the permeability rate, combined with a determination of colloidal content and ignition loss, is very much needed. In making a mechanical analysis to determine the uniformity coefficient the "clay" fraction, diameter 0.005 to 0 mm., roughly checks the colloidal content.

The speaker offers the foregoing for consideration and suggests a line of research in soils, physical and chemical as well as mechanical, that will supplement the data given.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).—In 1727, Couplet published his monumental work on soil pressures, giving a "complete theoretical analysis of soil physics," and, now, 200 years later, Professor Terzaghi

* Cons. Engr., New York, N. Y.

writes that the science of foundations is of the present and future. From this it must be assumed, that the science of foundations has no past, and there is no possibility of criticizing that statement if the rigid definition of a science is insisted upon. Still, it is difficult to slight such announcements as appear in the *Annales* of the French Academy of Sciences (January 22, 1783) to the effect that the problem of determining the lateral earth pressure was susceptible of a rigorous solution for any special case by the application of the theory outlined on that date by Chauvelot.

The design of foundations has never been based on a true science of foundations, and, to disagree with the author, it is not even now (in 1928) based on any scientific principles. There are, in common use, certain methods on which various types of foundation design are based. Such methods are adaptations of principles of mechanics, taken either from statics or pneumatics, with or without correction factors.

It cannot be expected that the old, although not accurate, methods of design should be discarded in favor of principles from a "science of foundations" when such science does not exist in the common knowledge of the day. As the author states, there is insufficient data available from which even empirical rules may be deduced, and so little is known of the materials which are encountered (soils and rocks) that no skeleton or shelving of general data and hypothesis can be formed as a basis for the accumulation of missing information. Until such a framework can be formed, no one can announce even the birth of a science of foundations.

The present methods for the design of foundations are open to many more criticisms than are enumerated by the author. However, these methods have been used for the preparation of plans for many foundations which, for the large percentage of cases, have been successful. Note that this statement uses the expression "preparation of plans" and not "design". The success of a plan is measured by the result obtained in supporting a superstructure without apparent or measurable danger. The economy of a foundation plan is quite another matter.

It is quite difficult to clear away all this accumulation of precedent and "experience" and suddenly substitute a new "science of foundations." Certainly, it is not a one-man job and no one-man's work can be expected to supply all the necessary frame upon which such a science can be built. The profession should be very thankful to the author for his numerous reports of the past few years, all pointing to the desired goal. It is high time that the Society has produced some constructive results in providing a basis for the development of this much needed science.

The greater part of the work in the subject of soils during the last three centuries has been in the development of the art of foundation construction rather than in the preparation of data for the scientific development of foundation design. The method of shoring or underpinning a structure is far more advanced than the design of the foundation which may be placed under such structure. The chief drawback to the lack of scientific development has been the absence of any authoritative nomenclature or definite standardization of terms. The tendency has been to copy expressions from the mechanics of solids, although it has been many years since soil mechanics was separated from hydrodynamics and from the statics of solids. Some writers have gone far in the opposite direction, coining new expressions which, often after translation, are crude and misinterpreted or entirely incomprehensible to the reader.

The relation between the size and carrying capacity of a loaded area can never be rigidly fixed by any usable expression. The author deals only with the individual case, corresponding to the case of a single pile, and has omitted entirely the discussion of the effect of loaded areas in close proximity on each other. Even in the individual case, the author has omitted a very important factor, namely, the shape of the loaded area. In the bearing of a solid against a solid, within the elastic limits, the usual assumption that the shape of the loaded area has no effect, probably brings in no appreciable error. The study of loaded plates of various shapes leads one to doubt the accuracy of the assumption. In a perfectly elastic material, with no cohesion and no surface tension, the shape of a loaded area can have no effect. However, in a substance like a clay, there is a resistance to settlement under loads (bearing power, so-called), which is a function of the area and also a function of the perimeter. The latter term is now being disregarded. Some of the author's conclusions should be modified to suit this perimetral resistance. Two circular areas of diameters in the ratio 1:2, loaded equally per unit area, have equal resultant resistances as a function of the area, but the smaller has twice the length of edge per unit area. The smaller area will settle the least.

An equally important problem is the determination of the effect of one loaded area upon adjacent areas. The complexity of such a usual problem as the design of spread footings for the columns of a frame building with its interior and exterior footings as well as square and non-square loaded areas, is appalling when one thinks of the speed with which plans for such footings are being prepared daily. Many instances are known where unequal settlement is very evident. Unfortunately, few such cases are on record.

Still another disregarded factor is the amount and nature of cover (backfill) above the bearing area of a footing. In a few isolated ideal cases, theoretical methods seem to point to the true values of the side frictional support of piers or caissons, based on the formulas of Coulomb and others, centuries old.*

Edge resistance and side-frictional resistance tend to equalize the distribution of resulting carrying capacity per unit base area. The true state of affairs is much closer to the uniform than to the parabolic distribution of the soil reaction. The diagrams in Fig. 3 are an indirect proof of the last statement; for, if the parabolic distribution results in bending moments in continuous footings of more than twice the calculated moment (based on a uniform distribution), it is inconceivable why at least one-half such footings built have not failed. The "factor of safety" of a structure, based on good

* *Engineering News-Record*, Vol. 88, 1922, p. 1052.

concrete and a reduced load-summation design and built with concrete "not quite as good as specified", is certainly in the neighborhood of two.

The writer would like to bring up a problem which has interested him in connection with a recent arbitration in which he was involved. If one of two exactly similar six-story buildings is removed, and replaced by a taller building with footings placed at exactly the same depth as had previously existed, but with greater load intensity, should there be expected a settlement of the adjacent building, which has not been disturbed? Is such a design proper, or should some means be devised to prevent the old building from settling, even if the new footings are not built to any level lower than the sub-grade of the adjoining building? The problem has also some interesting legal aspects. The writer might mention his argument that, even if the new footings are built no lower than the sub-grade of the old ones, their necessary settlement on receiving load will cause them to take a position sufficiently lower to cause a "sympathetic" settlement of the adjoining footings.

The subject of piles for foundation purposes is very ably treated by the author. The objection to the use of the recognized pile-driving formula is a special case of the general objection made by the Belgian scientist, Boussinesq, that all soil pressure designs are fundamentally problems in statics and the experimental work on which such designs are based, is dynamic in method.

As to the basis for a soil classification, the writer* is still of the opinion that:

"* * * the classification of soils and other granular materials should be along the lines of strain characteristics rather than those of stress, namely, elastic strain, plasticity, and fluidity, for these phenomena are evident and measurable, and embrace within them the results of all the stresses which may act. Generally speaking, the elasticity factor will take care of the solid, the fluidity of the fluid, and the plasticity of the colloid properties of the material being classified."

W. S. Housel,† Assoc. M. Am. Soc. C. E. (by letter).—Without doubt, it is a source of satisfaction to many to see a revival of interest in the problem of the bearing value of soils. A number of men, including the author, have devoted a great deal of time and effort to a better understanding of foundation problems from the standpoint of soil conditions.

During 1927, the writer, acting for the Board of Wayne County Road Commissioners at Detroit, Mich., has been engaged in conducting a most extensive series of tests to determine the bearing value of soils. The work has been done in the field, under conditions governing actual construction, and, at the same time, every effort has been made to keep the accuracy of the data comparable to what might be obtained with a similar set-up in the laboratory.

More than fifty tests have been made on different sizes and shapes of bearing areas, some tests on piles, and some on combinations of bearing plates and piles. Measurements of lateral pressure and a study of the general phenomena of the transmission of pressure in soils constitute an important part of the

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1567.

† Instr. in Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

investigation. The laboratory work is being conducted by the Civil Engineering Department of the University of Michigan. Through the courtesy of the Michigan State Highway Department, the facilities of the State Highway Laboratory have been made available for this work. With the field results of this investigation at hand a few comments on the principles of soil action as outlined by Professor Terzaghi may be of interest.

The most consistent results of all the tests made by the writer indicate that there are two factors of strength by virtue of which cohesive soils, such as clay, are capable of supporting loads. One of these factors is the shearing resistance on the perimeter of the bearing area; the other is the resistance of the soil to compression, due to a condition of strain set up in the soil around the bearing area. This last phenomenon has been called a pressure bulb.

During the progress of a test it is noticeable that the first failure taking place is failure in shear along the perimeter of the bearing area. In the latter part of the test there is a noticeable upheaval of soil surrounding the bearing area which indicates that a condition of strain exists. That this upheaval cannot be classified merely as flow of soil from beneath the bearing plate is shown by the fact that for some time after the upheaval is first noted the bearing plate continues to support the increased load applied with relatively small settlements. This action continues with increased loads up to a rather definite point of failure, at which point the soil flows from beneath the bearing plate with apparently no restraint. This marks the point at which the pressure bulb fails.

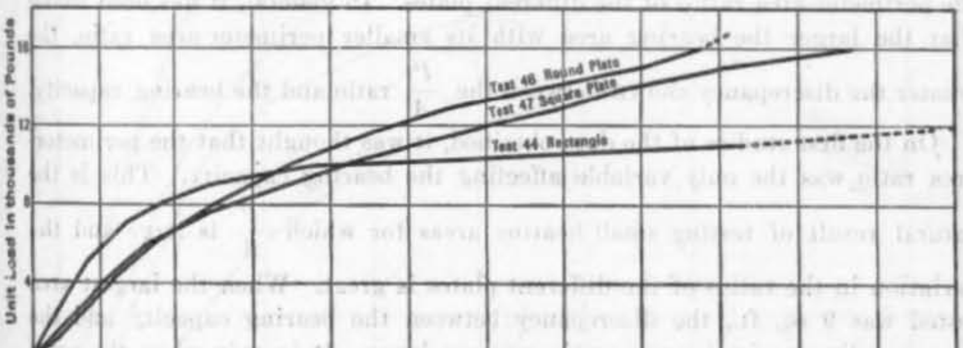


FIG. 21.—COMPARISON OF DIFFERENT SHAPED PLATES OF SAME AREA, 4 SQUARE FEET.

The load deflection diagrams obtained by the different tests bear out these general observations. Fig. 21 shows a set of typical load deflection diagrams, in this case for three different bearing plates, each with an area of 4 sq. ft. As shown, one plate was round, one square, and the third rectangular, with a ratio of width to length of 1 to 7. The round plate has a perimeter-area ratio, $\frac{P}{A}$, of 1.78; the square plate, of 2.0; and the rectangular plate, of 3.04. The material is stiff yellow clay and the size of the test pit is the same as the bearing plate, 4 sq. ft.

It is significant that, in the first part of the tests, the range of bearing capacity in which shear failure about the perimeter is taking place, the relation between the bearing capacity of the several plates is similar to the relation between the values of x for the different plates. The bearing capacities are not in exact proportion to x because, even in their lower ranges, the pressure bulb is beginning to have some effect as a factor of strength.

In the latter part of the test, in the range of bearing capacities where the pressure bulb is a controlling factor, there is a complete reversal of bearing capacity relations. The plates with the largest $\frac{P}{A}$ values have the smallest bearing capacities. The important consideration in upper range of capacities is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb. The strained condition in the soil, giving rise to the phenomenon of the pressure bulb, tends to become circular. Thus, the round plate is the best shape to take advantage of this factor of strength, the square plate is not quite so good, and the rectangular plate is the least effective of all.

Fig. 22 shows the load deflection diagrams obtained by testing areas of 1, 2, 4, and 9 sq. ft. in the same shape of plate. As in Fig. 21, the material is yellow clay and the pit is the same shape and size as the bearing plate in each case. The decreased capacity of the larger plates with smaller values of $\frac{P}{A}$ is the first impression gained from a study of these diagrams. Again, it is significant that the variation in bearing capacity is not in direct proportion to the perimeter-area ratios of the different plates. In general, it has been found that the larger the bearing area with its smaller perimeter-area ratio, the greater the discrepancy shown between the $\frac{P}{A}$ ratio and the bearing capacity.

On the first studies of the data obtained, it was thought that the perimeter-area ratio was the only variable affecting the bearing capacity. This is the natural result of testing small bearing areas for which $\frac{P}{A}$ is large and the variation in the ratios of the different plates is great. When the largest area tested was 9 sq. ft., the discrepancy between the bearing capacity and the corresponding perimeter-area ratio was not large. It is only when the principle is applied to the much larger areas used in practice that the fallacy of this assumption becomes apparent.

However, for the purpose of discussion, it may be profitable to start with this assumption, which is admitted to be false, and develop the principle as applied to the action of soils and see where it leads. It is assumed that the bearing capacity of two spread foundations of different sizes is proportional to their perimeter-area ratios, the settlement being constant in both cases.

Let p be the bearing capacity, P, the perimeter, and A, the area of the footings considered. Then, for a constant settlement in the two footings, and according to the assumption,

$$\frac{p_1}{p_2} = \frac{P_1}{A_1} \div \frac{P_2}{A_2} \qquad \text{(7)}$$

If the bearing areas considered are round: $P_1 = \pi D_1$; $P_2 = \pi D_2$; $A_1 = \frac{\pi D_1^2}{4}$; and $A_2 = \frac{\pi D_2^2}{4}$. Substituting these values in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \qquad (8)$$

If the bearing areas are square: $P_1 = 4 D_1$; $P_2 = 4 D_2$; $A_1 = D_1^2$; and $A_2 = D_2^2$. Again, substituting in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \qquad (9)$$

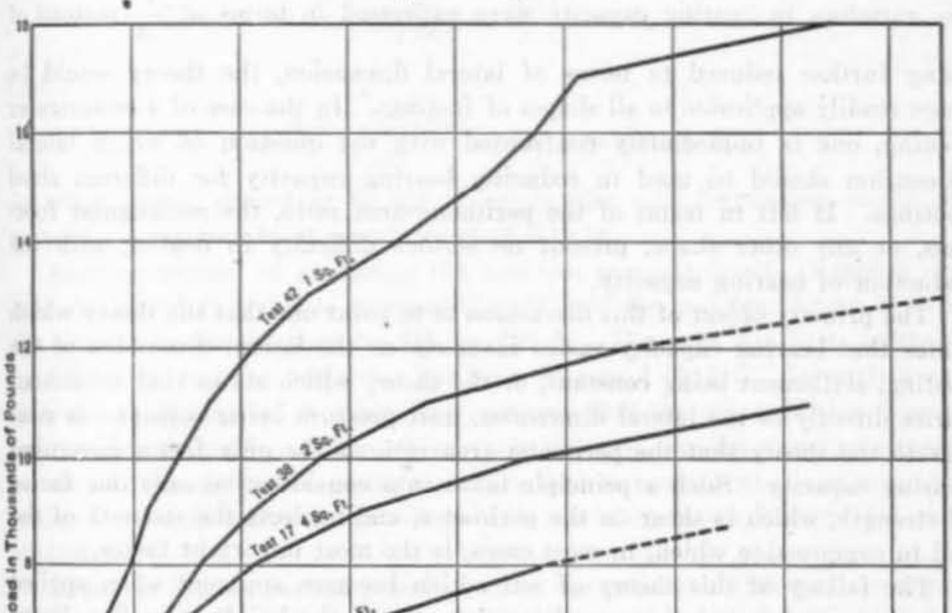


FIG. 22.—COMPARISON OF ROUND PLATES WITH DIFFERENT AREAS.

Thus, it would seem that bearing capacity varies inversely as the diameter in round plates, and inversely as the lateral dimension in the square plates, when the settlement is constant. If the bearing capacity or unit pressure were kept constant, the settlement would vary directly as the lateral dimension, being of course greater for the larger bearing areas. This conclusion, based on an assumption which is believed to be false, corresponds exactly to the theory as stated by Professor Terzaghi. As has been shown, this mistaken theory is the natural result of insufficient data on small sizes of test areas.

Aside from the consideration that the theory is based on a false assumption, there seems to be additional objection to the use of such data as have been obtained, in terms of settlement for different sizes of foundations. To state the theory in terms of bearing capacity for a given settlement seems to be more direct and better adapted to application to practical problems than to leave it in terms of settlement for a given unit pressure. In addition, if the variation in bearing capacity were expressed in terms of $\frac{P}{A}$ instead of being further reduced to terms of lateral dimension, the theory would be more readily applicable to all shapes of footings. In the case of a rectangular footing, one is immediately confronted with the question of which lateral dimension should be used in reducing bearing capacity for different sized footings. If left in terms of the perimeter-area ratio, the rectangular footings, or any other shape, present no serious difficulty in dealing with the reduction of bearing capacity.

The primary object of this discussion is to point out that the theory which states that bearing capacity varies inversely as the lateral dimension of the footing, settlement being constant, or the theory which states that settlement varies directly as the lateral dimension, unit pressure being constant, is similar to the theory that the perimeter-area ratio is the only factor governing bearing capacity. Such a principle takes into consideration only one factor of strength, which is shear on the perimeter, and neglects the strength of the soil in compression which, in most cases, is the most important factor.

The fallacy of this theory of soil action becomes apparent when applied to large spread foundations, such as the case of the building in San Francisco, Calif., cited by Professor Terzaghi. The data from tests for bearing capacity showed an average settlement of 0.10 in. for a unit pressure of 4 800 lb. per sq. ft. These tests were evidently made on a test area of 1 sq. ft. The building was placed on a mat foundation, 218 by 252 ft., designed to carry the dead load of the building at a unit pressure of 4 800 lb. per sq. ft., which was considered allowable.

Applying the proposition that settlement varies directly as the lateral dimension, unit pressure being constant, the settlement should amount to $252 \times 0.10 = 25.2$ in., or $218 \times 0.10 = 21.8$ in., depending on which lateral dimension is considered. The obvious absurdity of these results is excused on the basis that the soil is only slightly cohesive. The test data are quite representative of what might be obtained in a cohesive soil, and to expect a settlement under such conditions comparable to the results obtained from the equations given, would be no less absurd, even if the soil were of the most cohesive type.

To analyze the test data obtained in the example under consideration in terms of bearing capacity instead of settlement leads to an interesting result. Suppose that it was decided to set 0.10 in. as the limiting settlement, and it was desired to find the bearing capacity of a mat foundation, 218 by 252 ft., which would not exceed this limiting settlement. It is assumed that the tests were made on a round plate (1 sq. ft.) with $\frac{P}{A}$ equal to 3.55. The perimeter-area ratio of the mat foundation is,

$$\frac{(218 \times 2) + (252 \times 2)}{218 \times 252} = 0.017$$

Let p be the bearing capacity of the large foundation. Using the direct ratio of bearing capacities to perimeter-area ratios:

$$\frac{p}{4\,800} = \frac{0.017}{3.55}$$

and,

$$p = \frac{0.017 \times 4\,800}{3.55} = 23.0 \text{ lb. per sq. ft.}$$

In other words, according to the principle, the mat foundation would not support its own weight at a settlement of 0.10 in.

Another method of analyzing the problem presents itself. Suppose that it is considered desirable to increase the width of the mat foundation by 1 ft., the dimensions would then be 219 by 252 ft. The area has been increased by 252 sq. ft. and the perimeter has been increased by 2 ft. In such a case it would be difficult to convince any engineer that the only additional carrying capacity gained by adding 252 sq. ft. to the area was the shearing strength on an additional 2 ft. of perimeter. Yet that is exactly what the principles under discussion mean, inasmuch as x is the only factor involved in their application.

In spread foundations of practical sizes, the perimeter-area ratio is always small and in such cases to neglect the shearing strength on the perimeter would not lead to serious error. The other factor of strength is the compressive strength of the soil, or the resistance of the pressure bulb to deformation. The total carrying capacity of a foundation due to this second factor of strength is directly proportional to the area. This would seem to indicate that the much criticized practice of using the same bearing capacity for all sizes of footings was, after all, fundamentally sound. This statement could be defended as having considerable merit, inasmuch as it involves less departure from sound principles than the proposition that bearing capacity varies inversely as the lateral dimension.

The greatest criticism that could be made of existing practice in foundations is that the selected allowable bearing capacity is in most cases based on a guess, governed by experiences in previous foundations, and not on the most reliable information. It seems fair to make the statement that, at present, the only reliable method of determining bearing capacity is by actual test.

Inasmuch as it is not feasible to test areas as large as the practical footing, the tests on smaller areas seem to be the only solution. The small areas involve a large perimeter-area ratio and the shearing strength on the perimeter is a very considerable factor of strength. It thus becomes necessary in reducing the bearing capacity to practical sizes, to consider this factor, but not to the exclusion of the other and more important factor of strength due to the resistance of the soil to compression.

The equation for bearing capacity that the writer has found most useful in interpreting test data, and which fits the test data most satisfactorily, includes both factors of strength.

Let p = bearing capacity, in pounds per square foot.
m = shear on the perimeter, in pounds per linear foot.
n = resistance of the soil to compression, in pounds per square foot.
P = perimeter of bearing area, in feet.
A = area of footing, in square feet.
W = total allowable load on the footing.

Then,

$$W = A\,p = P\,m + A\,n$$

and,

$$p = \frac{P\,m}{A} + n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Let the perimeter-area ratio, $\frac{P}{A} = x$; then,

$$p = m\,x + n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

A study of the form of Equation (11) reveals that it is a linear function involving two variables, namely, bearing capacity and perimeter-area ratio. If the bearing capacity for a certain type of soil is determined by test for two bearing areas having different perimeter-area ratios, it is possible to solve the two resulting equations for the constants, m and n. Having thus determined the shearing strength on the perimeter and the compressive strength of the soil for which the tests were made, the bearing-capacity equation (Equation (11)) can be used to determine the allowable unit pressure for larger footings such that the limiting settlement will not be exceeded.

In Fig. 23 bearing capacity has been plotted against perimeter-area ratio and the corresponding equations for these curves are noted in Table 6. Curves 1, 2, 3, and 4 are bearing capacities for a 1-in. deflection, which was taken as the allowable settlement. Curves 5 to 10 are for the bearing-capacity limit, which is defined as that unit pressure beyond which progressive settlement takes place. It is the unit pressure beyond which the material acts as a viscous fluid, and the consolidation of the material is not sufficient to permit a condition of equilibrium to be attained with a comparatively small amount of settlement.

These equations and curves have been determined by series of tests on the three different types of soil indicated as yellow clay, stiff yellow clay, and blue clay. This classification is merely used as identification and is not intended to have any other connection with the physical characteristics of the material. These characteristics are given by Figs. 21 and 22.

There are a few points worthy of emphasis in connection with this set of bearing-capacity equations and curves (Fig. 23):

(1) As the size of the footing is increased and x approaches zero the bearing capacity approaches, not zero, but a constant value which is the compressive strength of the soil, and is indicated as the intercept on the load axis.

(2) For a type of soil which has a high degree of fluidity and low cohesive strength the bearing capacity is more nearly constant for all sizes, indicating that x is the least important factor in the bearing capacity.

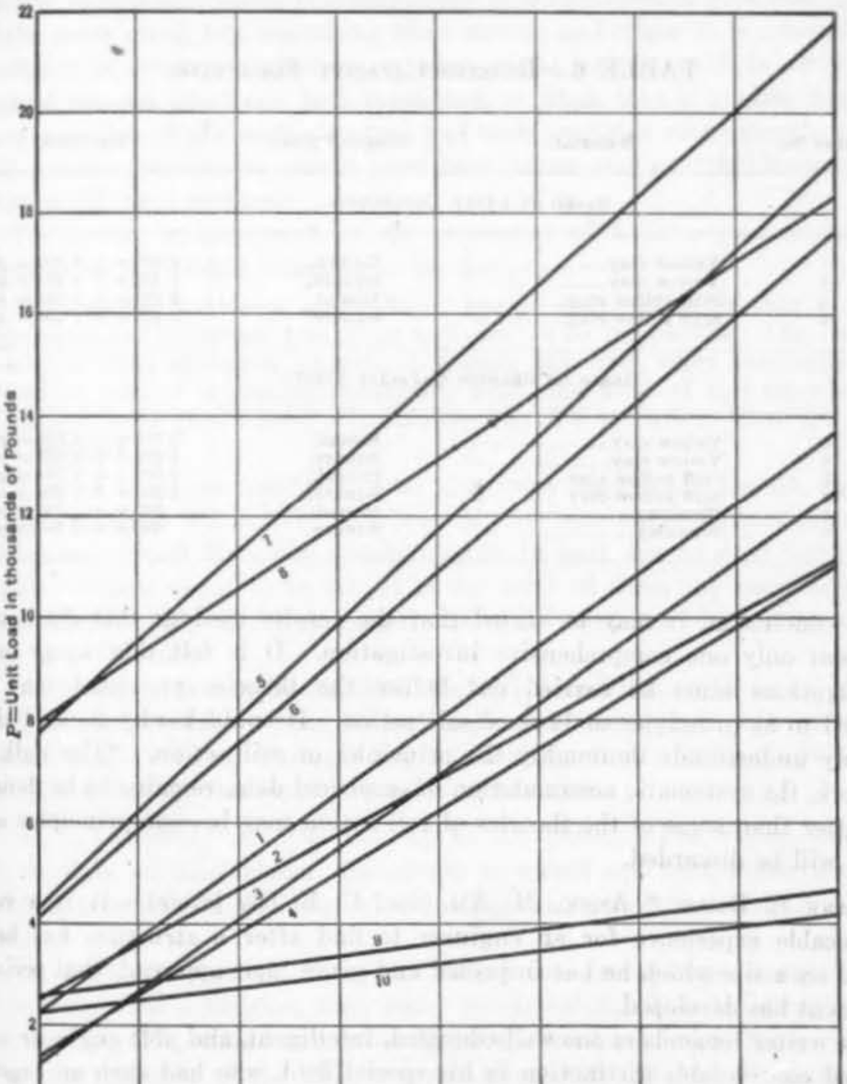


FIG. 23.—BEARING CAPACITY CURVES.

(3) In practical sizes of footings, having perimeter-area ratios between 0 and 1, the shape of the footing, whether round or square, is not so important. This is consistent with the idea that the loss in bearing capacity (as controlled by the pressure bulb), due to the sharp corners, is a smaller part of the total

bearing capacity in large footings. In the smaller areas used in the tests, the loss due to sharp corners is large in proportion to the total area of the plates.

(4) In a type of soil, such as the blue clay, having a high degree of fluidity, the pressure bulb is a controlling factor throughout, and the difference in bearing capacity, due to the shape of the plate, is practically constant. This is clearly shown by the fact that Curves 9 and 10 (Fig. 23) are almost parallel.

TABLE 6.—BEARING CAPACITY EQUATIONS.

Curve No.	Material.	Shape of plate.	Equation.
		BASED ON 1-INCH DEFLECTION.	
1	Yellow clay................	Round.	$2\,880\,x + 2\,260 = p$
2	Yellow clay................	Square.	$2\,150\,x + 2\,530 = p$
3	Stiff yellow clay.........	Round.	$2\,820\,x + 1\,260 = p$
4	Stiff yellow clay..........	Square.	$2\,420\,x + 1\,420 = p$
		BASED ON BEARING CAPACITY LIMIT.	
5	Yellow clay................	Round.	$3\,700\,x + 4\,300 = p$
6	Yellow clay................	Square.	$3\,370\,x + 4\,170 = p$
7	Stiff yellow clay	Round.	$3\,500\,x + 7\,780 = p$
8	Stiff yellow clay	Square.	$2\,600\,x + 7\,950 = p$
9	Blue clay....................	Round.	$520\,x + 2\,580 = p$
10	Blue clay....................	Square.	$490\,x + 2\,250 = p$

In conclusion, it may be stated that the results used in this discussion represent only one comprehensive investigation. It is felt that many such investigations must be carried out before the theories presented may be referred to as principles or laws of soil action. It could hardly be said that anybody understands thoroughly the principles of soil action. "The bulk of the work, the systematic accumulation of empirical data, remains to be done"; and, after that, some of the theories of soil action may become principles and others will be discarded.

FRANK S. BAILEY,* Assoc. M. Am. Soc. C. E. (by letter).—It is a very disagreeable experience for an engineer to find after a structure has been erected on a site which he has inspected and given tacit approval, that serious settlement has developed.

The writer remembers one well-educated, intelligent, and able engineer who attained considerable distinction in his special field, who had such an experience with a settling tank designed for the treatment of industrial wastes. A part of the tank settled and cracked and the representative of the company for which it was constructed, made such caustic criticism of the engineer that he, in turn, was filled with resentment and anger. This engineer died at a comparatively early age and a promising career was cut short.

Another well and favorably known engineer who reached still greater distinction, had a similar experience on a larger scale when a considerable por-

* Structural Designer, New England Power Constr. Co., Boston, Mass.

tion of the sewage treatment plant which he had designed also settled enough to require much expenditure for repairs. This engineer died before his prime, and in both these cases the disappointment and chagrin caused by the imperfection of their finished work may have been contributory causes to their unfortunately early deaths.

Both these engineers may have given and probably did give what they believed to be a sufficiently thorough study to the foundation problems involved in the cases cited, but something went wrong, and while it is generally unprofitable to attempt conjecture on what might have been, it is nevertheless natural for one who knew both these men to think that if studies similar to those recorded in the author's paper had been available soon enough, perhaps still greater precautions would have been taken and no troublesome settlement would have occurred.

The writer is impressed by the statement of such a proficient mathematician as the author is known to be, that,

"Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results."

The writer knows engineers who have been responsible for the design of many structures which have been erected and have stood without appreciable settlement. Such favorable results may be in part due to good fortune, but another reason seems to be that it is the habit of these engineers to make a most careful study of the characteristics of the earth on which a given structure is to be built.

Although it cannot be expected that all foundations will be so perfectly designed in the future that no settlements of consequence will occur, it is certain that Professor Terzaghi has acquired much new information that can be applied to improvement in foundation practice.

T. L. CONDRON,* M. AM. SOC. C. E. (by letter).—The writer contributes to the data on this subject, the results of recent soil tests which may prove helpful to others in deciding what loading may safely be used on a similar soil.

In 1927 the writer's firm was called into consultation regarding the foundations for a building then under construction. This building is located in Chicago, Ill., about two miles from the center of the "Loop District". The contract cost of the structure was about $750 000 and the ground area, about 13 000 sq. ft. The building had been designed with "spread footings", resting on clay, from 20 to 30 ft. below street grade, or from 6 to 16 ft. below Chicago City Datum (Lake level). At the time the consultants were called in, the general excavation was nearly completed down to Elevation — 6.0, or 20 ft. below street grade, and five of the footings, or about 20% of the foundations, were in place. Some question had arisen as to the bearing power of the soil and two soil tests had been made by the Contractor

* Cons. Structural Engr. (Condron & Post), Chicago, Ill.

under the direction of the Architect. The results of these two soil tests caused great concern to all. For each test, a load of 4 000 lb. was placed on an area of 1 sq. ft. The settlement under the first test was reported as 8½ in. in 30 min., and for the second test, 2⁷⁄₁₆ in. in 45 hours. The second test was reported as being made in accordance with the recommendations of the Society's Special Committee on the Bearing Value of Soils for Foundations. The results of these two tests as reported to the writer are shown in Fig. 24(*a*), as well as the result of a third test which is described later.

In view of the very unfavorable showing of these two tests, it was proposed to remove the five spread footings that were in place and substitute for all foundations, concrete cylinders extending down to hardpan, at an estimated extra expense of $55 000 to $60 000.

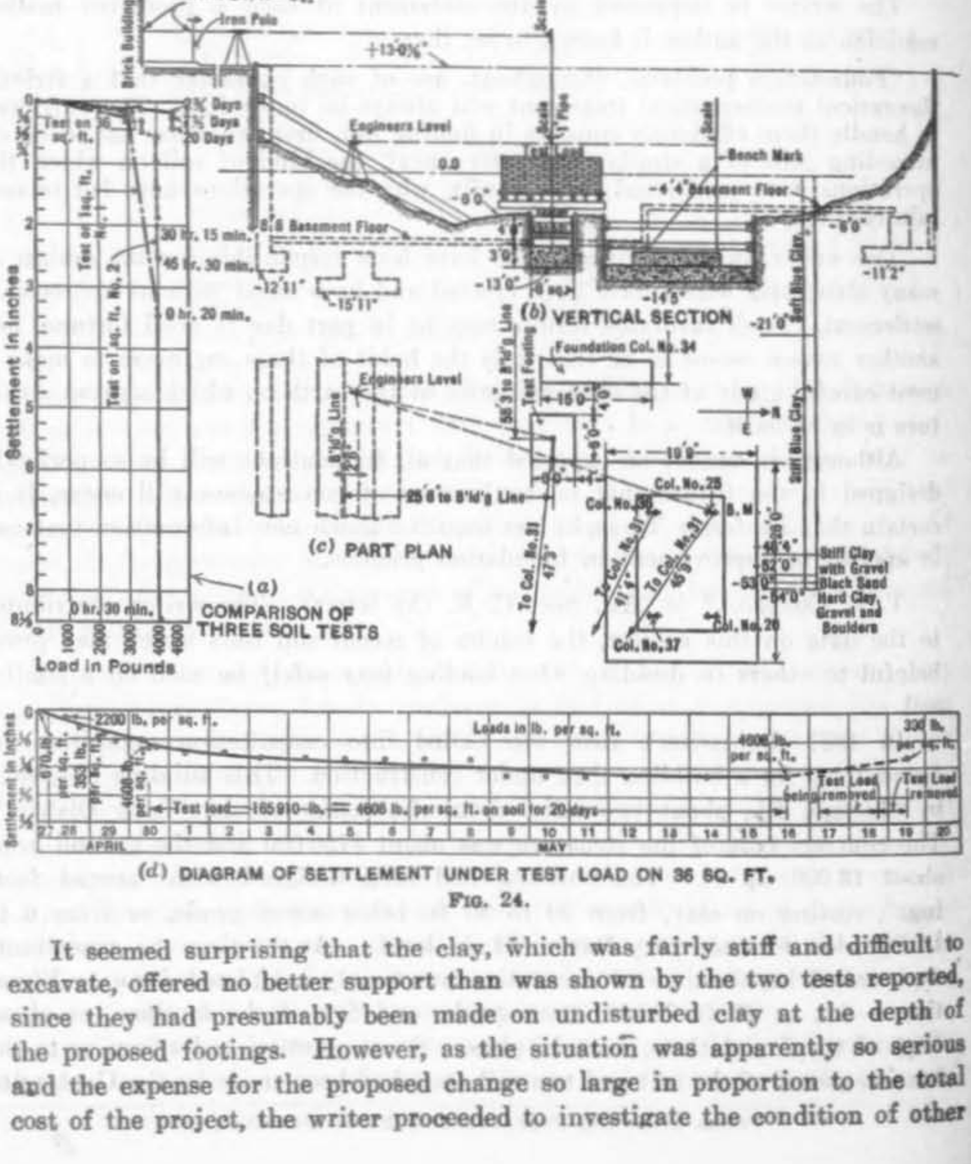


FIG. 24.

It seemed surprising that the clay, which was fairly stiff and difficult to excavate, offered no better support than was shown by the two tests reported, since they had presumably been made on undisturbed clay at the depth of the proposed footings. However, as the situation was apparently so serious and the expense for the proposed change so large in proportion to the total cost of the project, the writer proceeded to investigate the condition of other buildings of similar height and character in the immediate vicinity. The proposed building was intended for non-commercial use and was to be only six stories high, except for a tower to enclose tanks. Several buildings were found near-by resting on spread footings designed for a soil pressure of 4 000 lb. per sq. ft. and none of these buildings showed any evidence of unequal settlement although they were from 10 to 15 years old. The footings for the proposed building were to be loaded to not more than 3 000 lb. per sq. ft. Consequently, the consultants recommended that a further soil test be made before arriving at a decision.

In making this test it was felt that, so far as possible, the conditions should be as nearly identical as possible with those for the designed footings. Consequently, a location was selected (see Fig. 24 (*c*)), where a test footing 6 ft. square could be built and left in place without interfering with any part of the general foundation work. An excavation was made from Elevation — 6.0 to Elevation — 13.0, a depth of 7 ft. The upper 4 ft. of this excavation was sheeted and braced and the lower 3 ft. carefully cut to size and filled with 1 : 2.5 : 3.5 concrete (Fig. 24(*b*)). The lower end of a 1½-in. steel pipe, 20 ft. long, to which a gauge was to be attached for readings, was embedded in the concrete block. When the concrete had hardened, a timber grillage was built on which was placed a timber platform to receive the test load of pig iron. Bench-marks were established on one of the footings in place and on near-by buildings and light poles. The arrangement of this test and its location with reference to two footings then in place and to proposed footings, is clearly shown in Fig. 24(*b*) and Fig. 24(*c*). On this diagram is also shown to scale the result of a typical boring which disclosed "blue clay" increasing in stiffness down to Elevation — 52.0, where clay mixed with gravel was found.

Careful level readings were taken on this test footing as the test load was applied, as shown on Fig. 24(*d*), the initial reading being taken when the load of the concrete and platform was 23 970 lb., or 670 lb. per sq. ft. on the soil. On the third day the total load, including the pig iron, was 165 910 lb., or 4 608 lb. per sq. ft. on the soil. At that time the settlement recorded was one-thirty-second more than ⅛ in. After the test load had been in place 12 days, the total settlement was measured as one-thirty-second more than ¼ in. and no further settlement could be measured during the next 7 days. The test load was then removed and a rebound of $\frac{1}{16}$ in. was recorded. It is interesting to compare the results of this test with the results of the first two tests, as shown in Fig. 24(*a*).

In the writer's opinion this test warrants the conclusion that 3 000 lb. per sq. ft. would be an entirely safe load on this soil. The client was so advised and work was resumed on the building. The full weight of the building has now (1928) been on the foundations for a month or two and no measurable settlement has been reported during the period of construction.

LAZARUS WHITE,* M. AM. SOC. C. E. (by letter).—In the Nineties, when the writer was an undergraduate student, he was thoroughly imbued with the classical methods of computing earth pressures, bearing values of soil,

* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

distribution of pressures, and pile-driving formula commonly taught—Rankine, Baker, Cain, and Wellington—and, after his graduation, he set about to apply them. Fortunately, he was also imbued with the scientific philosophy of Tyndall and Huxley—to accept no theories or formulas unless borne out by experiment. Huxley stated that one clear experiment not accounted for by the accepted theory might forever upset that theory.

The writer's first foundation experience, involving much wood pile-driving, had a disastrous effect on his faith in the *Engineering News* formula. No single experiment conformed to it and to "hit the target" of an observed result, was like shooting at a distant pin with a blunderbuss. Later, during subway construction, he observed that contractors completely and successfully ignored Rankine and Cain in timbering work, and "got away with it" in so many instances that by no scientific philosophy could their theories be justified. Again, his faith in Baker's "Masonry" was upset by the observation that uniform loading did not produce uniform settlement. Hence, with practically all the classical notions upset, the writer decided to observe foundation phenomena for himself and to accumulate data bearing on a new science of foundations.

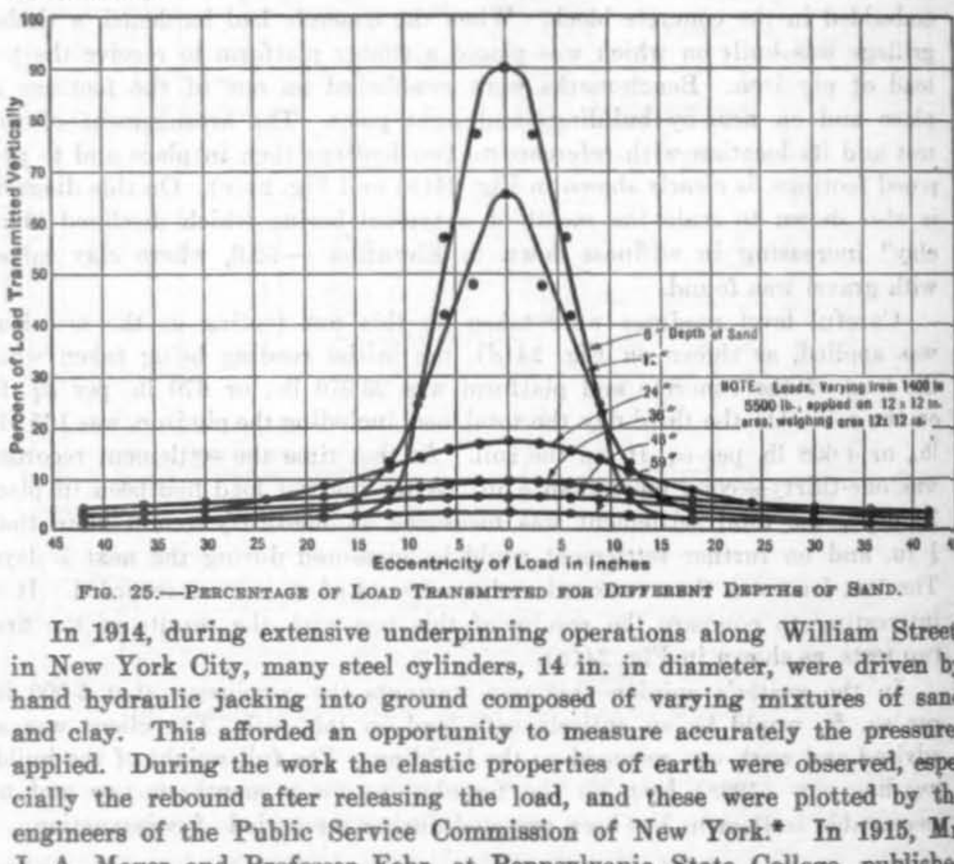


FIG. 25.—PERCENTAGE OF LOAD TRANSMITTED FOR DIFFERENT DEPTHS OF SAND.

In 1914, during extensive underpinning operations along William Street, in New York City, many steel cylinders, 14 in. in diameter, were driven by hand hydraulic jacking into ground composed of varying mixtures of sand and clay. This afforded an opportunity to measure accurately the pressures applied. During the work the elastic properties of earth were observed, especially the rebound after releasing the load, and these were plotted by the engineers of the Public Service Commission of New York.* In 1915, Mr. J. A. Moyer and Professor Fehr, at Pennsylvania State College, published the results of tests† in the distribution of soil pressures measured at various

* *Engineering News*, Vol. 85, No. 27, p. 1268, Fig. 2A.

† *Engineering Record*, Vol. 71, No. 11, March 13, 1915, p. 330.

depths and various eccentricities in relation to the applied loads. (See Fig. 25.) Using these observations as a basis, John F. Greathead, Assoc. M. Am. Soc. C. E., Section Engineer in the days of the William Street Subway, plotted lines of equal vertical pressure beneath a 13½-in., circular plate,* disclosing in a manner which can be readily grasped the general distribution of pressure beneath a footing. This he called the "bulb of pressure"; so far as is known, it was the first time such a diagram was published. (See Fig. 26.) Due to the manner in which the experiments were made—the pressure being recorded on an area equal to the area originating it—Fig. 26 does not show the variations in pressure immediately beneath the footing.

M. L. Enger, M. Am. Soc. C. E., published another bulb of pressure and, because of the use of smaller recording areas, was able to show variations in pressure immediately below the footings. (See Fig. 27.) In general form his diagram is similar to that of Mr. Greathead, but it is more accurate.†

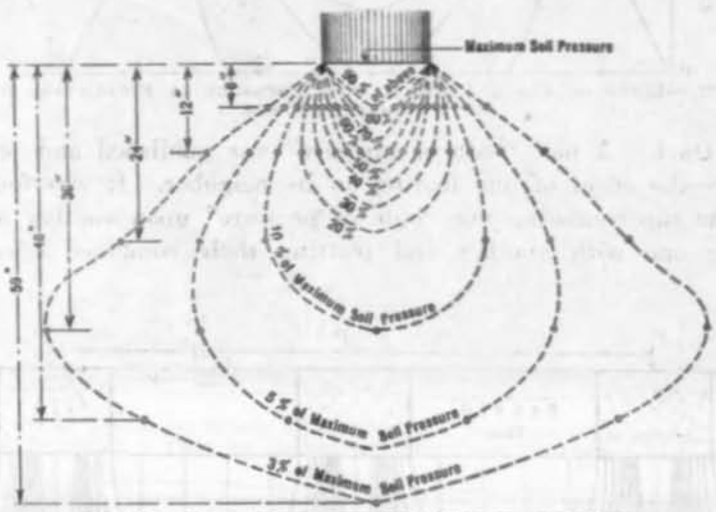


FIG. 26.—LOAD DISTRIBUTION BY BULB OF PRESSURE.

At the same time Professor Enger discovered the great variations in intensities of pressure under footings of different areas, as compared with average unit loads. His formula for such variations is $p = 91 \frac{d^{1.86}}{h^{1.95}}$; in which, p is the ratio (percentage of average unit load) at the depth, h, immediately below the center of the footing, and d is the diameter of the footing. This is of great interest, because if it is assumed that the soil has elastic properties and that a stress-strain relation exists (the applied loads giving the stress and the settlements, the strain or compression), the bearing value of large footings cannot be directly proportional to their areas.

Later, the Joint Committee on Stresses in Railroad Track of the Society and the American Railway Engineering Association in its report,‡ showed the stresses in ballast below railroad ties as ascertained by an elaborate

* *Engineering News-Record*, December 30, 1927, Fig. 3, p. 1037.
† *Engineering Record*, Vol. 73, No. 4, January 22, 1916, p. 106.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1545.

apparatus. Of special interest was a pressure capsule by which intensities of pressure at various points through a mass of ballast could be determined simultaneously. This report is a mine of information for the foundation engineer, especially the full demonstration of the elasticity of the soil beneath

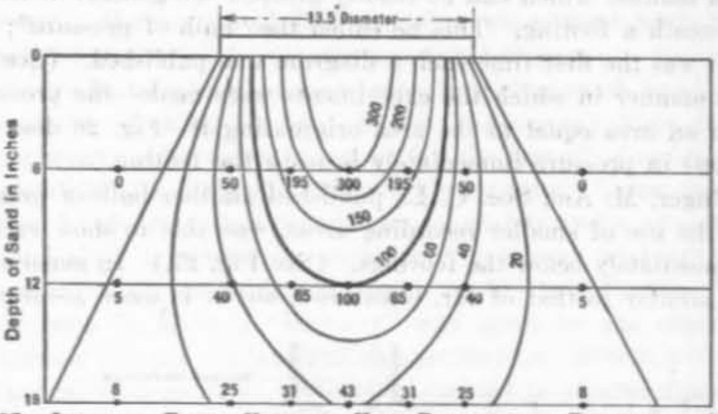


FIG. 27.—LINES OF EQUAL VERTICAL UNIT PRESSURE IN PERCENTAGE BASED ON 4-INCH PLUG TESTS.

a railway track. A new "bulb of pressure" was published and—even more interesting—the effect of one footing on its neighbor. It was found to be the same as superimposing one "bulb of pressure" upon another or, rather, overlapping one with another and plotting their combined effect. (See Fig. 28.)

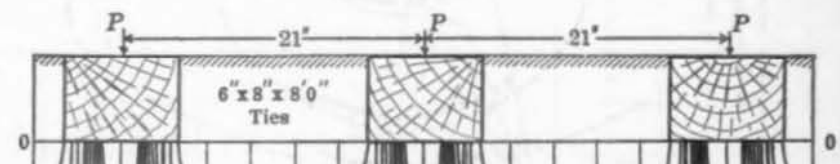


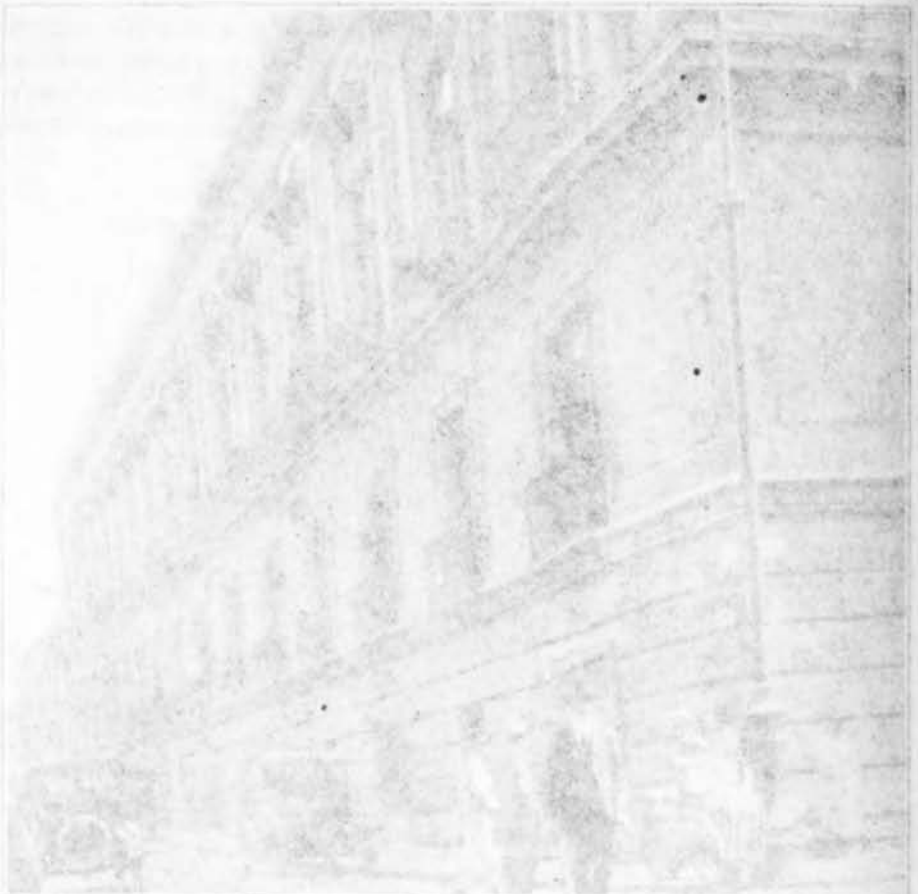
FIG. 28.

In 1925, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., published* a curve indicating that the bearing values of footings or loads for equal settlement

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

Fig. 29.—View of School of Mines, City of Mexico, Showing Differential Settlement of About 3 Feet at Center of a Building Uniformly Loaded.

Fig. 30.—View of National Theatre, City of Mexico, Showing 6-Foot Settlement of Building and Entire Area Around It.



[illegible]

FIG. [illegible].—VIEW OF NATIONAL THEATRE, CITY OF MEXICO, [illegible] were not proportional to areas, but to diameters. Mr. Goldbeck invented a pneumatic pressure gauge, with electrical contacts,* which gave very accurate results. For bearing blocks of 0.371 to 8.33 sq. ft., the device checked the Enger formula quite closely.

In the writer's experience, buildings on compressible soil do not settle uniformly, but in parabolic or catenary curves; the center of the building settles much more than the perimeter. This is particularly marked in the City of Mexico where numerous structures show the characteristic curve in their courses. The reason for this should be apparent from data given herein. Since buildings exert pressure on huge areas, the intensity of pressure at the center is very great, and they cause a corresponding compression of the ground, so that the building finally assumes a dished shape. (See Fig. 29.) The fallacy of the old assumption that structures, uniformly loaded, settle uniformly, is thus graphically shown. Fig. 30 shows the settlement of a building with the entire area around it. The original level of the building was the same as the sidewalk. The total settlement was approximately 6 ft.

The Society's Special Committee on Bearing Value of Soils for Foundations, etc., has performed its work painstakingly, but in a field very difficult and rather barren of results; that is, soil classifications and nomenclature to define soils. An infinite number of definitions can be made and yet one may be "left in the dark". Professor Terzaghi has taken foundation engineers "out of the woods" as far as nomenclature is concerned, and, by establishing a few simple physical classifications, it now seems possible to evaluate soils in terms of bearing power.

In describing the characteristics of clay and its behavior in the presence of water due to viscosity and surface tension, he performed a valuable service. Recently, in Detroit, Mich., while excavating in clay and tapping wet areas under pressure, subsidence of surrounding buildings was observed which conformed to the principle laid down by Professor Terzaghi. A wet clay layer, in which water was supplied from seamy bed-rock below, when tapped at a depth of about 100 ft., yielded a considerable quantity of water. Simultaneously with the pumping of this water, settlements of neighboring buildings, several hundred feet away, were noted. Through the underlying rock the water beneath the subsiding area was partly drained—thus causing shrinkage in the wet clay layer above, which was equivalent to superimposing a load sufficient to squeeze the same volume of water out of the clay.

Another effect, noted at Detroit and Albany, N. Y., was that the wrecking or removal of buildings was followed by slight settlement of neighboring structures, where the region was underlaid by a stratum of wet clay.

It has been long observed that buildings on compressible soil do not settle uniformly as to time, but rapidly at first and then more and more slowly. This has been vaguely ascribed to the fact that water is squeezed from the saturated soil below; but by means of the "thermodynamic parallel", Professor Terzaghi has been able to evaluate this phenomenon and to plot a theoretical time-settlement curve of great value. This also has a geological application to alluvial deposits, and indicates that thousands of years may

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XVI, p. 309, 1916.

elapse before an alluvial deposit is consolidated. In this connection the geology of a stratum is of great significance in locating valuable structures. Nearly all old geological strata are fully consolidated and, therefore, have great bearing capacities, whereas recent alluvial deposits bordering streams, old ponds, or lake beds are dangerous. The latter are not fully consolidated, and their excess of water beyond that which is normal for the old strata is not yet squeezed out. Although the deposit may seem stable, an additional load will cause a flow of water and a subsidence or settlement. In some situations a trained geologist may be a better guide than the engineer, even if the latter has had much experience with foundations, and none in theoretical geology.

To illustrate how a structure settles, the information given in Figs. 31 and 32 is very interesting. Fig. 31 is the plan of a building on which settlements were observed between 1920 and 1924. Fig. 32 (*a*) to Fig. 32 (*g*), inclusive, illustrates the catenary curves of walls resting on compressible soil. The various curves show that the time settlements are rapid at first and then become slower.

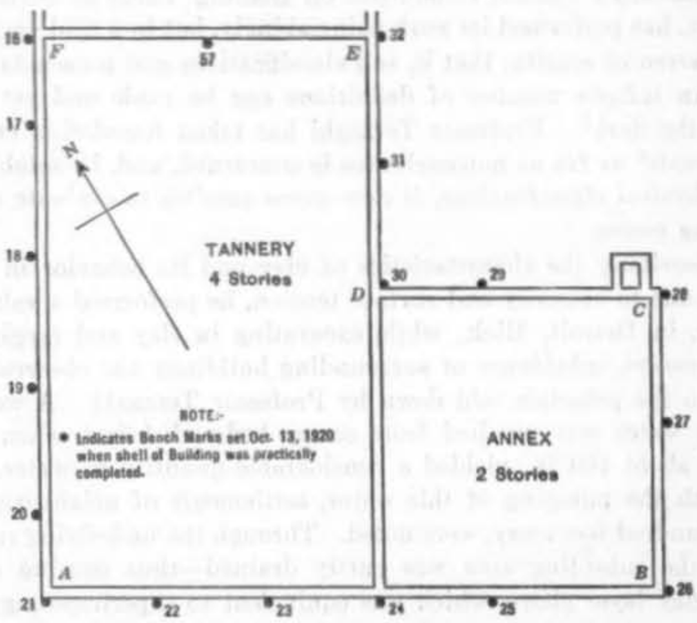


FIG. 31.—PLAN SHOWING LOCATION OF BENCH-MARKS.

The materials beneath the building were thoroughly explored. All test holes (see Fig. 32 (*b*)) were drilled 7 ft. south of the face of the south wall (*A*-*B*) and about 1 ft. inside the curb line. At Bench-Mark 20 (Fig. 32 (*b*)), a pipe was pushed to about 37 ft. Light driving sent it down to 41.7 ft. At this point the penetration was about ¾ in. per blow until, at a depth of 47.2 ft., the last twenty blows sent the pipe down $\frac{3}{16}$ in. and the driving was stopped.

At Bench-Mark 24 (Fig. 32 (*b*)), a pipe was pushed to 33.7 *ft. before* it was necessary to strike a blow. Then, for each successive increment of twenty blows, the penetration was as follows:

Total penetration, in feet.	Penetration for twenty blows, in inches.
35.5	6.5
40.7	4.5
46.1	3.5
46.9	2
48.0	1
48.29	0.19

After a total penetration of 48.29 ft., the next twenty blows failed to move the pipe. A 12-lb. sledge-hammer was used for driving.

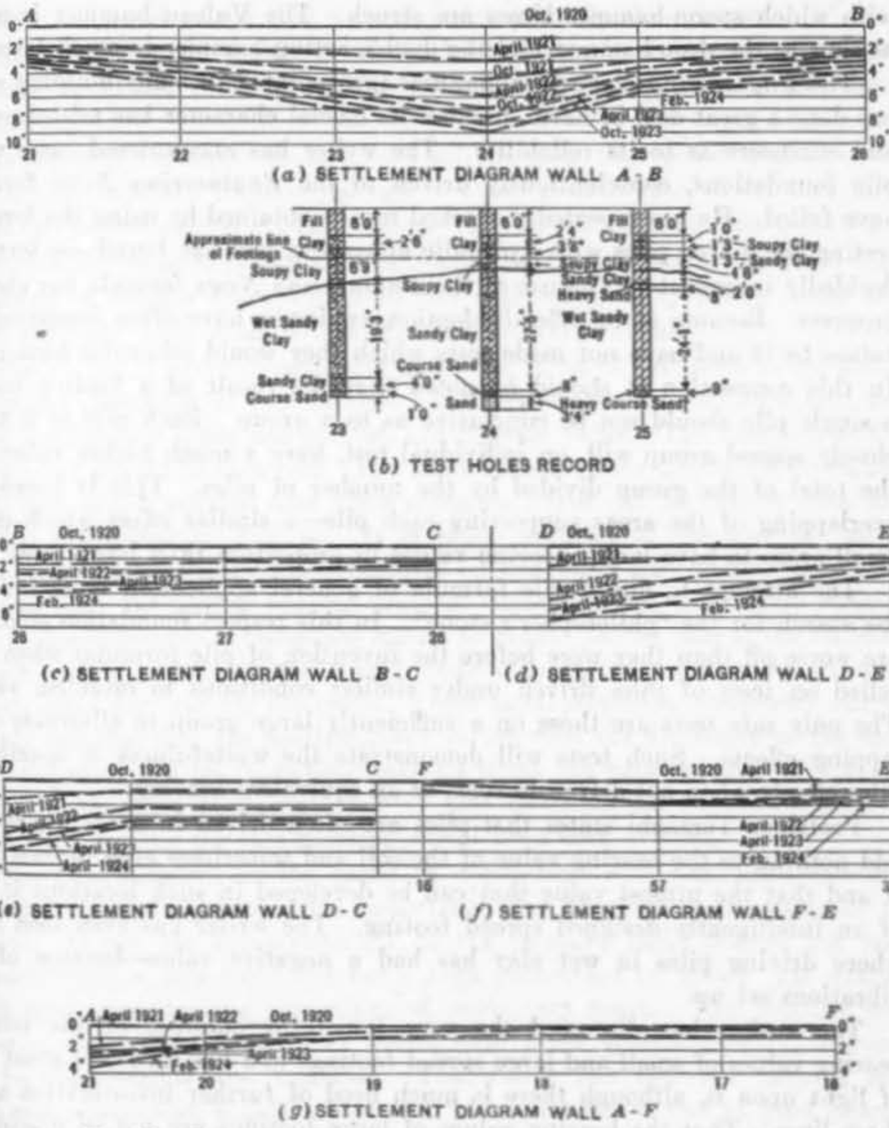


FIG. 32.—RECORD OF SETTLEMENTS OF BUILDING IN SHEBOYGAN, WIS.

The author has dealt the *Engineering News* formula "some staggering blows". Would that it had been "knocked out" completely. It was originated by Wellington, who really intended it to be applied only to uniform materials like sand, with the drop-hammer then used. He introduced constants to give it a reasonable value. Theoretically, it has a factor of safety of 6, that is, a wooden pile which, by the *Engineering News* formula, gives a working value of 15 tons, has an ultimate value of 90 tons, which is absurd for a small wooden pile.

To cover the use of steam hammers, then coming into use, Wellington arbitrarily changed his denominator, $S + 1$, to $S + 0.1$, giving no good reason except that the more rapid blow of the steam hammer ought to give that much better results. As a matter of fact, there is much variation in the rapidity with which steam-hammer blows are struck. The Vulcan hammer is nearly as slow as the drop-hammer and the double-acting hammer is much faster.

The *Engineering News* formula, now incorporated in many building codes, has done a great deal of harm, because its official character has misled owners and engineers as to its reliability. The writer has encountered cases where pile foundations, conscientiously driven to the *Engineering News* formula, have failed. He has repeatedly checked results obtained by using the formula, testing the driven piles with hydraulic apparatus, and has found the formulas decidedly inaccurate. The use of the *Engineering News* formula has checked progress. Because of its official adoption, engineers have often computed pile values by it and have not made tests which they would otherwise have made. In this connection it should be noted that the result of a loading test on a single pile should not be conclusive as to a group. Each pile of a rather closely spaced group will, on individual test, have a much higher value than the total of the group divided by the number of piles. This is because of overlapping of the areas supporting each pile—a similar effect which causes small areas to have larger bearing values in proportion than large areas.

The attempt to find a pile formula of general application is as futile as the search for the "philosopher's stone". In this respect foundation engineers are worse off than they were before the invention of pile formulas when they relied on tests of piles driven under similar conditions to establish values. The only safe tests are those on a sufficiently large group to eliminate overlapping effects. Such tests will demonstrate the wastefulness of spacing of piles closely when not driven to rock, or an equivalent bearing.

Professor Terzaghi states that piles are often driven in areas where they add nothing to the bearing value of the soil and sometimes even detract from it and that the utmost value that can be developed in such locations is that of an intelligently designed spread footing. The writer has even seen cases where driving piles in wet clay has had a negative value—because of the vibrations set up.

The author has discussed the very important question of the relative bearing values of small and large spread footings and has thrown a great deal of light upon it, although there is much need of further investigation along these lines. That the bearing values of large footings are not in proportion to their areas as compared to small footings is to the writer, sufficiently proved. He has encountered many cases where settlements of large footings were much more than those for smaller ones. In a case where a high building was founded on a soft coral rock, the smaller footing settled about ½ in. and the larger, 6 in., although in this case the larger footings were designed with lighter unit loads than the smaller ones. Similar results were observed in buildings underlaid by peat, soft silt, etc.

In designing spread footings the old and easy assumption for the designer (that all that is necessary is to assume a working unit load—so many tons to the square foot—and then to divide this into the column loads to get the areas of spread footings) has led, and will continue to lead, to failures. The heavier the loads the more conspicuous the failures. Account must be taken of the shape of the footings, their relative sizes, their spacing, whether or not they are too close together to work independently, the proportion of dead and live load, etc. On this point the writer begs to differ with Professor Terzaghi as to the value of loading tests. If intelligently made, they have great value.

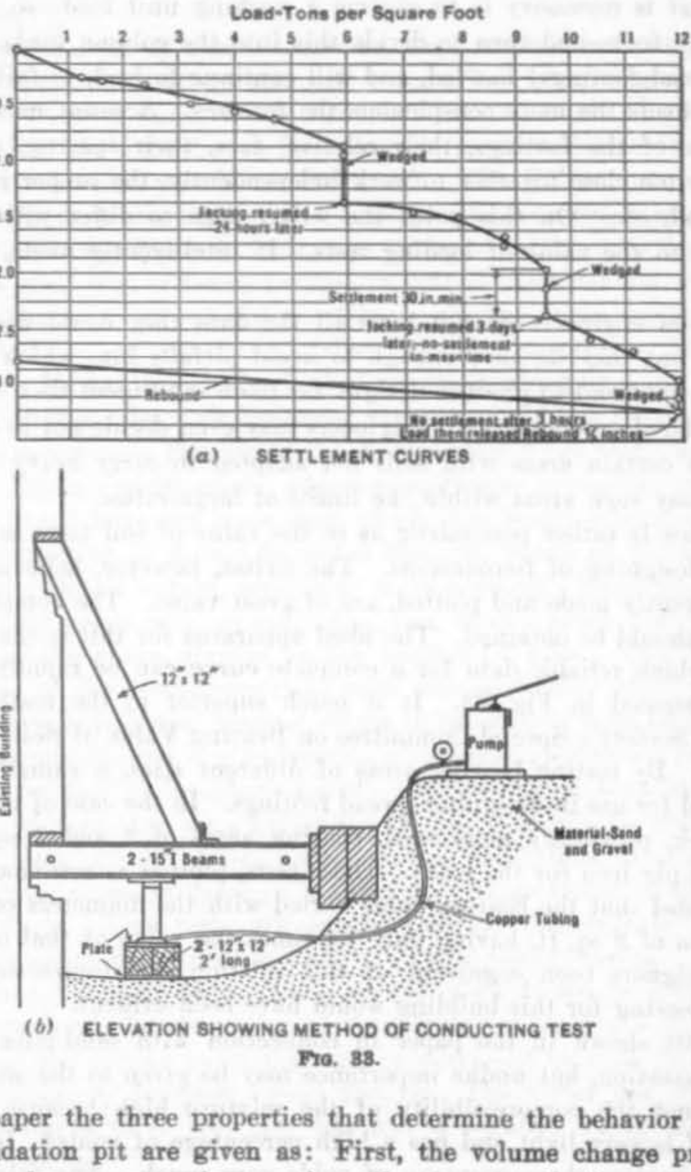
Foundation engineers do not have all the data they could wish—not by any means; but they do have enough to avoid pitfalls into which they have fallen in the past and to produce designs far more intelligent than those heretofore considered good practice. Engineers may even decide not to build high buildings on certain areas with soils not adapted to carry heavy loads, and there are many such areas within the limits of large cities.

The author is rather pessimistic as to the value of soil tests as an aid to the proper designing of foundations. The writer, however, believes that soil tests, intelligently made and plotted, are of great value. The complete settlement curve should be obtained. The ideal apparatus for this is the hydraulic jack, with which reliable data for a complete curve can be rapidly obtained. This is illustrated in Fig. 33. It is much superior to the method* advocated by the Society's Special Committee on Bearing Value of Soils for Foundations, etc. By testing bearing areas of different sizes, a valuable relation may be found for use in designing spread footings. In the case of the building on coral rock, previously mentioned, bearing areas of 1 and 2 sq. ft. were tested, using pig iron for the loads. These tests, plotted as settlement curves, plainly revealed that the bearing value varied with the diameters of the footings, the area of 2 sq. ft. having only 1.4 times the value of that of 1 sq. ft. Had the designers been cognizant of this relation, the impracticability of any spread footing for this building would have been evident.

The results shown in the paper in connection with sand-mica mixtures are most interesting, but undue importance may be given to the shape of the grains. Is not the compressibility of the mixture high because the mica powder itself is very light and has a high percentage of voids? Its mixture with sand increases the percentage of voids very much. Any mixture with a large voids-ratio, or, in other words, with a low specific gravity, is bound to be compressible and unstable. The writer has observed natural sand deposits, apparently highly micaceous, which are dense and of good bearing value, although it may be that actual measurement would show the percentage of mica to be small.

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 527.

The writer does not wish to disparage the use of sand-mica mixtures to simulate earthy materials with various voids-ratios, but wishes to express an opinion that the compressibility of soils is due more to lack of consolidation or to abnormally low specific gravity for the material rather than to the presence of mica or flat grains, although that is a factor in compressibility.



(a) SETTLEMENT CURVES

(b) ELEVATION SHOWING METHOD OF CONDUCTING TEST

FIG. 33.

In the paper the three properties that determine the behavior of the soil in the foundation pit are given as: First, the volume change produced by an increase of pressure acting on the soil, etc.; second, the permeability of the soil, etc.; and third, the cohesion or shearing resistance of the soil.

The writer would like to add a fourth, which may perhaps be a combination of the first two, but which is nevertheless important and easily ascertained; that is, the density of the soil or rock in relation to an old and well consolidated soil of similar nature. For instance, if the material is sand, determine its weight as compared to a sand of an old and well consolidated geological period, and if it is clay, find a similar relation. A very wet, compressible clay will necessarily be lighter because of its water content than a dry well consolidated clay of an old period; silt will be very light when compared to a consolidated earth, and, similarly, a coral rock will be very light compared to an old limestone. This relation is not hard to find, because old earths and rocks have a surprisingly uniform specific gravity.

The author draws a very sharp distinction between cohesive and cohesionless soils. That there are great differences in cohesiveness between different soils is not open to question, but in the writer's experience there is quite a residue of cohesion in all soils. Even clean sand such as that suitable for concrete will break in the bank with a curved fracture and stand vertical for a certain height. The writer is inclined to doubt the great effect of depth on settlement, unless the added excavation reveals a decided improvement of the ground. For instance, Professor Terzaghi states that, for a footing in perfectly cohesionless material, a depth of $\frac{t}{d} = 1$ reduced settlements to only one-third of what they would be if the footing rested on the surface of the ground.

Engineers are greatly indebted to the author for his discussion of the distribution of soil reaction over rigid loaded slabs. He has shown clearly (for the first time to the writer's knowledge) the difference between the result as ordinarily computed with the common theory of uniform pressures and that of the much more correct theory of Professor Enger. A great difference is shown, the more correct theory yielding greater bending moments. This may explain why, in Detroit, a concrete mat supporting a large office building on clay is reported to show numerous cracks indicating failure. Single spread footings, such as the ordinary case of an individual footing for each column, the loads being concentrated under the center of application, will yield smaller bending moments than under the theory of uniform loading.

The writer feels with Professor Terzaghi that the future of foundation engineering as an applied science is decidedly encouraging and also that engineers are merely on the threshold of this science. The principle obstacles to progress having been removed, that is, various century-old assumptions that do not conform to Nature, the engineer can now go ahead.

Much can be added to the present knowledge of foundations by applying the methods originated by Professor Enger and Mr. Goldbeck for determining soil pressure. By placing pressure capsules in the soil beneath actual footings the distribution of stress can be determined so that it is possible to compute correctly the bending moments in footings. The relation of depth of footing to settlements and that of areas of footings to settlements, can also be determined, if not by this method then by direct loading tests on footings of different sizes and at different depths. By the use of hydraulic apparatus it is possible to obtain a wide range of tests in a limited time. By this method the effect of various shapes of footings on bearing capacities and settlements may also be found, and by testing groups of piles singly and in combination, the investigator can determine overlapping effects, etc., the proper spacing of piles for the best results, and the relation between the bearing capacities of a single pile and a group.

In the past engineers were content to state that a building settled very little, but accurate levels recorded, with comparatively little cost, would yield valuable and surprising data. In the future, various representative buildings should be chosen for the purpose of making accurate observations as to settlement. Professor Terzaghi is correct; foundation problems are of such a character that strictly theoretical mathematical treatment will always be impossible, but by systematically accumulating field data it will be possible to approach this ideal much more closely.

The writer hopes that the problem of soil classification will soon be solved along the lines suggested in the paper. He feels that the Engineering Profession is much indebted to Professor Terzaghi, first of all for uprooting certain persistent prejudices and then for leading engineers forward in new directions toward "the promised land" of a real science of foundations.

R. D. N. Simham,* Assoc. M. Am. Soc. C. E. (by letter).—The writer desires to limit his observations on this interesting paper to the most important question of settlement of soils and its relation to the area of bearing and intensity of loading. It is a fallacy to generalize from experimental results without thoroughly understanding the premises on which, and the objects for which, the experiments are conducted. There are limitations to experimental researches that require corresponding limitations to be made in the laws deduced from them.

In the first place, any results that have been determined by individual experiments under limited considerations and very often in a slipshod manner, that is, without any attempt to follow a general or comprehensive plan and co-ordinated line or method of experimentation, cannot be complete evidence on which to establish any general or fundamental law of science. The writer has learned from experience that any attempts to deduce, from an unco-ordinated set of experiments, some sort of a general formula or law that would explain the entire phenomena of soil behavior, has invariably raised serious complications. Therefore, experiments can afford no relief to the present imperfect state of the science of foundations unless they are based on: (1) Constructive lines of thought; (2) a thorough realization of the assumed conditions; (3) a careful diagnosis of every factor that influences the behavior of a soil; and (4) a thorough co-ordination of all the information gained. "Knowledge comes, but wisdom lingers."

There is always a tendency to take some plausible results of experiments and immediately deduce laws of relationship between certain factors involved and then apply them to every single condition that occurs thereafter. Whenever inconsistencies are discovered, all kinds of sophistical arguments are made to explain matters, either by suggesting that the variations are exceptional, that they are unexpected, or are due to defective experiments, or other causes, but all the time insisting that the law, once derived by some mathema-

* Town Planning Asst., Madras, India.

tical manipulations of a set of convenient results, must apply to all sets of conditions.

In an ideal soil, uniform in every respect, it is possible to state that settlement is proportional to intensity of loading in some way and to the area in some other way. With regard to the area, the shape of the bearing surface has an influence on settlement; that is, the settlement of a foundation with the same load and bearing area would be different for dissimilar shapes of the bearing surface. Thus, in a simple way, the relationship between settlement of soil, s; load intensity, w; bearing area, A; and the ratio of mean width to mean length of bearing section (or some other relation that would truly represent the effect of variation in shape and the consequent variation of volume of depth of soil affected), r, may be given in the general form:

$$s = \frac{w^x A^y r^z}{k} \qquad \text{(12)}$$

In this equation, k is a constant that expresses the characteristics of the soil.

If this may be taken as truly representing a law of behavior for given soils, then it would be easy to direct some experiments along definite lines and to arrive at the range between which the values of x, y, and z vary. However, in the determination of the value of k, the investigator would probably get into difficulties. The writer thinks that it is impossible to secure identical behavior of soils throughout the processes of varying load, area of bearing, and shape. Any settlement may be the result of several different reactions within the soil. There may be (1) a subsidence due to closer re-arrangement of materials of soil during a particular stage; (2) settlement due to plastic or flattening effect at some other stage, or combined with the former; (3) sinking due to escape or lateral flow of water or escape of other loose materials in the composition of soil; (4) shrinking due to compressibility of soil materials; and (5), under extraordinary circumstances, reduction caused by the crushing of the granules or disintegration of soil materials. Equation (12) may perhaps be modified further to cover these factors also,

$$s = \frac{w^x A^y r^z}{k - (c_1 + c_2 + c_3 + c_4)} \qquad \text{(13)}$$

In Equation (13), k may represent a value for soil under standard conditions, or a certain condition of soil behavior when settlement is a minimum. c_1, c_2, c_3, c_4, etc., may represent the reductions necessary in the values of k to allow for subsidence, settlement, shrinking, reduction, etc.

Any soil that is confined before loading to the limit of the natural angle of repose, $x\,O\,A$ (Fig. 34), when loaded, slowly or rapidly adjusts itself in different angles until it attains the limiting angle of compactness of soil materials as indicated by the position, OB, in Fig. 34. Beyond this, a loading will cause the soil to flatten to the limiting position of plasticity, $C'C$. During this period, there may not have been any change in the volume of soil, but merely a bulging outward. Then sinking of soil occurs by the escape of water or loose material within the soil, until the limiting angle of permeability or soil escape, $x\ O\ D$, is reached. Further loading creates pure compression of the soil materials, and the soil shrinks to the limit of compressibility, $E'E$. Beyond this stage the soil sinks by the crushing of the granular shape. Thus, the actual curves of settlement *versus* load, based on experimental results, would seem to illustrate, not one uniform law throughout, but different laws at different stages. Knowing exactly what is taking place within the soil, itself, it should be possible to determine these laws independently through carefully planned experiments, and then, by an easy mathematical computation, a combined or general law of relationship might be deduced.

In another way, assuming that the total settlement of the soil is the combined effect of these factors, it may be found convenient to express the relation for determining the settlement in a somewhat better form, thus:

$$s = \frac{w^{x1} A^{y1} r^{z1}}{k_1} + \frac{w^{x2} A^{y2} r^{z2}}{k_2} + \frac{w^{x3} A^{y3} r^{z3}}{k_3} + \frac{w^{x4} A^{y4} r^{z4}}{k_4} \ldots\ldots (14)$$

in which the quantities, $\frac{w^{x1} A^{y1} r^{z1}}{k_1}$, etc., represent the settlement due to the individual effects of each of the factors mentioned.

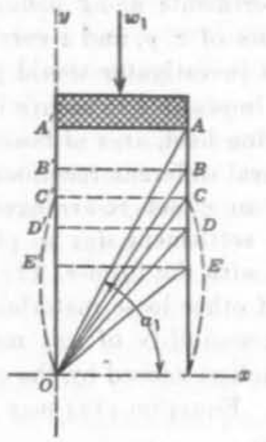


FIG. 34.

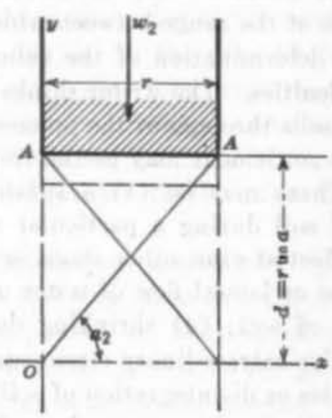


FIG. 35.

There are two more considerations which, instead of increasing the settlement, would reduce it. The side soil protection and frictional resistance offered by the marginal soil should modify the settlement directly below the foundation, and then Equations (13) and (14) will be further generalized to the following forms:

$$s = \frac{w^{x} A^{y} r^{z}}{k - (c_1 + c_2 + c_3 + c_4)} - \frac{V^m}{C} - \frac{P^n}{D} \ldots\ldots\ldots\ldots (15)$$

and,

$$s = \frac{w^{x1} A^{y1} r^{z1}}{k_1} + \frac{w^{x2} A^{y2} r^{z2}}{k_2} + \frac{w^{x3} A^{y3} r^{z3}}{k_3} + \frac{w^{x4} A^{y4} r^{z4}}{k_4} - \frac{V^m}{C} - \frac{P^n}{D} \ldots (16)$$

In Equations (15) and (16), V and P represent, respectively, the effective volume of soil contributing to the side protection of the compressed soil and the perimeter of the bearing section; while m, n, C, and D are constants.

These do not necessarily comprise all conceivable factors affecting soil behavior. There is no doubt a difference in the behavior of soil under dead and live load and slowly or rapidly applied load. Eccentricity of loading and character and nature of distribution of thrusts on soil have also to be considered. In fact, it is necessary to know the law connected with each individual factor that influences the soil, and then only can any comprehensive law or laws, which would correctly represent the soil behavior, be attempted. Any such attempt at the present state of knowledge on this subject will naturally be abortive. However, for practical purposes, it may perhaps be sufficient to know what volume of soil will be affected by any load over any bearing area and deduce, with reference to it, some simple relation for the settlement of soil of any specific composition under different loads.

The volume of soil that is directly affected by load is as shown in Fig. 35. This concept does not take into consideration the soil that receives any indirect or virtual effects of load. It limits the volume to that portion lying constrained laterally within the vertical planes bounding the bearing area. The bottom would be defined by a horizontal plane (*Ox*, Fig. 35) passing through the highest point of intersection of the plane (*OA*, Fig. 35) of shearing or cohesion and the vertical plane, *Oy*. The direct settlement of the soil may be considered the result of (1) a closer re-arrangement of soil material; (2) actual compressibility; and (3) reduction due to permeability of this volume of soil or a large proportion of it. Any relation so determined will be useful in a practical way in studying comparative settlements or comparative bearing values of soil.

With these assumptions the relation between settlement and volume for any specific composition of soil may be simply written,

$$s = \frac{w^p\,(c\,V)^q}{k} \qquad \text{(17)}$$

in which, V is the volume defined in connection with Fig. 35, and p, c, q, and k are constants. In Fig. 35, let A = the area of the bearing surface; r, the mean or limiting width of soil that restricts the depth, d, of soil affected directly by the load; and a, the angle between the plane of shear or cohesion and a horizontal plane through O. Then,

$$V = A\,r\tan a \qquad \text{(18)}$$

Substituting this value of V in Equation (17),

$$s = \frac{w^p\,(c\,A\,r\tan a)^q}{k} \qquad \text{(19)}$$

For any specific soil c, $\tan a$, and k will be constant, so that settlement in such case would be represented by the relation,

$$s = \frac{w^p\,A^q\,r^q}{C} \qquad \text{(20)}$$

in which, C is a general constant for the characteristics of the specific soil.

As an example, select two kinds of soils to be compared for bearing values. Assume that s, p, c, A, r, and q remain the same in both cases; w_1 and w_2 are the supported loads; a_1 and a_2, the shearing angles; and

k_1 and k_2, constants for the soil in each case. Then,

$$\frac{w_1}{w_2} = \frac{k_1}{k_2}\left(\frac{\tan a_2}{\tan a_1}\right)\frac{q}{p} \qquad (21)$$

Equation (21) explains in a simple way the values found by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., with various mixtures of sand and clay.*

FRANK A. MARSTON,† M. AM. SOC. C. E. (by letter).—Considerable progress has been made by Professor Terzaghi in determining adequate and practical methods of classifying soils. For several years, the writer's firm has been carrying on laboratory and field studies on soils with the assistance of and by means of apparatus devised by the author.

In doing such work, there is a real benefit derived from the actual handling of the soil samples. It is not enough to have a trained laboratory man perform certain tests, but, in addition, some work should be done by the engineer responsible for the final decisions in order to acquire the "feel" of the various soils and to observe closely their characteristics as evidenced by varying behavior under test. While the external appearance of a soil sample should be studied, it may be most deceiving as to the actual characteristics and cannot be depended upon solely as a means of grouping soils.

As time goes on, methods and apparatus will be further simplified and their use will be more widespread. The great difficulty at present in the practical use of these improved methods of soil analysis is in the interpretation of the results. Only the gradual accumulation of data will provide the means for accurately translating laboratory results into terms of large-scale field experience. In preparing papers such as this, Professor Terzaghi is rendering the profession a real service by arousing the interest of many investigators. If, as a result of these discussions and investigations, soil studies can be conducted according to standardized methods, records will be made that can be compared and used as a basis for judgment by engineers generally. A tremendous amount of data has been published regarding experiences with foundation conditions, but the lack of a suitable classification for the individual soils has made many of these data of little value.

An illustration can be cited showing the slowness of consolidation of the core material in a certain hydraulic-fill dam, with some data as to the character of the material according to these new lines of classification. All the material for this dam was obtained from a borrow-pit in shale located on a steep hillside which was covered with a layer of disintegrated material 3 to 4 ft. thick. This layer was the principal source of the core material.

Ten samples of the core material were taken at intervals from the top to a depth of 15 ft., at two points (Holes Nos. 1 and 2, Table 7), on the center line of the dam. These holes were 12 ft. and 150 ft., respectively, from the point of overflow of water from the pool during construction.

Samples of core material from the upper parts of the holes were obtained by pushing down a 3-in. pipe and withdrawing it filled with material. This

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 271.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

method was continued until a depth was reached at which the core material was so soft that it would not remain in the pipe. Below this point, the holes were excavated by means of an earth auger which worked with fair success, except that some soft material slid off the auger as it was pulled up. The sampling tubes, 1 in. in diameter and 12 in. long, were used with some difficulty, due to the soft condition of the material which did not always fill the entire length of the tube.

When a ball of the core material was squeezed in the hand, it was found to be so plastic that it flowed out between the fingers. In one test hole, the material at a depth of 12 ft. contained so much water that, after standing over-night, the hole was found to be filled with water to the level of that in the reservoir. Table 7 gives the results of some of the tests of the core material.

There was little difference in the character of the samples from the two holes. The moisture in the samples as taken from Hole No. 1 varied from 19.3 to 42.4% and averaged 33.5% of the weight of dry solids. The samples from Hole No. 2 varied in moisture content from 26.4 to 48.2 and averaged 35.5 per cent. The moisture content did not vary uniformly from the top to the bottom of the holes. The average volume of voids in the samples from Holes Nos. 1 and 2 was 48.6 and 48.0%, respectively, computed on the volume of the wet material as sampled. The specific gravity of the dry core material was 2.748.

A comparison of the liquid limit given in Column (5), Table 7, with the moisture content given in Column (11) (both being expressed in the same terms), shows that about one-half the samples contained more moisture when taken out than at the lower liquid limit.

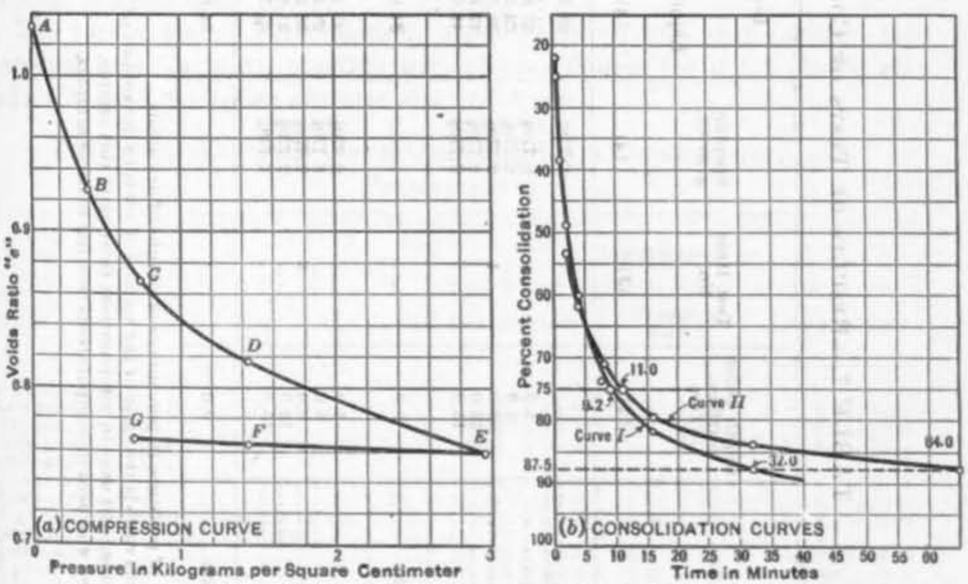


FIG. 36—SAMPLE NO. 712 FROM TEST HOLE NO. 2.

A compression test was made on Sample 712 of this core material for the purpose of determining the compressibility and permeability of the material (see Fig. 36 and Table 7).

TABLE 7.—RESULTS OF TESTS OF CORE MATERIAL FROM AN HYDRAULIC-FILL DAM.

Sample No.	Depth below surface, in feet.	Test Hole No.	Specific gravity.	Lower Limits in Percentage.			Voids Ratio at Limits.†			Moisture content of sample as removed, in percentage of weight of dry solids.
				Liquid.*	Plastic.*	Difference.	Liquid.	Plastic.	Difference.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
603	....	....	2.778‡	33.5	23.1	10.4	0.931	0.642	0.289	....
(Old dry sample)	....	....	......	....	....	....	......	......	......	....
703	3.0	....	2.770§	26.3	23.3	3.0	0.738	0.645	0.093	19.3
704	6.0	....	2.770§	33.2	23.4	9.8	0.917	0.648	0.269	31.3
705	9.0	1	2.770§	32.5	22.0	10.5	0.900	0.609	0.291	34.3
706	12.0	....	2.770§	37.3	28.7	8.6	1.034	0.795	0.239	42.5
707	15.0	....	2.770§	41.0	27.8	13.2	1.135	0.770	0.365	40.2
Average (703-707)	9.0	....	......	34.06	25.04	9.02	0.945	0.693	0.252	33.5
708	3.0	....	2.770§	42.2	31.5	10.7	1.078	0.872	0.206	48.2
709	6.0	....	2.770§	28.2	20.9	7.3	0.779	0.579	0.200	28.6
710	9.0	2	2.770§	32.3	23.2	9.1	0.895	0.642	0.253	40.5
711	12.0	....	2.770§	31.9	23.5	8.4	0.884	0.651	0.233	26.4
712	15.0	....	2.748‡	35.7	25.2	10.5	0.982	0.693	0.289	34.0
Average (708-712)	9.0	....	......	34.06	24.86	9.20	0.924	0.687	0.237	35.5

* Weight of moisture in percentage of weight of dry solids.

† The voids ratio is the ratio of the volume of voids to the volume of solids.

‡ Determined and used in computations relating to this sample.

§ Assumed and used in computations relating to these samples.

The dry material was made into a paste and placed in the cylindrical container of the compression apparatus in such a manner as to exclude air and completely fill the space. The paste was approximately at the liquid limit. Pressure was then applied at 0.39 kg. per sq. cm. Observations were made, by means of Ames dials, of the decrease in thickness of the specimen at frequent time intervals until there was no further consolidation. The load was then increased immediately to 0.74 kg. per sq. cm. and similar observations of thickness were made. Similarly, the test was repeated for loads of 1.44 and 2.96 kg. per sq. cm.

When consolidation had been completed, the load was reduced by the same increments by which it had been increased, allowing sufficient time for the sample to expand or rebound to a constant volume between changes in load. The moisture content of the sample was determined before and after testing. Other values used in the analysis were: Specific gravity, 2.748; plastic limit, 25.2 ($e = 0.693$); liquid limit, 35.7 ($e = 0.982$); diameter of the sample, 7.00 cm.; and reduced thickness, 0.705 cm. In Fig. 36 (*b*), Curve I is the consolidation curve when the pressure was increased from 0.74 to 1.44 kg. per sq. cm., and Curve II is the consolidation curve when the pressure was increased from 1.44 to 2.96 kg. per sq. cm.

Point *A* (Fig. 36 (*a*)) represents the voids ratio at the beginning of the test; Points *B*, *C*, *D*, and *E* represent the voids ratios of the completely consolidated specimen under the loads of 0.39, 0.74, 1.44, and 2.96 kg. per sq. cm. Points *F* and *G* represent the voids ratios when the specimen had rebounded after removing loads of 1.52 and 0.70 kg. per sq. cm., respectively. These observations indicate that the material consolidates almost as slowly as a highly colloidal clay.

The coefficients of consolidation, compressibility, and permeability were computed (see Table 8), together with similar figures for a fat plastic clay and a fine sand having an effective size of 0.1 mm.

TABLE 8.—COEFFICIENTS OF CONSOLIDATION, COMPRESSIBILITY, AND PERMEABILITY.

	Coefficients of.		
	Consolidation.	Compressibility.	Permeability.
Fat Plastic Clay:			
Low pressure	1.78×10^{-8}	4.4×10^{-4}	7.6×10^{-7}
Medium pressure	2.62×10^{-8}	2.3×10^{-4}	6.0×10^{-7}
High pressure up to 2.96 kg. per sq. cm	2.10×10^{-8}	1.1×10^{-4}	2.8×10^{-7}
Core Sample No. 712:			
Low pressure	2.78×10^{-8}	1.69×10^{-4}	4.7×10^{-7}
Medium pressure	11.7×10^{-8}	0.74×10^{-4}	8.7×10^{-7}
High pressure up to 2.96 kg. per sq. cm	5.87×10^{-8}	0.38×10^{-4}	2.2×10^{-7}
Fine Sand, Effective Size = 0.1 mm.:			
Low pressure	$171\,000 \times 10^{-8}$	0.5×10^{-4}	$85\,300 \times 10^{-7}$
Medium pressure	$378\,000 \times 10^{-8}$	0.22×10^{-4}	$88\,300 \times 10^{-7}$
High pressure up to 2.96 kg. per sq. cm	$500\,000 \times 10^{-8}$	0.16×10^{-4}	$80\,000 \times 10^{-7}$

It will be seen that the coefficients of consolidation and permeability for the core material are similar to those of the fat plastic clay and vary greatly from those of the fine sand. The greater the coefficient of consolidation, the more rapidly consolidation will take place. For the core material, the rate of consolidation is exceedingly slow. The indications are that the core in the dam built with this material to a depth of 15 ft. at least, is in about the same state of consolidation as it was when the dam was built, eight years before the samples were taken.

The influence of grain size on the variation in lower plastic and liquid limits and the difference between these limits is illustrated by data given in Table 9. These data were obtained by separating into its respective portions (according to grain size) a sample from the borrow-pit from which the core material was obtained. This material is a bluish or greenish shale which breaks into bulky pieces.

TABLE 9.—LOWER LIQUID AND PLASTIC LIMITS OF THE SEVERAL FRACTIONS OF FINE MATERIAL FROM BORROW-PIT.

Size, in millimeters.	Proportion of total weight, percentage.	Liquid limit.	Plastic limit.	Difference.
Composite sample	100	33.0	24.7	8.3
0.1 — 0.02	35.2	26.9	29.8	0
0.02 — 0.006	22.2	38.2	36.8	1.4
0.006 — 0.002	17.2	42.2	29.6	12.6
Less than 0.002	25.4	66.8	36.1	30.7

For material between 0.1 and 0.02 mm. in size, the liquid and plastic limits should be practically identical. The discrepancy shown in Table 9 is due to the difficulty in determining accurately the plastic limit in such material. The effective size of the fine material was 0.00075 mm., and the uniformity coefficient was 20.6. The coarsest fraction, 0.1 — 0.02 mm., had relatively low liquid and plastic limits and the difference between them should have been practically zero. The very fine material (less than 0.002 mm.) had high liquid and plastic limits and there was a wide difference between them.

An examination of the borrow-pit material before construction by the methods indicated would have made it possible to forecast the behavior of the material.

ARTHUR M. SHAW,* M. AM. SOC. C. E. (by letter).—This paper is quite unique in that it discusses clay and other soils in the same terms as have been used in the discussion of other materials of construction. While many experimental data have been compiled concerning the bearing power of soils, it ordinarily has not occurred to the engineer dealing with these widely used materials to investigate such properties as tensile strength, elasticity, and especially strength in shear, although the author shows that all these properties are of vital importance in at least some types of soils. The writer was particularly interested in the discussion of the relation of area to the supporting power of soils, having noticed the apparent inconsistent behavior of

* Cons. Engr., New Orleans, La.

weak soils under loads of uniform weight per square foot, but varying materially in area covered.

The following discussion will be limited to muck soils, overlying soft clay, the type of soils commonly found in the "prairie" areas of the lower delta of the Mississippi River.

A number of years ago, it became necessary to place a fuel-oil tank convenient to a pumping plant on a reclamation project near New Orleans, La. The soil, of the type described, had been drained only partly and under a test load (placed on a 3 by 3-ft. platform) appeared to be safe for a load of approximately 500 lb. per sq. ft. A spread foundation was designed to limit the load to 400 lb. per sq. ft., but before the tank was quite three-quarters full, it began to settle unevenly and to an undesirable degree. Hasty shoring brought the tank to a vertical position, an occasional re-adjustment of the shores being necessary to keep it plumb. After about six months of subdrainage, and compacting by the superimposed load, the soil was capable of supporting the full load of 400 lb. without material additional settlement. The total settlement was 2.5 ft.

On a similar reclamation project, experiments were made with the freshly drained muck soil to determine loads which might be used in the construction of light buildings for housing employees. A limit of 250 lb. was adopted and results indicated that this was about correct. General settlement was expected because of the drainage and shrinkage of the muck, but this was fairly uniform and did not cause any serious complications except in connection with one or two buildings which were provided with brick flues. These were carried on short posts (old pile-heads) which rested on a thin local stratum of sand only about 5 ft. below the surface. The considerable separation of grade of the living-room floor and the fireplace has resulted in a novel architectural effect. The Superintendent (who has been in charge since the work was started in 1917) advised that he now (1928) is using a load of 500 lb. per sq. ft. for small footings, but for large footings, he has found it necessary to adopt a considerably lower load limit.

In the construction of levees on similar foundations, the expedient of driving sheet-piles to prevent lateral movement or "bulging" of the muck has been adopted in local practice although it has been found that this is necessary only in extreme cases. Just as good results usually can be secured, and at much less cost, by constructing the levee in multiple layers and by adopting a design that will tend to increase the resistance of the soil to horizontal stresses. Fig. 37 illustrates the section and the method which the writer has used successfully in the construction of levees on exceptionally soft muck lands. The levee section was controlled by the limit of reach of available dredging equipment, as it is seldom practicable to secure a dredge in this section with a reach from the side of the hull in excess of 60 ft. By the use of longer boom dredges, such as were advocated by the writer a number of years ago,* a material increase in height of levees in such soils should be possible. A special type of dredge, which has been in use in California for many years, is well adapted

* "The Selection and Operation of Dredges", *Engineering Record*, December 16, 23, and 30, 1916.

to levee building in soft soils or under other conditions requiring an exceptionally long reach. Following are the principal dimensions of this dredge: Width of hull, 70 ft.; length of hull, 140 ft.; maximum reach from side of hull, 180 ft.; and nominal capacity of bucket, 6 cu. yd.

Repeated experiments have shown that both the spread base and the construction of levees in multiple operations have a beneficial effect in increasing the stability of muck and soft clay soils. With regard to the spread base, the gradually increasing weight, from toe of levee toward the crown, apparently compresses the muck in the zones subjected to only partial loading and renders it more resistant to lateral movement. The gradual application of the load in multiple layers permits the excess water to be expelled from the subsoils and increases their supporting power.

In the lands referred to, there is a great variation in the weight of materials entering into levees, due to different proportions of muck top-soil and clay subsoil. There also is a wide range in weight of the muck soils, some containing a considerably larger proportion of silt than others. As would be expected, the muck soils dry out rapidly after being placed in embankments, above water level, but the "sharkey" clay gives off its water content very slowly. Excavations into clay levees more than two years old have shown a "gummy" condition at points 5 ft. above soil-water level.

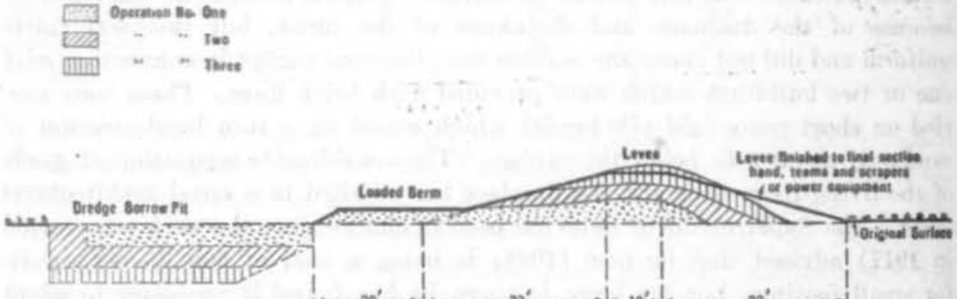


FIG. 37.—CONSTRUCTION OF A LEVEE BY MULTIPLE OPERATIONS.

While concentrated loads, such as those of embankments, may result in failure of the lower strata by a horizontal movement, fills over large areas may be constructed to a considerable height on muck soils with no danger of settlement beyond that caused by the squeezing out of some of the water. The writer has had occasion to make many hydraulic fills over soft muck lands, the added material frequently being clay or sand, but he has never experienced difficulty from excessive subsidence in such cases, provided the fills extended to hard ground or were tapered off gradually at the edges, thus securing the gradual loading mentioned in the foregoing as being desirable for levee construction in similar soils.

Most engineers who have had experience in the construction of railroad embankments across river bottoms have known of localities where embankments 6 to 10 ft. in height could be built without complications, but where a fill of 15 or 20 ft. in height would subside (sometimes suddenly). This subsidence usually would be accompanied by an upheaval of the natural surface, 20 to 50 ft. beyond the toe of the slope. Two such phenomena occurred during the construction of the Illinois Central Railroad from Fort Dodge, Iowa, to Omaha, Nebr., in 1900, one at a point about ten miles north of Council Bluffs, Iowa, and the other in the South Omaha cut-off line, near the bluffs in East Omaha. In the latter case service on the near-by tracks of the Missouri Pacific Railroad was interrupted by the upheaval.

D. P. KRYNINE,* M. AM. SOC. C. E. (by letter).—The author's works† have opened a new era in the study of soils for engineering purposes. Investigations of 10 or 15 years ago generally dealt only with sands, with no consideration for physical properties. Owing to the studies of Professor Terzaghi, the search has penetrated into the interior of clays and sands, the physical properties of which are now being studied by the civil engineer. The present generation is witnessing the birth of a new science, which may be called, "Engineering Soil Science", or "Engineering Soil Theory", and which represents applied soil physics and soil mechanics. It is similar to agricultural soil science; but the latter, by considering principally the influence of water and air on the soil, leaves unstudied the load factor, which is the most important for a civil engineer. Therefore, the establishment of a new science has been inevitable.

Slab Foundations.—In general, the writer agrees with the following statements made by the author:

(1) For soils with great cohesion the settlement produced by a given load increases in direct proportion with the diameter of the loaded area (Fig. 2).

(2) The distribution of the soil reactions over the base of a rigid slab is not uniform. The pressures are equal to zero at the edge of the slab and greatest at the center (Fig. 3).

(3) The settlement of a building is due to consolidation of the soil and to the lateral flow (Fig. 5).

However, there are some objections to each of these conclusions.

First.—Attention is called to a paper by Dr. Ing. Koegler,‡ in which it is stated that the settlement produced by a given load seems to increase directly in proportion with the diameter of the loaded area only to a certain value of this area, after which it becomes more or less constant. In the opinion of Dr. Koegler the elastic condition under a great slab does not permit lateral flow, and the vertical settlement takes place as if the supporting soil were confined laterally. Naturally, the opinion of Dr. Koegler, which seems to be based on his experiments, must be verified.

It also must be noted that his experiments were performed with sands, that is, cohesionless soils. Therefore, the results cannot be generalized until a greater number of experiments with different soils are made; but the example of the settlement of the Standard Oil Building in San Francisco, Calif., leads the writer to believe in these theories. Under a load of 4 800 lb. per sq. ft., the average settlement of the bearing plate was 0.10 in. The settlement of the building should be 152 × 0.10 = 15.2 in.; but the actual settlement was only 2 in. If the soil were homogeneous, this fact must be con-

* Prof. of Highway Eng., Moscow Superior Technical School and Moscow Inst. of Transportation Eng., Moscow, Union of the Socialistic Soviet Republics.

† "Erdbaumechanik," Vienna, 1925, and others.

‡ "Die Belastung des Baugrundes," *Der Bauingenieur*, October 27, 1927.

sidered as a proof of Dr. Koegler's statement, but phenomena considered in practice are not as simple and schematic as those studied in a laboratory.

In the example cited the soil was not homogeneous, the settlement of the bearing plate ranging from 0.04 to 0.17 in. The difference is not very great, but in any case there were hard and soft spots in the soil, so that the process of settlement was complicated by the presence of the hard spots which probably produced negative vertical forces acting on the slab. The writer thinks that in the given case and in similar ones it would be better not to deal with the average settlement of the bearing plate, but to elaborate a method of slab design, taking into account the lack of uniformity in the soil.

In regard to the settlement of the caissons at the Chicago Union Terminal, in Chicago, Ill., the writer is not in a position to give a definite opinion on the subject inasmuch as he does not have the description of the work at his disposal. Therefore, he does not know how, where, and when the settlement of the caissons was measured. However, he believes that a caisson cannot follow the law of proportionality of the settlement to the diameter of the loaded area because of the friction against the soil.

Let,

H = the height of the circular caisson, in feet (at the Chicago Union Terminal, $H = 60$);

d = the diameter of the circular caisson, in feet;

f = the average frictional force, in pounds per square foot, of the superficial area of the caisson; and,

q = the load, in pounds per square foot.

Then, the resultant vertical force, q', would be not q, but:

$$q' = q - \frac{f\,H\,\pi\,d}{\frac{\pi\,d^2}{4}} = q - f\,\frac{4\,H}{d} \qquad (22)$$

Equation (22) is an approximate formula which shows that the resultant vertical force acting per unit of the loaded area increases with the increase of the diameter of the caisson. Therefore, the settlement of a large caisson seems to be more considerable than that calculated by Equation (2) not only because of the direct influence of the diameter, but also because of the great resulting force acting on a unit of loaded ground area in comparison with that acting on a smaller one.

Second.—If a slab that is considered uniformly loaded is placed on top of the ground in an excavation, a reaction of the soil develops. Consider only the vertical component of this reaction. The vertical reaction may be confined directly beneath the loaded area (as shown in Fig. 3); or it may be effective beyond this area (see Fig. 38). In the latter case a condition must be satisfied, such that,

$$P + Q_1 + Q_2 = Q_0 \qquad (23)$$

The case shown in Fig. 38 evidently corresponds to the upheaval produced by the vertical component of the reaction. Since it is admitted that the bulging is due to the lateral flow only (Fig. 5), it must be concluded that: $Q_1 = 0$; $Q_2 = 0$. This being the case the derivative of the distribution curve,

$y = f(x)$, at the edge of the slab, which represents the ratio of an infinitely small increase of the reaction force to the corresponding increase of the abscissa beyond the edges, must be equal to zero:

$$f'(b) = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which, $2b$ is the width of the slab (Fig. 39). As a matter of fact some soil investigations indicate that Equation (24) must be satisfied.*

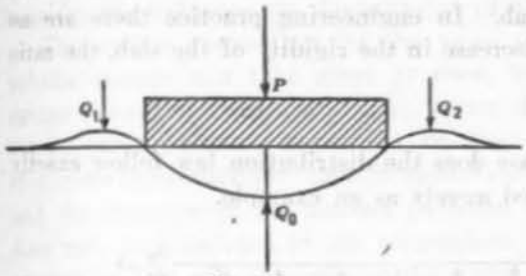


Fig. 38.

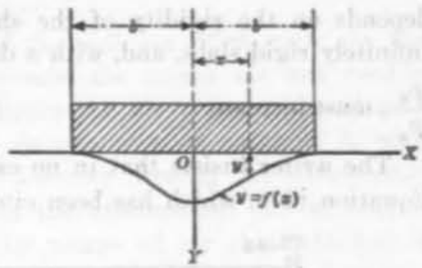


Fig. 39.

As to the curve of pressure distribution, the writer's opinion is that it cannot have a parabolic shape for the following two reasons:

(*a*) The ordinates of a parabola may be calculated accurately according to the equation:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

in which, y_0 is the greatest ordinate in the center; x is the distance from the center of the slab; and $2b$ is the width of the slab. The first derivative of y in Equation (25) is:

$$\frac{d\,y}{d\,x} = -\,y_0 \frac{2\,x}{b^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

which is not equal to zero, when $x = b$.

In order to satisfy the conditions in Equation (24), the shape of the distribution curve may be modified slightly, for instance, so that:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right)^m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

For any value of $m > 1$, and for $x = b$, the derivative of this function is equal to zero:

$$\frac{d\,y}{d\,x} = -\,\frac{2\,m\,x\,y_0}{b^2}\left(1 - \frac{x^2}{b^2}\right)^{m-1} \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Fig. 40 represents two curves placed side by side for comparison. Equation (25) expresses the left half and Equation (27) the right half. Equation (27) recalls that of the error function, or, better, of the second curve of Pearson.†

(*b*) The area of the curve of pressure distribution, as drawn on the author's Fig. 3, represents the load on a strip of slab 1 ft. wide. If the average pressure on the slab is p_a, it may be written:

$$2\,b\,p_a = \frac{2}{3} \times 2\,b\,p_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

* Koegler, "Uber die Verteilung des Bodendruckes unter Gründungskörper," *Der Bauingenieur*, February 5, 1926.

† *Philosophical Transactions*, Royal Soc. of London, Series A (1895), Vol. 186, Pt. I, pp. 360 and 372; also, Fig. V, p. 365.

and, furthermore,

$$p_0 = \frac{3}{2} p_a \cdots\cdots\cdots\cdots\cdots\cdots\cdots (30)$$

Equation (30) signifies that the pressure at the center of the slab is always 50% greater than the average. In the writer's opinion the ratio, $\frac{p_0}{p_a}$, depends on the rigidity of the slab. In engineering practice there are no infinitely rigid slabs, and, with a decrease in the rigidity of the slab, the ratio $\frac{p_0}{p_a}$, must increase.

The writer insists that in no case does the distribution law follow exactly Equation (27), which has been cited merely as an example.

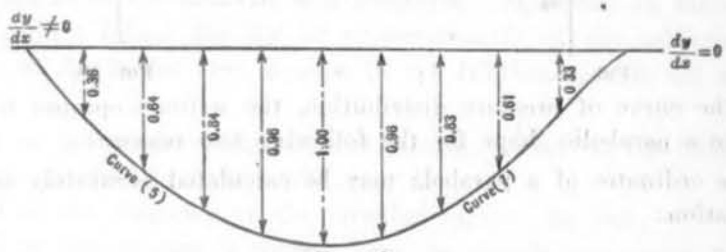


FIG. 40.

Third.—The writer believes that the two parallel lines corresponding to two principal sources of settlement (see Fig. 5) are not really parallel, and that the settlement at the center of a slab is greater than at its edges. For this belief he presents the following reasons: (1) In the middle of a building, because of symmetry, there can be no lateral flow; hence, the deflection in the middle of a slab depends only on the vertical component of the pressure; (2) the vertical pressure at the edge being equal to zero, there can be no settlement due to the vertical pressure; hence, the deflection at the edge of a slab depends only on the lateral flow; (3) if the curve of deflection due to vertical pressure (compression) be drawn, it will represent a curve with ordinates decreasing from the center to zero at the edges; on the contrary the ordinates of the curve of deflection due to lateral flow must increase from zero at the center to a certain value at the edges; and (4) if Koegler's theories are correct, the lateral flow does not increase from the center toward the edges of the slab during the entire time, but stops when $x < b$ (see Fig. 39). Hence, the settlement at the center of the slab seems to be greater than at the edges. The settlement of small rigid disks, 3 or 5 ft. in diameter, naturally cannot give any idea of the difference of settlement in the center and on the perimeter of the slab.

Generally, a foundation slab is a statically indeterminate structure. The distribution of soil reaction depends on the rigidity of the slab. Therefore, bending moments and shearing forces vary with the change in its rigidity. The distribution of soil reactions is intimately bound to the deflection of the slab, although Hooke's law is not applicable to soils. Hence, the soil deflections, reaction distribution, and stresses in the slab are interrelated. This idea is not new. In 1914, Brugsch and Briske published a paper on the influence of the deflection of the foundation on the design of statically indeterminate structures.*

Entrapped or "Pinched" Air.—Professor Terzaghi greatly merits the approval of the Engineering Profession for demonstrating that clay and water form an intimately connected system. Hence, the mechanics of clay, without considering the influence of water, practically became absurd. The writer desires to show that in some cases not only the water, but also the air, has a considerable influence on the behavior of clay.

The author states that "if the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface". The writer thinks the air present in soil may act in three different ways: (1) So that, in one way or another, it communicates to the surrounding atmosphere and assumes directly the temperature and the pressure of the nearest particles of the atmosphere; (2) so that it does not communicate to the atmosphere by means of air channels, but is pressed against soil particles by the water in the soil; this is analogous to a bubble in a spirit-level, and may be termed entrapped or "pinched" air; such air bubbles may often be microscopic in size; and (3) as adsorbed air which sticks to the particles of soil.

The first of these conditions is the one recognized by Professor Terzaghi. As to the second, which naturally must have great influence on the elastic properties of soils, the writer is not sure of its presence in the deep layers of the ancient natural clays. As to relatively modern deposits, however, and especially the artificial fills, there is no doubt as to the presence of entrapped air. Therefore, an investigation of the effect of air and, if necessary, the introduction of the element, "air", into the system, "soil ÷ water", seems to be of definite importance.†

The writer believes that a study of capillarity as applied to water in the soil, leads to the conclusion that there is also a certain quantity of entrapped air.‡ If the clay pores are not completely filled with water, there must be air present. Furthermore, because clay pores represent excessively thin clefts, it is very difficult to imagine that the air may easily escape from them. At any rate, the influence of the entrapped air on the elastic properties of soils should be investigated.

Pile Foundations.—It is interesting to consider the author's views on pile foundations in reference to the pile-driving formula of Professor Guersevanoff, which is widely used in Russia.§ Professor Terzaghi's notations are used with the following additions:

r = rebound of the hammer after the blow; and

a = coefficient of the lost work, depending on the nature of the pile, method of its driving, etc.

* Brugsch and Briske, "Einfluss der Nachgiebigkeit des Baugrundes auf die Berechnung äusserlich statisch unbestimmter Bauwerke," *Beton und Eisen*, 1914, pp. 15, 53, 85, and 183.

† D. P. Krynine, "On the Technical Rôle of Air in Soils" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

‡ D. P. Krynine, M. V. Ivanova, and T. A. Ovsiannikoff, "On the Capillary Rise" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

§ *The Cement* (Russian Magazine), 1916; First issue, p. 2; Second issue, p. 73. This magazine is no longer published.

The work done by the drop of a hammer consists of three parts: (1) Pile penetration; (2) elastic deformation; and (3) lost work; for instance, increase in temperature, etc. This may be expressed:

$$R\,h = [A] + [B] + [C] = Q_d\,s + R\,r + a\,R\,h \quad \text{.......(31)}$$

In his analysis, Guersevanoff ignores the term, $R\,r$, so that Equation (31) becomes:

$$(1 - a)\,R\,h = Q_d\,s \quad \text{.......(32)}$$

The coefficient, $1 - a = b$, depends on the value of Q_d and the cross-section of the pile, F; that is,

$$b = f\left(\frac{Q_d}{F}\right) \quad \text{.......(33)}$$

By making various assumptions he comes to the conclusion that the shape of the curve of Equation (31) is a hyperbolic one:

$$b = \frac{R + m^2\,G}{R + G} \times \frac{1}{1 + \frac{Q_d}{F}\,n} \quad \text{.......(34)}$$

The coefficient, n, depends both on the nature of the pile and on the method of its driving. For instance, for wooden piles without followers, $n = 10\,\frac{K\,g}{c\,m^2}$; for concrete piles with followers, $n = 0.5\,\frac{K\,g}{c\,m^2}$. Definitely, the formula, in kilograms and centimeters, is:

$$Q_d = -\,\frac{n}{2} + \sqrt{\frac{n^2}{4}\,F + n\,\frac{F}{s}\,R\,h\,\frac{R + 0.2\,G}{R + G}} \quad \text{.......(35)}$$

This formula has somewhat the appearance of Equation (3), but Equation (35) neglects partly the length of the pile, and the modulus of elasticity of the pile material enters implicitly into the value of n.

The author's concept of hydrodynamic stresses induced in soils is truly brilliant, but there is some question as to the squeezing of water out of soil beneath the point of the pile. Suppose the pile is driven into clay containing 50% of voids which are completely filled with water. Then, the thickness of the film of water around the pile must be, approximately (deformations of the pile disregarded):

$$T = \frac{0.5\,\frac{\pi\,d^2}{4}\,h}{\pi\,d\,h} = \frac{d}{8}$$

in which, d is the diameter of the pile. In Fig. 41 the full line represents the position of the pile before the blow, and the dotted line the position after the blow. Thus, the film of water around a 10-in. pile would be 1¼ in. thick, which seems to be too much. Therefore, the writer thinks that the vertical force transmitted by the pile: (a) Compresses the soil, thereby causing the pile to move downward and producing a certain lateral flow of hard particles; and (b) acts on the soil water just as in any case of clay loading.* In Fig. 42 the part of the vertical force acting on the water is divided into two components:

* Terzaghi, "Erdbaumechanik," Abschnitt 12, 20, and others.

(1) The horizontal force, moving water away from the pile and thus compressing the entrapped air; and (2) the force parallel to the surface of the pile. The reaction against this component of force is what causes the film to form on the surface of the pile. The posterior absorption of the film by the soil may occur, in the writer's opinion, only if there is air in the voids. The layers of clay in Fig. 42 are assumed to be approximately horizontal.*

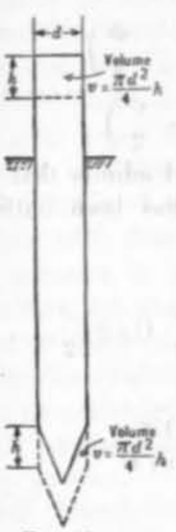


FIG. 41.

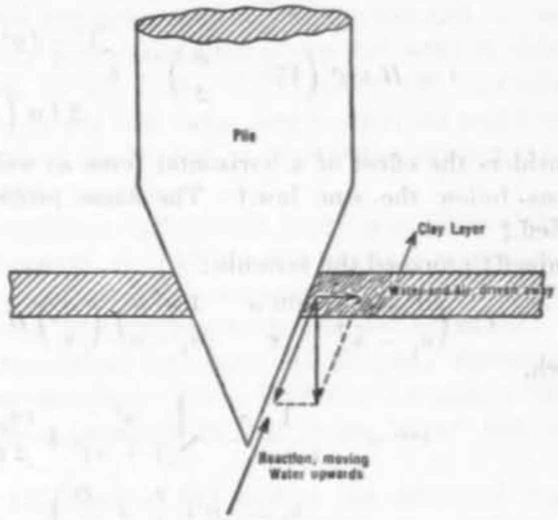


FIG. 42.

Russian Contribution to the Science of Foundations.—The author deals with the present status of the science of foundations as well as with its future.

Russian engineers have made some noteworthy contributions to the theory of pile-driving.† They have also published some works dealing with the determination of the depth, t, of the foundation. Through a paper by Professor Kjürdümoff‡ three formulas, those of Paucker and Jankowsky, came from Russia into European literature.§

Let,

t = the depth of the foundation;

H = the height of a sand prism, with base area and weight equal to those of the building; and,

ϕ = the angle of the internal friction.

Then, Paucker's formula is:

$$t = H\,tg^4\left(45^\circ - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots (36)$$

The first formula of Jankowsky is:

$$t = \frac{H}{2}\,tg^4\left(45^\circ - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots (37)$$

* D. P. Krynine, "Elementary Proof of Shale-Likeness of Clay Particles," *Public Roads*, January, 1928.

† Jankowsky, "On the Resistance of Pile Foundations," *Journal*, Ministry of Ways of Communication, 1887; Guersevanoff, the previously cited paper, 1916; Dmohovsky, Papers on the influence (a) of the geometric shape of piles; and (b) of the eccentricities of loads, Works of the Moscow Inst. of Transportation Eng., 1927. There are also other papers by the same author and his textbook on "Engineering Foundations," 1928.

‡ Kjürdümoff, "Zur Frage des Widerstandes der Gründungen auf natürlichen Böden," *Der Civilingenieur* 1892.

§ The derivation of these formulas is to be found in Terzaghi's "Erdbaumechanik," 1925, p. 229.

and his second formula is:

$$H = 2\,t\,\frac{tg^2\left(\frac{45^\circ + \phi}{2}\right)}{tg^2\left(\frac{45^\circ - \phi}{2}\right)} \qquad (38)$$

Belzetsky's formula* is:

$$t = H\,tg^4\left(45^\circ - \frac{\phi}{2}\right) - b\,\frac{1 - tg^4\left(45^\circ - \frac{\phi}{2}\right)}{2\,tg\left(45^\circ - \frac{\phi}{2}\right)} \qquad (39)$$

He considers the effect of a horizontal force as well and admits that the soil reactions follow the sine law.† The same problem has been outlined by Prokofieff.‡

Miniaeff§ proposed the formula:

$$t = \left(\frac{1+n}{n_1 - n}\right)\left(\frac{\sin\alpha}{\pi} - \frac{1-n}{n_1 - n}\right)\left(\frac{\alpha}{\pi}\right)H - \frac{b}{2}\,C\,tg\,\frac{\alpha}{2} \qquad (40)$$

in which,

$$\cos\alpha = \frac{1}{1+n} - \sqrt{\frac{n^2}{(1+n)^2} + \frac{(n_1 - n)\,\pi\,\alpha}{2\,(1+n)\,H}}$$

$$n_1 = tg^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

and,

$$n = tg^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

Equation (40) is based on the following hypotheses: (1) The distribution of stresses in soils follows the same laws as in a solid; (2) "sets" in a dry substance are not functions of forces; and (3) stresses in dry substance are determined only by elastic deformation. The problem consists in determining if, under a given set of forces, an elastic deformation of the soil takes place. If not, a supplementary set of forces must be applied, namely, the weight of the soil above the level of the bottom of the foundation.

Pouzyrewsky‖ considered a condition of the soil when all the deformations are elastic only, and proposed the formula:

$$t = H\,\frac{C\,tg\,\phi + \phi - \frac{\pi}{2}}{C\,tg\,\phi + \phi + \frac{\pi}{2}} \qquad (41)$$

In his paper on the theory of dry substances, Nemiloff¶ developed a formula which, like the formula of Belzetsky, takes into account the width, b, of the foundation. It is worthy of note that, in 1913, he proposed an apparatus

* Belzetsky, "Statics of Constructions," 1914, p. 95.

† *Loc. cit.*, p. 83.

‡ Prokofieff, "Theory of Constructions," 1928, Vol. II, p. 258.

§ Miniaeff "On the Distribution of Stresses in Dry Substances," Tomsk, Siberia, 1915, p. 85.

‖ Pouzyrewsky, "Foundations Design" (in Russian), 1923, p. 38.

¶ Nemiloff, "Contribution to the Theory of Dry Substances," *Journal*, Ministry of Ways of Communications, 1913, Issue 9, pp. 126–148.

for determining the coefficient of internal friction* based on practically the same idea as that for the apparatus used in 1917 by the Society's Special Committee to Codify Present Practice of the Bearing Power of Soils for Foundations, etc.†

This fact shows that there is a need for a scientific center to look after the technical literature in civil engineering the world over, in order to avoid the duplication of work and to utilize the technical ideas of different nations.

Generally, Russian civil engineers seem to like the soil and dry substances theory, and there are many papers and discussions dedicated to this subject. The writer calls attention to the works of Prilejaeff‡ and Spalving.§ The latter gives a new theory; he not only takes into account the equilibrium of a soil prism as Paucker and others do, but he studies the stresses that develop in the interior of this prism as well.

Since 1922, Russian engineers (principally highway engineers) have given their attention to the necessity of studying the physical properties of soils. The writer was the first in Russia who expressed in printed form the necessity of co-operation between highway engineers and soil scientists.‖ In 1923, the Russian Highway Department and, later, its Highway Research Bureau, began its activities along this line. When Professor Terzaghi's "Erdbaumechanik" appeared in 1925, it became clear that a new "slant" had been given to this science.

Soil Classification.—Engineering soil science has advanced considerably in recent years; but a definable system of soil classification had not appeared until this paper was presented. The research engineer is in a difficult position when receiving a sample of soil from a job. He may tell the builder as much as he wishes about the internal friction of the soil, about its compressibility, its permeability, etc.; but the only question asked is, "How many pounds pressure will this soil support?"

A laboratory cannot produce the answer to this question categorically. The most practical solution of the problem seems to be to elaborate a combination of simple laboratory experiments with a simple routine test in the field.

S. P. WING,¶ Assoc. M. Am. Soc. C. E. (by letter).—The writer is connected (1928), with the design of a dam that is to be located at what is known as Site No. 7 at Bridge River in British Columbia. This work is at the lower end of a 30-mile glaciated valley that is about ½ mile in width, with surrounding hills of granites and schists. The actual dam site is about 800 ft. in width between granite outcrops. Borings showed that the rock on either side pitched

* Nemiloff, "Contribution to the Theory of Dry Substances," *Journal*, Ministry of Ways of Communications, 1913, Issue 9, p. 140, Fig. 3.

† *Proceedings*, Am. Soc. C. E., August, 1917, Papers and Discussions, p. 1179; and, further, Fig. 1, p. 1174.

‡ Prilejaeff, "Contribution to the Theory of Pressure on Sustaining Walls" (in Russian), 1913.

§ Spalving, "A New Theory of Dry Substances," *Water Transport* (Russian magazine), February, 1927.

‖ D. P. Krynine, "American Methods of Earth-Road Building and Their Application in Russia," *The Messenger of Technics and Economics* (Russian magazine no longer published), January, 1922.

¶ Engr., Constr. Dept., British Columbia Elec. Ry., Vancouver, B. C., Canada.

sharply downward, and drilling in the center was stopped when bed-rock was not reached at a depth of 225 ft.

As borings showed a heavy bed of clay lying about 30 ft. below the surface, numerous other test holes were sunk, and a clear conception of the underlying beds was obtained. At the neck of the valley are beds of coarse talus and gravel, perhaps 200 ft. deep as a maximum, resting on bed-rock. Overlying these strata is a continuous bed of glacial or post-glacial clay perhaps 10 ft. thick at the valley neck, but rapidly deepening to a thickness of 90 ft., or more, up stream and forming an impervious blanket in contact with rock on either side. Overlying this are recent deposits of silt and sand and gravel from 20 to 30 ft. thick, forming the present valley floor.

Tests were made with a view to determining the bearing power and settlement of both the clay and the overlying beds. The amount of settlement is considered of great importance because the project calls for a rock-fill dam with concrete facing and a water-tight apron extending up stream to a point where cut-off can be made into deep clay. The design of this lining will have to provide for any differential settlement.

The soil-pressure testing apparatus was similar to that recommended by the Society's Special Committee on the Bearing Value of Soils for Foundations, etc.,* but was designed for a maximum load of 60 tons, so that loading areas of 1, 2.5, and 4 sq. ft. could be used. In general, the apparatus proved satisfactory, but in future tests an oil-jack would probably be substituted for the screw-jack which was used as a fulcrum, because the force required to move the latter in leveling induced extra settlement of the test post.

Test Pit No. 3.—The first tests were made at Test Pit No. 3. Fig. 43 shows the log of the test hole at this location. The material was sand and was composed of sharp angular fragments, largely quartz. Eight feet of the top-soil was removed and the tests on different areas were made in the bottom of the open pit on the undisturbed bed of sand. The mechanical analysis of this sand was as follows:

Sieve No.	Percentage retained.
4	1.4
8	4.2
14	6.5
28	15.8
48	42.5
100	26.9
200	2.2
200	0.5†
Total sample	100
Percentage of mica	0.6

This sand was saturated and the pit required sheet-piling and continuous pumping to keep it unwatered. However, during tests the pit was allowed

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 529, Fig. 1.
† Percentage passing.

to fill with water. Figs. 43 and 44 give the results of the loading tests. An attempt was made to measure the flow of the surrounding soil during the loading of the 4-ft. square plate. Stakes were driven 18 in. into the ground at a distance of 1 ft. from the loaded area. These did not show consistent results, but at the end of the test they had risen an average of 0.015 ft. This settlement may have been due to the effect of unwatering.

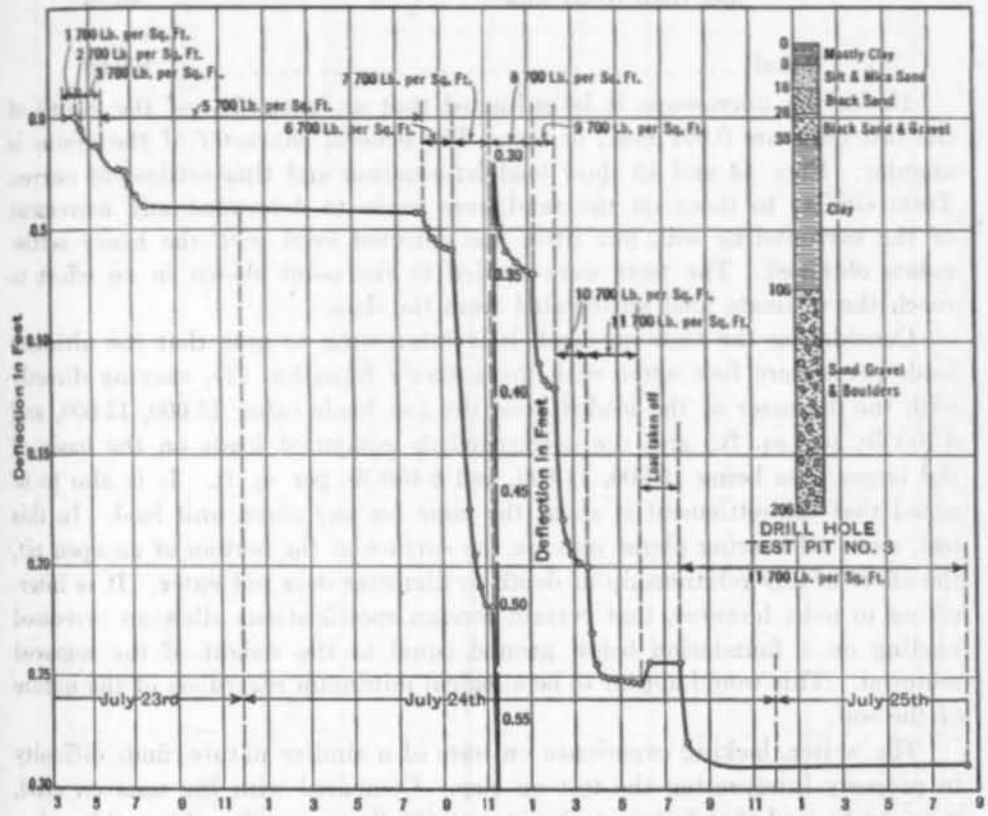


FIG. 43.—TIME SETTLEMENT CURVE FOR SILT AND GRAVEL.

Test Pit No. 2.—Fig. 45 shows the test-hole log at Test Pit No. 2. A pit, 5 by 7 by 13 ft. deep, was sunk, sheet-piled, and pumped dry to a depth 3 ft. below the area being tested and the test was made on a 2-ft. square area of undisturbed clay in place.

The material was clay and was stiff and difficult to spade, but under the men's feet it became plastic. Its characteristics follow:

Weight of clay as sampled, in pounds per cubic foot	112
Percentage of water (in terms of weight of dry material in 1 cu. ft.)	40
Percentage of lower limit of liquid state	42.5
Percentage of lower limit of plastic state	36.6

A change in moisture content from 37 to 26% causes shrinkage of 12% in original volume.

The mechanical analysis by elutriation is as follows:

		Percentage.
Particles,	0.075 to 0.050 mm.	0.063
"	0.050 to 0.025 mm.	1.32
"	0.025 to 0.010 mm.	16.43
"	0.010 to 0.005 mm.	35.52
"	less than 0.055 mm.	45.25
Total		100.00

Under the microscope it is estimated that at least 25% of the grains of the last class are 0.002 mm., or less. The general character of the grains is angular. Figs. 44 and 45 show load-deformation and time-settlement curves. Tests similar to those on the sand were made to determine any movement of the surrounding soil, but little was detected even with the heavy settlements obtained. The tests were carried to the point shown in an effort to reach the ultimate load anticipated from the dam.

Considering the tests on sand, it is interesting to note that the ultimate loads per square foot agree with the author's Equation (2), varying directly with the diameter of the loaded area, the test loads being 15 000, 13 000, and 6 700 lb. per sq. ft., and the corresponding computed loads on the basis of the larger area being 15 000, 11 800, and 8 400 lb. per sq. ft. It is also to be noted that the settlement is about the same for any given unit load. In this test, since all bearing plates were on the surface at the bottom of an open pit, the effect of any relationship of depth to diameter does not enter. It is interesting to note, however, that certain foreign specifications allow an increased loading on a foundation below ground equal to the weight of the removed material. This would appear to be a logical minimum regardless of the nature of the soil.

The writer, lacking experience on tests of a similar nature, finds difficulty in properly interpreting the test on clay. Compared with the tests on sand, it might be said that failure took place at 500 lb. per sq. ft. After this point, any increased load caused heavy settlements but, as the curves show, those decreased with time, and practical equilibrium was established, even with as heavy loads as 12 000 lb. per sq. ft. and total settlements of 2.3 ft., the settlement in the last ten days at this load being only 0.01 ft. It is possible that with this heavy settlement some extra bearing surface of the apparatus became effective, but it is not thought that this was important. Release of the load and its re-application caused some slight additional settlement, as shown by the curve (Fig. 45). In the main it is thought that the increased supporting power of the clay was due to local consolidation under the bearing plate. That similar results will occur with a diameter of loaded area 100 times greater, seems doubtful. As the author's formulas are only applicable to loads within what might be called the elastic limit, they give no guide as to the future behavior of this material. Fortunately, in the present case, a diversion dam of small proportions is to be built first, and it is intended to make more loading tests on the exposed foundation and then to compare those

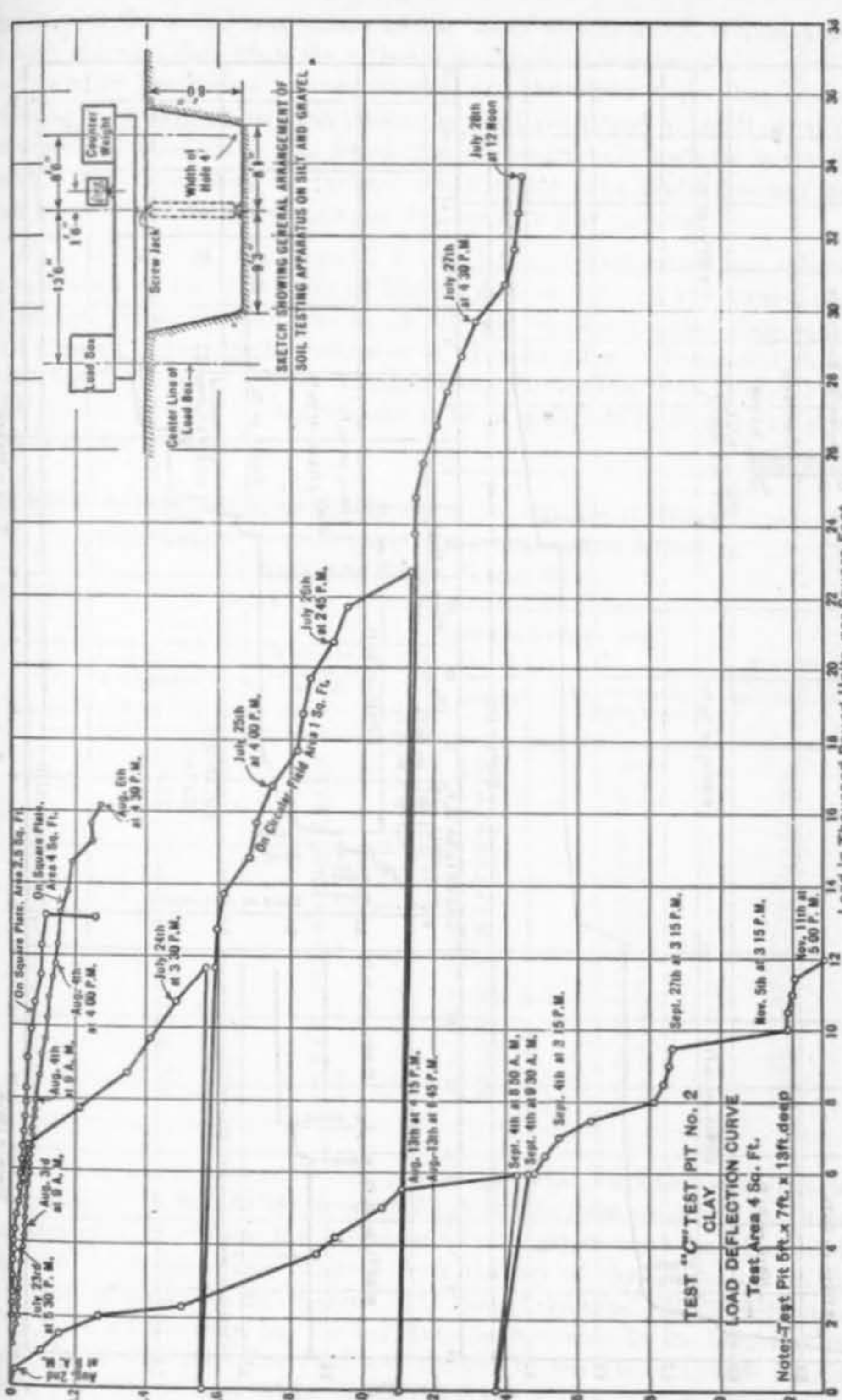


FIG. 44.—LOAD DEFORMATION TEST OF SILT AND GRAVEL OVERBURDEN.

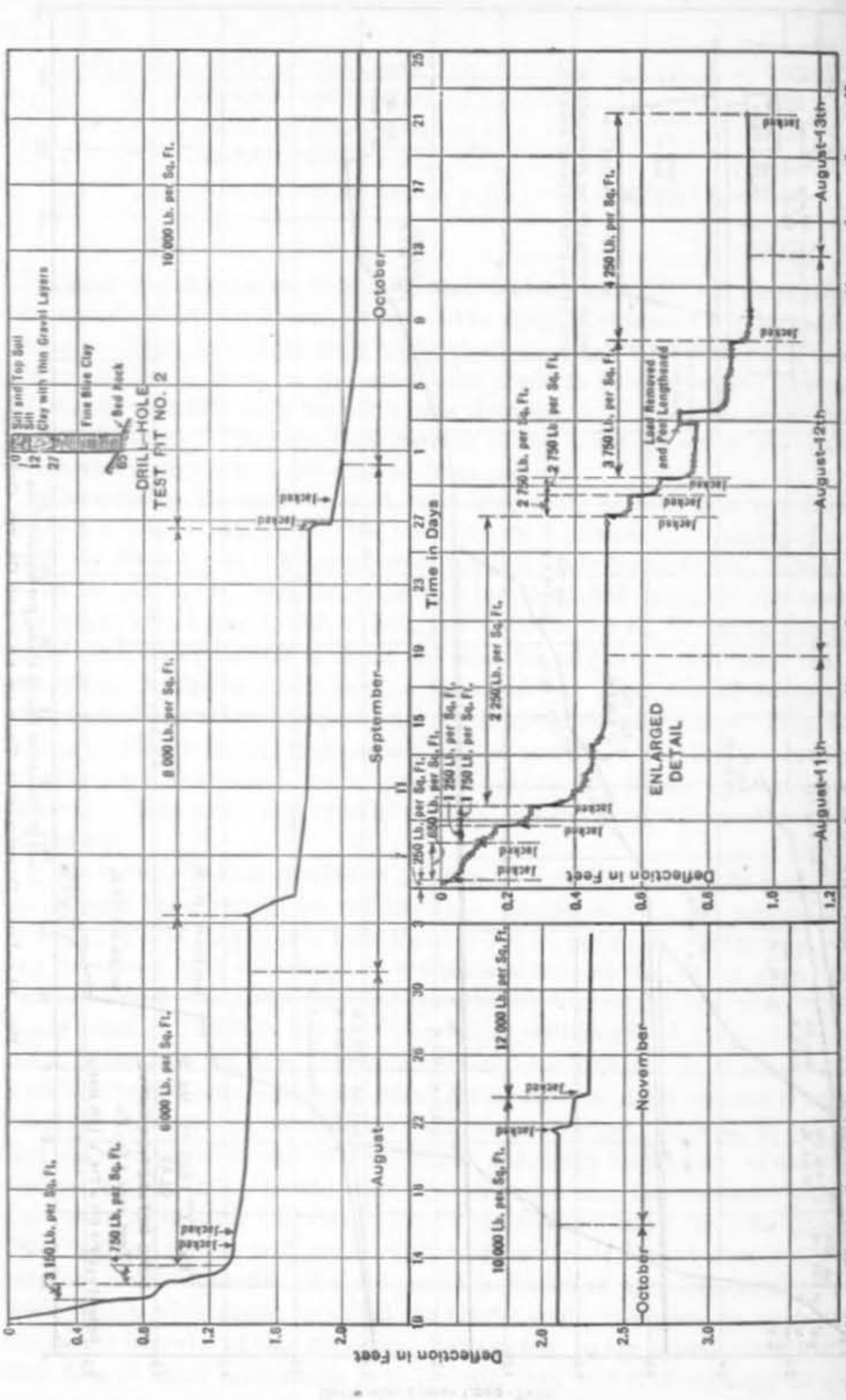


FIG. 45.—TIME SETTLEMENT CURVES FOR CLAY.

results with the actual settlements of the larger structure. It is thus hoped to have adequate data when the ultimate structure is designed.

The paper has been a provocative one, and the writer hopes that in contributing the results of his experiments, he will have helped in building up the much needed data. It is also hoped that the author will indicate any omissions in this description of the soil which would have made the data and tests more helpful to other engineers dealing with like material.

M. L. Enger,* M. Am. Soc. C. E. (by letter).—The author has referred to tests made at the University of Illinois prior to 1916 on the transmission of pressure through sand. The results of tests made since that time, which were reported in part in the discussion of "Transmission of Pressure Through Solids and Soils and the Related Engineering Phenomena",† by George Paaswell, M. Am. Soc. C. E., are thought to be of sufficient interest to be given here in greater detail.

TABLE 10.—Summary of Experiments to Determine Distribution of Pressure in Sand Ballast Arranged in the Order in Which the Tests Were Made.

Series.	Depth of ballast, in inches.	Diameter of plates, in inches.	Maximum Applied Load.		Reaction percentage of applied load.
			In pounds.	In pounds per square inch.	
1c	38	21	6 700	19.2	41
1c	38	21	7 500	21.4	56
1a	38	36	13 650	13.3	57
2a	30	36	21 800	21.4	86
2b	30	30	15 500	21.8	85
2c	30	21	11 900	34.0	93
2d	30	13.5	4 150	28.8	77
3a	24	36	21 800	21.4	102
3a'	24	36	21 800	21.4	96
3b	24	30	19 600	27.6	98
3c	24	21	13 600	38.9	124
3d	24	13.5	6 700	46.5	116
4a	18	36	24 000	23.7	116
4b	18	30	21 800	30.7	130
4c	18	21	13 600	38.9	186
4d	18	13.5	6 700	46.5	150
5a	12	36	24 000	23.7	118
5b	12	30	11 900	16.7	132
5c	12	21	11 600	33.2	143
5d	12	13.5	5 400	37.5	125
5d'	12	13.5	5 400	37.5	112

The experiments were made on a pile of loose bank sand having a diameter at the base of 16 ft. At the greatest depth (38 in.) the sand pile had a top diameter of 6 ft. When the experiments on a given depth had been completed, 6 in. of sand were removed from the top of the pile to obtain the next depth of sand over the capsules. The base of the sand pile had a diameter of 16 ft. in all the tests, but the top diameter increased as the height of the sand pile diminished. The load was applied to the sand through circular plates, 13.5, 21, 30, and 36 in. in diameter, by a jack, and was determined

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† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

by means of a calibrated steel spring. The arrangement of the apparatus is shown in Fig. 46.

The vertical pressures transmitted through the sand were measured by means of thirty-one pressure capsules placed in six radial lines in such a manner that there was one capsule directly below the center of the circular plate and three capsules on each circle of 3, 6, 9, 12, 15, 18, 21, 24, 27, and 30-in. radius about the center. The pressure capsules were the same as those used in the experiments reported in the Second Progress Report of the Special Committee on Stresses in Railroad Track.* They were calibrated before and after the experiments were made. The arrangement of the capsules is shown in Fig. 47.

TABLE 11.—VERTICAL UNIT PRESSURES AT VARIOUS DISTANCES FROM AXIS OF APPLIED LOAD ON CIRCULAR PLATES, EXPRESSED AS PERCENTAGES OF THE AVERAGE APPLIED UNIT PRESSURE.

Series.	Depth of ballast, in inches.	Diameter of plate, in inches.	Distance from Axis of Applied Load, in Inches.										
			0	3	6	9	12	15	18	21	24	27	30
1*a*	38	36	50	49	49	42	36	29	16	14	9	8	4
2*a*	30	36	74	73	78	68	53	44	32	21	13	6	6
3*a*	24	36	111	115	86	84	68	44*	35	22	15	6	5
3*a*′	24	36	111	106	93	88	66	58	83	20	13	6	6
4*a*	18	36	160	150	132	110	80	61	31	17	10	5	6
5*a*	12	36	184	205	148	122	90	64	26	11	4	3	3
2*b*	30	30	64	64	64	53	38	32	20	13	8	5	2
3*b*	24	30	101	104	84	73	53	35	21	10	8	3	4
4*b*	18	30	136	154	114	91	64	37	15	9	4	3	2
5*b*	12	30	199	185	140	108	62	37	9	8	2	1	2
1*c*	38	21	18	17	16	11	10	6	8	1	1	0	0
1*c*′	38	21	18	16	17	17	11	12	6	3	2	2	1
2*c*	30	21	88	39	39	28	18	17	8	8	3	1	1
3*c*	24	21	72	80	63	46	24	18	6	5	3	1	3
4*c*	18	21	108	115	81	52	27	14	5	3	1	2	1
5*c*	12	21	189	141	104	64	24	10	1	0	0	0	0
2*d*	30	13.5	18	17	16	11	7	6	3	2	1	0	1
3*d*	24	13.5	37	37	28	18	9	7	2	2	1	0	0
4*d*	18	13.5	60	61	41	25	8	5	1	1	0	1	0
5*d*	12	13.5	109	102	51	15	4	1	0	0	0	0	0
1916 experiments.....	18	13.5	43	31	25	....	8	..	..	..	..	..	..
" "	12	13.5	100	64	40	....	5	..	..	..	..	..	..
" "	6	13.5	300	175	49	....	0	..	..	..	..	..	..

Table 10 shows the order in which the experiments were made and the maximum load applied in each test. The upward reaction computed from the readings of the capsules, expressed in percentage of the applied load, is also shown as a check on the accuracy of the measurements. It will be noted that the reactions are too small in the first three series of tests and too large in some of the later experiments. The error in the reaction is due to the fact that the average of readings at three points on each circle may not represent accurately the actual average pressure. Furthermore, the pressure on a capsule may be different from that which would be developed at that point if there were no capsule; that is, the pressure capsule itself has an effect on the intensity of pressure which it is to measure.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1409.

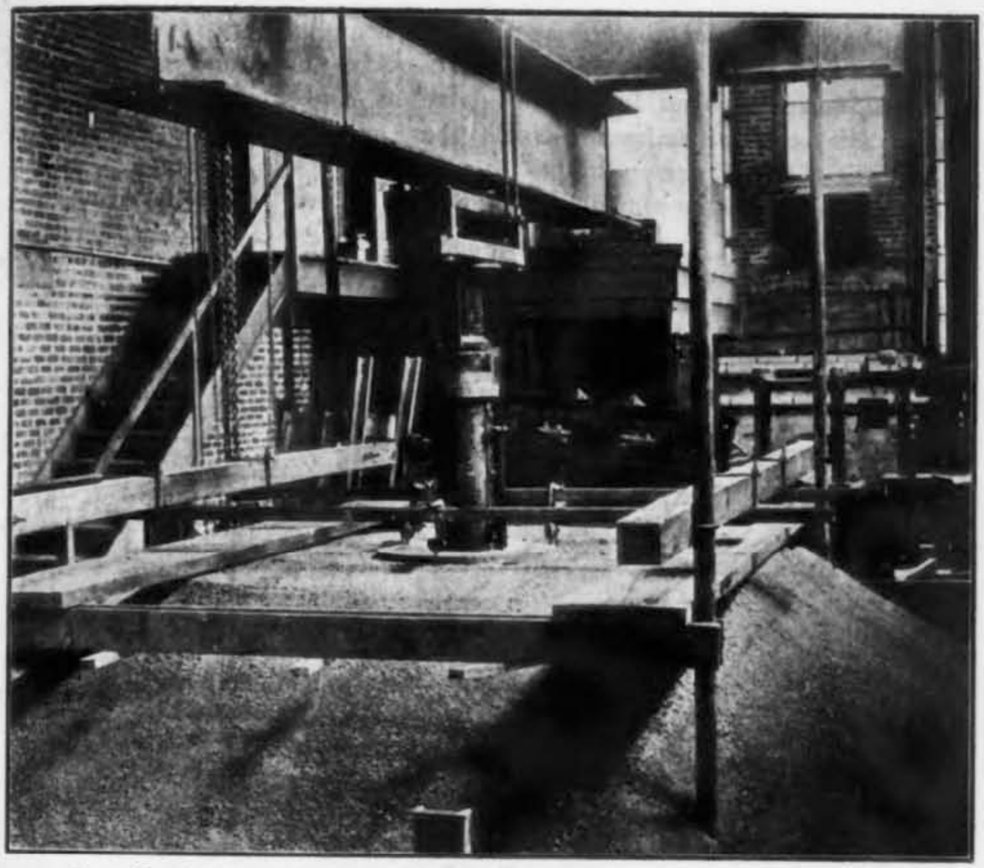


FIG. 46.—LABORATORY APPARATUS FOR TRANSMITTING LOADS THROUGH SAND.

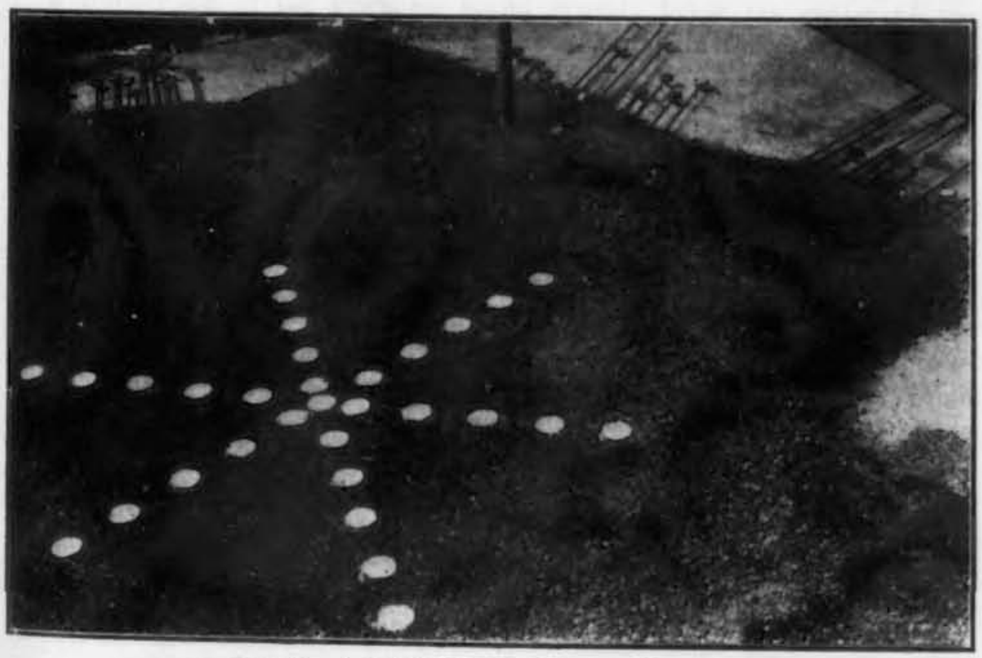
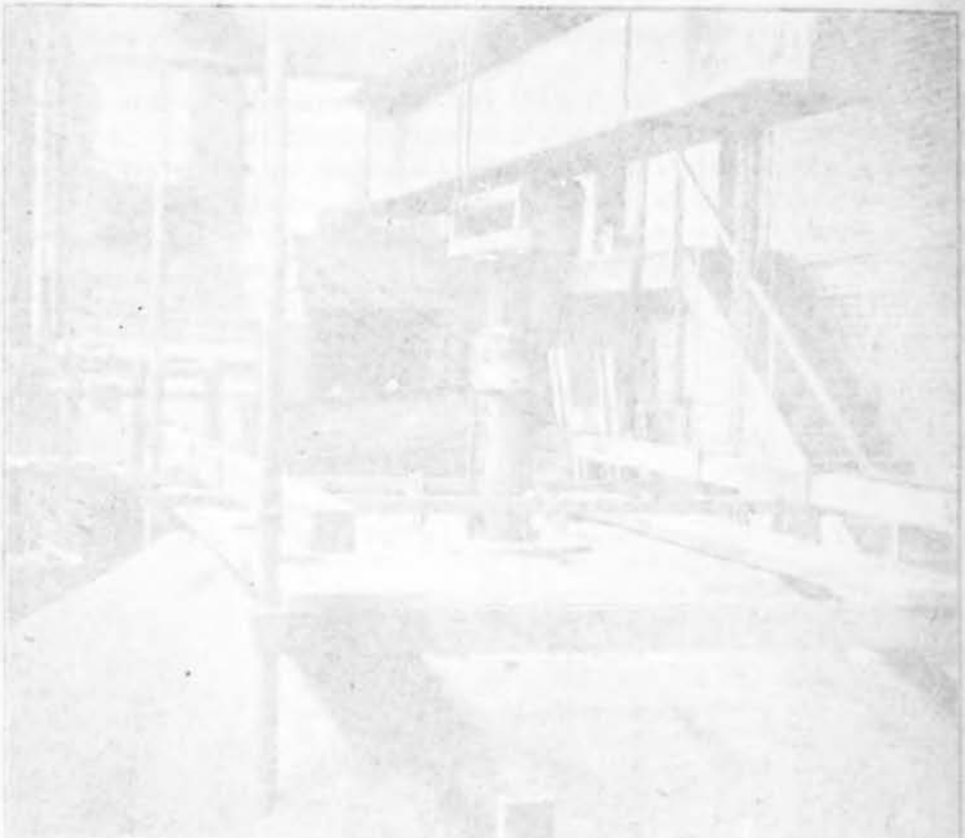


FIG. 47.—ARRANGEMENT OF PRESSURE CAPSULES.



[illegible]

[illegible]

The order of the experiments is of importance in interpreting the results because of the compacting action of the plates on the sand. This fact unfortunately was not foreseen, and in the first eight series of experiments no attempt was made to loosen the sand when the experiments on one size of plate had been completed. In the experiments beginning with Series 3*a* (Table 11), the sand was loosened before beginning experiments with a plate of another diameter.

A condensed summary of the results of the experiments is given in Table 11. The average intensity of pressure at various depths and distances from the axis of the applied load is given in percentage of the average intensity of load applied by the circular plate. It will be noted that there are many inconsistencies.

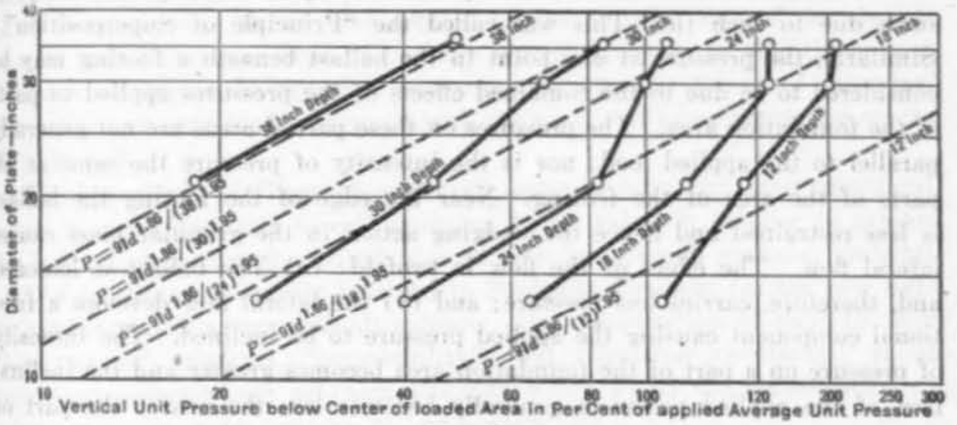


FIG. 48.

A comparison of the pressures found directly below the center of the loaded plates in these experiments with those found in the previous experiments at the University of Illinois is shown in Fig. 48. The dotted lines represent the equation,

$$P = 91 \frac{d^{1.86}}{h^{1.95}}.$$

which was deduced from the earlier experiments. The agreement is not good. It was stated in the paper in which the equation was published* that, "the equation represents roughly the results of the experiments, but it is not probable that it has general application." The difference in the results of the two sets of experiments may be explained in part by the difference in procedure in making the tests. In the experiments reported in 1916 a very high intensity of pressure was applied. The plate was forced down several inches to the level used when taking readings. As a result the sand was compacted beneath the center of the plate and was loosened beneath the edge. Such a condition may exist under heavily loaded footings on sand. These heavy pressures were not used in the later experiments because of the fear that many of the capsules would be broken. It is thought that the distribution of pressure beneath circular footings used in practice will be betweeen the ex-

* *Engineering Record*, Vol. 73, No. 4, January 22, 1916, p. 106.

tremes represented by the tests reported in 1916 and the tests represented by Table 11.

The intensity of pressure at any point in the ballast beneath a footing is not a definite percentage of the average intensity of the applied load, but is different for different degrees of compactness of the ballast and for different intensities of applied pressure. It is, therefore, not possible to draw "pressure bulbs" which have general application. The high intensities in the tests at the University of Illinois before 1916 are only to be found when the plate is carrying nearly its ultimate load.

It was shown in the Second Progress Report of the Special Committee on Stresses in Railroad Track* that the intensity of pressure at any point in ballast beneath several loaded ties may be found by adding together the pressures due to each tie. This was called the "Principle of Superposition".† Similarly, the pressure at any point in the ballast beneath a footing may be considered to be due to the combined effects of the pressures applied to parts of the foundation area. The pressures on these partial areas are not generally parallel to the applied load; nor is the intensity of pressure the same at all parts of the area of the footing. Near the edge of the footing the ballast is less restrained and hence the wedging action in the granular mass causes lateral flow. The effect of the flow is twofold: (*a*) The ballast is loosened and, therefore, carries less pressure; and (*b*) the lateral flow develops a frictional component causing the applied pressure to be inclined. The intensity of pressure on a part of the foundation area becomes greater and the inclination of the applied pressures generally becomes less the nearer the part of the area is to the axis of the applied load. The flow of the granular mass away from the center of the plate is shown by heavy radial scratches on the bottom of the circular cast-iron plates used in some of the experiments. In the experiments made for the Special Committee on Stresses in Railroad Track*, heavily loaded railroad ties on rock ballast were sometimes torn in two by the frictional forces developed. The flow of the sand during a sudden failure of the ballast beneath a loaded plate is strikingly shown by illustrations given in that report.‡

The distribution of pressure in a granular mass is most easily comprehended by a study of "lines of pressure"§ which are analogous to lines of force in a magnetic field. If a load of, say, 100 lb. is applied to a granular mass, then on every horizontal plane below the loaded area there must be an added pressure of 100 lb. If the distribution of the vertical pressure is known on a number of horizontal planes at different depths, 100 diverging lines may be drawn, each of which represents 1 lb. of pressure. In regions of high intensity of pressure the lines are close to each other. If, for example, the pressure is 30 lb. on 1 sq. in. of one of the horizontal planes, then there will be 30 lines passing through that square inch. The lines spread as the distance down

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1409.

† *Loc. cit.*, p. 1560.

‡ *Loc. cit.*, p. 1437, Fig. 9.

§ *Loc. cit.*, p. 1572.

ward increases. By subdividing a footing into small areas on which the direction and the intensity of pressure are nearly constant, a bundle of "lines of pressure" may be drawn for each of the areas and, by the principle of superposition, the intensity of pressure on any square inch of a horizontal plane may be found by counting the number of lines passing through that square inch and multiplying the number thus found by the number of pounds each line represents.

Lines of pressure for an 8-in. tie are illustrated in the Second Progress Report of the Special Committee on Stresses in Railroad Track.* The lines cross, indicating a concentration of pressure at a depth of 6 in. below the tie (found to be 163% of the average pressure by experiment). Below a depth of, say 6 in., the pressure in the ballast beneath the edge of the tie, and outward, is mostly due to the load applied by the opposite side of the tie. It may be stated that the lines of pressure should probably be slightly curved, although an empirical equation equivalent to straight "lines of pressure" gave results in very good agreement with the results of experiments for depths up to 24 in.†

The hypothesis of "lines of pressure" seems to have remained unnoticed, hence it seems desirable to call attention to its value in helping to explain the laws which govern the distribution of pressure through granular material.

The profession is greatly indebted to Professor Terzaghi for the new light he has thrown on a very difficult subject. The classification of clays by their "load-volume" relations, "permeability", and "shearing resistance", is an important advance over previous attempts which based the classification on color, grain size, water content, colloid content, etc.

The use of remixed material for determining "load-volume" relation and measuring "permeability" is open to the objection that the results do not apply to the undisturbed material. For example, in tests made on clay beneath the foundation of the new Materials Testing Laboratory at the University of Illinois, it was found that the water content of the undisturbed material was the same as that which corresponded to a pressure of more than 5 000 lb. per sq. ft. on the remixed material. The samples were taken at depths of about 3 and 6 ft. below the original surface where the pressures were, therefore, only a few hundred pounds per square foot. Both samples were taken below the water-table at the time of sampling, and the lower one was below the lowest water-table. Undisturbed clays probably have their flat, mica-like particles oriented in such a way that the voids are less than in the remixed material.

Methods should be devised for obtaining undisturbed samples for all the tests. This is very difficult even under the most favorable conditions and seems almost impossible in the case of material at great depths.

In the Red River Valley in Minnesota, North Dakota, and Canada, there is a widespread, deep bed of clay on which there have been many foundation failures. It has been found that buildings near railroad tracks have settled much more than similar buildings at a distance from the tracks, due to the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1577, Fig. 100.

† *Loc. cit.*, p. 1575, Table 19.

jarring action of heavily loaded trains increasing the instability of the clay mass.

Mr. Albert Jorgensen, who has conducted experiments in soil mechanics under the direction of the writer, makes the interesting suggestion that the distribution of pressure in clay beneath footings may be determined by experiments on models. A rather wet mixture of a uniform clay is to be placed in a large water-tight box with a thin layer of sand on the bottom and at the sides. A model of the particular footing to be tested is then to be placed and loaded. The surface of the clay is to be covered with water. The pressure on the footing will cause the water in the clay to flow away from the parts having the greatest intensity of pressure. When sufficient time has elapsed to reach the equilibrium condition, small samples are to be taken at various points in the mass beneath the footing and their water contents determined. From the "load-water-content" curve of the particular clay the water contents so determined may be translated into the intensities of pressure in the clay at the places from which the samples were taken. Measurements of the settlement of the footing, or of its parts, could also be taken. It is recognized that the box would have to be of large size to eliminate the effect of the boundaries on the distribution of pressure.

ALBERT JORGENSEN,* ESQ. (by letter).—This paper appeals to one who has studied it thoroughly because it shows that the author actually took some of the soil and determined its mechanical properties. He found that there is a great difference between the mechanical behavior of sands and clays and that clays vary greatly among themselves.

The capillary pressure of the water in the multitudinous, very small interstices in a clay is a factor of importance. It governs the volume occupied by a given clay under given conditions and under a given intrinsic pressure. This is one of the outstanding differences between sands and clays. The phrase, "under given conditions", is significant. The volume of a portion of clay at a given pressure (assuming that water is present) depends on the history of the clay.

If a sample of clay, in a condition near its lower liquid limit, is compressed by a certain pressure (p_1 lb. per sq. ft., for instance), it assumes a volume, definite for that clay, for the given pressure and the conditions of the test. Under the same pressure (p_1), but after having first been loaded to p_2 lb. per sq. ft., the same clay might occupy a different volume. For instance, if a sample of the same clay is made to sustain a pressure equal to p_1, but after having sustained a larger pressure (p_2), the volume will be smaller. Expressed differently, the pressure-moisture tests show that the volume at a given pressure depends on whether that pressure has been reached after positive compression only or after the clay has become re-saturated in recovering from a greater compression. The volume also depends on the intensity of the pressure attained before re-saturation was commenced and the number of times the clay has been compressed and re-saturated.

* Instr., Dept. of Gen. Eng. Drawing, Univ. of Illinois, Urbana, Ill.

The pressures may be applied mechanically in a manner designed to simulate actual conditions in the field; or they may be applied by the capillary pressure of the water which assumes enormous values if the sample is dried. Partly drying a sample would affect the clay to a degree comparable to that caused by a pressure which would produce the same change of volume.

Clay and mud beds, with the possible exception of recent delta and lacustrine deposits, have been subjected to unknown pressures and possibly may have been partly dried and re-saturated several times.

For these reasons a bed of clay below the level of ground-water is apt to be found dryer than could be explained by the existent pressure. For instance, the soil taken from beneath the location of a footing on the site of a certain building was found to be dryer even than the average unit pressure of the building would cause it to become. This soil was several feet below the level of ground-water. The pressure-moisture test, in this case at least, indicated that the soil had been compressed either by weight or capillary pressure.

The question then arises, as to what is the value of load-moisture tests in such cases. This would be answered if methods of sampling and technique for making tests on the soil in its original condition were at hand, but such methods do not seem to be forthcoming.

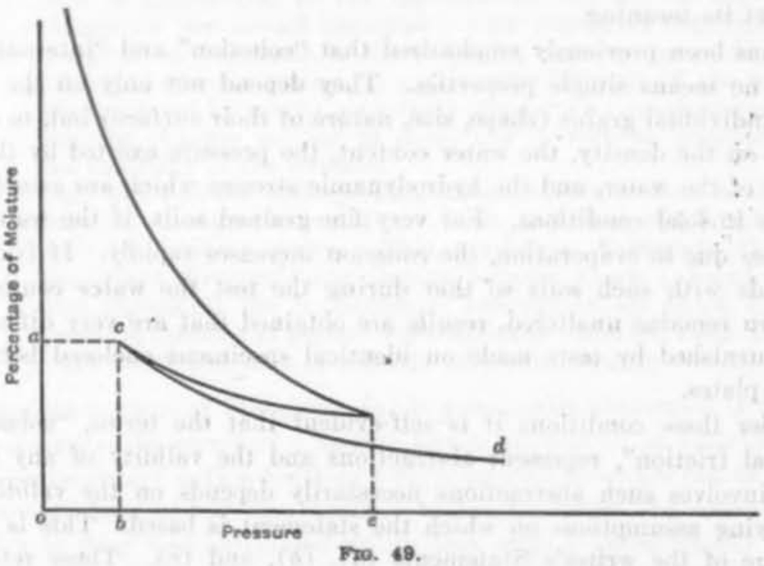


FIG. 49.

The next question which naturally arises is, "How may the tests be made on disturbed samples which will give a fair measurement of the condition of the undisturbed soil?"

The writer believes that a load-moisture test performed as follows, would give results which would aid greatly in formulating practical judgments concerning settlements due to change of volume of the soil. Wet some of the soil to a condition near its lower liquid limit and compress it in a cylinder by means of a piston. The soil should rest on a solid filter which will allow the water to pass to and from the soil as the pressure is varied. Compress the soil to such a point (Point e, Fig. 49) that when the pressure is reduced to b (that pressure imposed on the soil in place), the moisture content will equal that of the soil in place (Point *a*). This would require nicety in testing and could probably be attained only after several trials. Then compress the soil to any desired degree.

The writer believes that the curve, *c d*, so obtained, would be accurate enough as a basis for practical judgments. It might have more value if obtained after several compressions and re-saturations.

In order to apply a pressure-moisture curve to aid in estimating settlement, it is necessary to know the distribution and intensity of pressures which exist under foundations. Tests by M. L. Enger, M. Am. Soc. C. E.,* and Messrs. H. E. Goldbeck,† and J. A. Moyer,‡ indicate that the pressures are far from uniform and may have values several times the average unit pressure.

CHARLES TERZAGHI,§ M. AM. SOC. C. E. (by letter).—This paper has called forth discussions from nineteen engineers in the United States, Russia, India, and China, representing a wide range of experience. While demonstrating the widespread interest in foundation problems, the discussion also disclosed the fact that few readers have had an opportunity to consult the publication from which the writer's Statement (*a*) was reproduced. For this reason, and on account of the condensed form of the statement, there was a tendency to misinterpret its meaning.

It has been previously emphasized that "cohesion" and "internal friction" are by no means simple properties. They depend not only on the character of the individual grains (shape, size, nature of their surface) but, to a variable degree, on the density, the water content, the pressure exerted by the surface tension of the water, and the hydrodynamic stresses which are associated with changes in load conditions. For very fine-grained soils, if the water content decreases due to evaporation, the cohesion increases rapidly. If friction tests are made with such soils so that during the test the water content of the specimen remains unaltered, results are obtained that are very different from those furnished by tests made on identical specimens enclosed between permeable plates.

Under these conditions it is self-evident that the terms, "cohesion" and "internal friction", represent abstractions and the validity of any statement which involves such abstractions necessarily depends on the validity of the simplifying assumptions on which the statement is based. This is obviously also true of the writer's Statements (*a*), (*b*), and (*c*). These refer to two different materials: (*a*) Those for which the bearing power depends essentially on cohesion, internal friction representing a negligible item; and (*b*) perfectly cohesionless materials. The validity of the statements is limited by the following assumptions:

(1) The shapes of the loaded areas are geometrically similar.
(2) The term, "settlement", refers strictly to the total downward movement measured after the structure has reached a state of complete rest.

* *Engineering Record*, Vol. 73, p. 106.

† *Proceedings*, Am. Soc. for Testing Materials, 1917.

‡ *Engineering Record*, May 30, 1914 and March 13, 1915.

§ Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

(3) The comparison between the settlement of loaded areas of different size should be confined to those produced by unit loads which are less than about one-half the ultimate bearing capacity for the smallest area.

(4) Hooke's law need not necessarily be accepted, but Poisson's ratio is assumed to be a constant.

Assumptions (1) to (4) apply to both Cases (*a*) and (*b*). The following additional assumptions apply to Case (*a*):

(5) The mechanical properties of the loaded material (relation between stress and strain and the compressive strength of the material) are practically independent of time and of the depth below the surface.

(6) At any point of the underground, failure occurs as soon as the difference between the greatest and the least principal stress exceeds the compressive strength of the material.

For Case (*b*), Assumptions (5) and (6) should be replaced by the following:

(7) The mechanical properties of the material (coefficient of internal friction) are practically independent of time and of the depth below the surface.

(8) The loaded material is incompressible; that is, the settlement of the load is merely due to the deformation, but not to the volume change of the loaded material. This condition combined with Assumption (4) leads to the following consequence: If all the principal stresses acting on a prismatic element of the material are increased by multiplying their intensity by equal amounts, the element changes neither in shape nor in volume, irrespective of the intensity of the stresses.

(9) At any point of the underground, failure occurs as soon as the ratio between the greatest and the least principal stress exceeds a definite limiting value. This condition represents the most important distinguishing feature between Cases (*a*) and (*b*); it also represents the fundamental assumption on which all earth pressure theories of perfectly cohesionless materials since Coulomb were based and may be considered almost strictly correct.

The subject content of Statement (*a*) is graphically illustrated in Fig. 50. Curve C represents the result of a loading test performed on an area (circular, square, or rectangular) with a width of 1 ft., supported by either one of the two ideal materials described by the writer. Curve C_1 shows the characteristic results of a loading test performed on an area with a geometrically similar shape, but with a width of 3 ft. supported by the first of the ideal materials; Curve C_2 applies to the second. The term, "ultimate bearing capacity", refers to the pressures, p_1 and p'_1, respectively, at which a very small increase in load produces a very important increase in settlement. The settlements produced by equal unit loads acting on areas of various sizes will be termed the "equivalent settlements" of those areas. Using these terms, the laws illustrated by Fig. 50 would be:

(*A*) If the ultimate bearing capacity of the loaded ideal material is essentially due to cohesion, the results of the loading tests performed on areas with a width, d, equal to 1 ft. and 3 ft., respectively, would be represented by Curves C and C_1 (Fig. 50). Under a load of p per unit of area, which is assumed to be much smaller than p_1, the settlement, S_1, of the large area is approximately three times that of the smaller area, S; or, in other words, the equivalent settlement increases with increasing values of d. On the other hand, at a unit load of p_1, a small increase of the load produces for both areas a very large increase of the settlement, because both curves, C and C_1, bend steeply downward as soon as the load approaches p_1. Therefore, it is correct to state that the ultimate bearing capacity is independent of the size of the area.

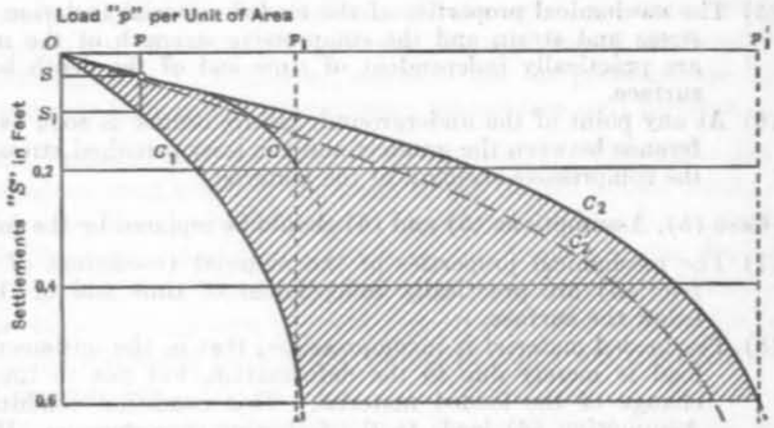


FIG. 50.—SETTLEMENT CURVES FOR LOADED AREAS.

(*B*) If the loaded ideal material is perfectly cohesionless, a similar pair of loading tests would be as plotted in Curves C and C_2 (Fig. 50). For a load of p per unit of area, the settlement, S, is practically independent of the size of the area, provided p is substantially smaller than p_1. On the other hand, the small area practically yields under a load of p_1 per unit of area, while the larger area must be loaded with $p'_1 = 3\,p_1$, in order to produce ultimate failure. Hence, it is claimed that the ultimate bearing capacity of the ideal, perfectly cohesionless materials (Law *B*) increases with the width of the area.

There was some doubt in the minds of most of the discussors as to whether Laws (*A*) and (*B*) were derived from theory or from experiments. To remove this doubt, it should be stated that the conclusions were obtained entirely by theory which was based on Assumptions (1) to (9), inclusive. As a matter of fact, if the validity of these assumptions is granted, Laws (*A*) and (*B*) need no proof, because they represent a simple consequence of the laws of similitude. Hence, any inconsistency between theory and practice must be ascribed to the properties of the material being at variance with those of the assumed, ideal materials.

If the properties of any soil conform reasonably well with the assumptions of theory, the laws represented by Fig. 50 must be valid. However, a comparison between the ideal materials and the actual soils leads to the conclusion that clean, cohesionless sands differ from the ideal cohesionless material inasmuch as they are not perfectly incompressible. If the intensity of all the stresses acting within the loaded material increases at the same rate, compression of the underground takes place which, in turn, causes a certain amount of settlement in addition to that caused by distortion. Therefore, the empirical curve (C'_2 in Fig. 50) would be somewhat steeper than the theoretical curve, C_2; also, the ultimate bearing capacity of the loaded material may be slightly reduced for the same reason. However, since the compressibility of sands has already been thoroughly investigated, it is a simple matter of estimating roughly the discrepancies in every case that may come up in practice. For dense sands, the discrepancy between theory and practice is very small, while for loose sands it is apt to be important.

By comparing the ideal cohesive material represented by Curves C and C_1 (Fig. 50), with actual plastic uniform clays, the conclusion is reached that all assumptions are fairly accurate, except for a certain increase in the cohesion of the clay caused by progressive consolidation of the loaded material. Yet this increase could not possibly account for the striking discrepancy between the settlements which should be expected for clay foundations from the results of loading tests and the actual behavior of buildings standing on such materials. Among the fundamental assumptions of the theory, there is just one which could possibly be made responsible for the apparent defect of the theoretical results. According to Assumption (2), the term, "settlement", refers strictly to the total downward movement until the structure comes to rest. In practice, the settlement observations on buildings are made, at best, a few years after the buildings have been constructed, and when the settlement is still taking place.

The time which has to pass until the settlement of a fair-sized building resting on clay would approximately conform to Laws (A) and (B) obviously depends on the period required for the excess water to drain out of the clay beneath the building. This time, in turn, depends on the permeability of the clay and can be estimated by the theories previously published by the writer.* Suppose, for instance, that the settlement due to pure compression of the clay beneath an area 1 ft. wide was, after 5 days, 0.5 in. According to theory, for an area with a geometrically similar shape, 20 ft. wide, subjected to exactly the same load per unit of area, it would take $5 \times 20 = 100$ days to settle through the same distance of 0.5 in., provided the excess water could not escape except through the exposed surface of the clay, surrounding the loaded area; or, in other words, provided the bases of the loaded footings were water-tight. On the other hand, if the bases of the loaded footings were permeable, the settlement of both the small and the large areas, after 10 days, would be approximately identical. However, for the small area, the settlement would stop after it had reached during a period of several months a total of about 1.5 in., while for the larger one it would continue with decreasing speed until after several generations it would reach an ultimate value of $1.5 \times 20 = 30$ in.

Thus, it seems highly probable that the apparent contradiction between theory and practice for clay foundations is due not to a defect of the theory, but to the attempt to correlate the settlement of small loaded areas—measured almost as the footing comes to rest—with that of full-sized buildings in which the settlement has scarcely begun. As a matter of fact, in old cities with

* "Erdbaumechanik", Wien, 1925.

buildings resting on clay, or on very slightly permeable mud, such as parts of London or Constantinople, for example, there is ample evidence of progressive subsidence. The City of Chicago, Ill., too, despite its comparative youth, is crowded with instructive examples. A section of Cambridge, Mass., has subsided more than 2 ft. since 1880. If it were possible to compare the results of a loading test on clay with the settlement which a raft foundation, resting on the same clay, had undergone not 1 year but 200 years after the building was constructed, the agreement with theory would be much more satisfactory.

These statements are corroborated by the fact that the examples presented in the discussion as arguments against the validity of the writer's statement, referred entirely to clay foundations, on which the settlements are retarded by the enormous resistance of the material against the escape of the excess water. No contradiction was found to exist between theory and practice for foundations of friable limestone, muck soils, silt, and sand, and other fairly permeable materials (see, for instance, the discussions of Mr. White on limestone, Mr. Shaw on muck soils, and Mr. Wing on sand and gravel).

The bearing capacity of most of the more permeable soils commonly found in practice, for example, sands with traces of silt, or deposits of "muddy sands", is appreciably influenced by both cohesion and friction, the relative importance of cohesion depending on the composition and on the density of the material. For such soils, a loading test performed on an area 3 ft. wide should furnish a curve somewhere within the shaded area in Fig. 50, its shape and position depending on the degree to which cohesion participates in producing the bearing capacity.

Attention should also be called to the fact that Laws (*A*) and (*B*), or any rules for the interpretation of load test results, are based implicitly on the assumption that the soil is practically uniform and homogeneous to a depth at least equal to 1.5 times the width of the raft foundation, the settlements of which are to be predicted, or of the width of the area over which the footings are distributed.* Since in practice this condition seldom exists, the field for successful application of the load-test method for determining the admissible bearing value of the soil is far more limited than most engineers seem to realize.

In his paper the writer quoted the rules illustrated by Fig. 50 (Laws (*A*) and (*B*)), merely for the purpose of demonstrating that no real contradiction exists between the different settlement observations quoted in the paper (Chicago, San Francisco, or the test results of Goldbeck and of von Emperger), and that the apparent contradiction between these test results disappear if they are "examined in the light of applied mechanics". In all these cases, the properties of the loaded materials (hardpan in Chicago, succession of layers of clean or sticky sand in San Francisco), or the conditions under which the tests (by Goldbeck and von Emperger) were made, warranted the application of these principles (see Fig. 50). In contrast to this, no such application would be justified for correlating the results of a small-scale loading test on clay with the observations covering a period of a few months or years of the history

* According to Assumptions (5) and (7), p. 381, strict validity of the laws, requires uniformity of soil conditions to a depth equal to infinity.

of the settlements of a raft foundation resting on the same clay. In brief, the application of any rule to practical cases should be preceded by a careful investigation in order to determine to what extent, if any, the practical conditions agree with the fundamental assumptions of the rule. Due to the wide variation in soil properties no rules can be established that are valid for all soils.

The discussions deal essentially with two main topics—the *Engineering News* formula and the interpretation of the results of loading tests.

Among those who discussed the *Engineering News* formula, only one had anything to say in its favor. The others rejected it more or less emphatically, and without exception, on the basis of past experience. However, little attention was paid to what the writer considers by far the most important feature of the situation, namely, that the most general pile-driving formula (Equation (3)), is fundamentally rigorous, provided one grants the validity of the basic assumptions. The most important of these assumptions concerns the identity of the static and the dynamic pile-driving resistance. The failure of the formula to give satisfactory results for pile-driving through clay or saturated silts is due merely to the fact that, in these cases, the conditions existing in practice fail to conform with the theoretical assumptions.

These remarks lead to the attitude of the discussors and of engineers in general toward the writer's statements as illustrated in Fig. 50. Some experienced engineers have accepted these statements as not being in contradiction with their own observations. Others, also, backed by broad personal experience, have rejected them. This fact should arouse the suspicion that conditions are not simple enough so that solutions can be found in the shape of rough and ready half empirical formulas. In a third group of discussions, attempts have been made to disprove the statements by proposing formulas which are claimed to agree better with practice.

These reminded the writer of the many discussions he has read about pile-driving formulas in past years. The method of approaching the problem was always the same. As often as a new formula appeared it was found invariably that the results furnished by it did not agree with many of the observed facts; therefore, it was argued that the formula must be modified. Nobody seemed to suspect that its deficiency might be due to the nature of an assumption in common to both the criticized and the proposed formulas. Hence, the discussion of the pile-driving formula revolved for half a century in a vicious circle.

None of the discussors seemed to realize that the writer's statements stand as firmly "on their feet" as applied mechanics, provided the fundamental assumptions conform with the conditions existing in practice. The assumptions cover all the essential mechanical properties of the soils, except those which depend on time as, for instance, the compression of a clay or of a very fine saturated silt. In a similar manner, the assumptions on which Equation (3) is based consider all the essential elements of pile-driving, except the fact that, in many cases, the dynamic and static pile-driving resistance are not identical. Hence, it is justifiable to conclude that Laws (A) and (B) must be correct, except for cases in which the time factor partly or wholly upsets their validity for the brief period covered by the observations.

Instead of presenting this simple, but very important argument, the members of the third group of discussors merely tried to replace the laws, which have a limited but known range of validity, by others which are neither justified by rigorous analysis nor supported by a known range of validity. In doing so, this group repeated the mistake made by discussors who tried to replace the classical pile-driving Formula (3) by other more objectionable ones, instead of inquiring as to the fundamental cause of the observed disagreements.

These remarks also apply to discussors of the second group who attempted to question the soundness of the writer's statements solely on the ground of past experience. After realizing the limitations of theory, these engineers may ask: What then is the use for establishing rules at all? The answer to this question touches the most vital point of the science of foundations, and it cannot possibly be over-emphasized.

Settlements, as encountered in practice, depend as a rule on a number of circumstances, such as depth of foundation; presence of other structures in the vicinity of the new building; the occurrence of alternate layers of slightly permeable and markedly permeable strata; total thickness of the compressible deposits; relative position of the weakest spots of the underground with reference to the base of the foundation; and many others. In brief, the settlement of a structure is a function of so many variables that it would be hopeless to arrive even at crude and empirical conceptions of the causes of settlements unless systematic efforts are made to evaluate the relative importance of each one of the individual variables. This can be done merely by establishing a theory, not of the complex materials encountered in practice, but of a set of ideal materials with certain very simple properties, each one of which will serve as a means of studying one individual aspect of the real soils.

By such systematic analysis only can engineers gradually gain a deeper insight into the workings of actual soils. Since by necessity the very best analysis of that type falls far short of taking into account all the conditions that exist in practice, there always will be a certain gap between theory and practice, which can be bridged merely by experience. Considering the existence of this gap, the writer agrees heartily with Mr. Gow's warning against readily accepting common formulas for use in actual practice. The writer even doubts whether formulas with a fairly general degree of validity could be established at all. Theory in connection with foundation engineering should be regarded merely as an essential supplement to experience, to be used as a tool for acquiring a deeper insight into the causes of observed foundation defects and for evaluating the relative importance of the factors in existence.

Without a more profound analysis of phenomena encountered in practice, experience is doomed to remain the same accumulation of incoherent, meaningless facts which fill the space between the covers of many textbooks on foundation engineering. The time may come when a man with experience, but without a thorough theoretical training, will be considered almost as unfit to be trusted with a design of an important foundation as the theoretician who has had no experience. In structural engineering, this time came nearly a century ago.

The writer hesitates to accept Mr. Gow's advice concerning the *Engineering News* formula. If, as Mr. Gow states, it is a fallacy to rely too strongly on the results of individual pile tests, the application of the *Engineering News* formula seems to represent an even greater fallacy.

It is significant that Mr. Goodrich abandoned the formula long before the writer had even heard about its existence. The observations made by Mr. Goodrich concerning the stress distribution over a square bearing plate should be an incentive for similar investigations of raft foundations resting on clay soils by means of Goldbeck pressure cells or similar devices.

The experience reported by Mr. Endersby reminds the writer of a similar incident which occurred in Galveston, Tex. On the south side of the Island of Galveston, to a depth of about 50 ft., the underground consists of very fine sand mixed with powdered shells. Because of the very dense structure of this sand, it is considered impossible to drive piles to a depth of more than 15 or 20 ft. without smashing the pile-head. According to Colonel Schley, Government Engineer of Galveston, some piles driven with a steam hammer to refusal settled afterward several inches under a static load of less than 15 tons. This and the incident mentioned by Mr. Endersby are obviously decompression phenomena which can also be reproduced in the laboratory. If a layer of sand that is laterally confined is subjected to a rapidly applied pressure up to a certain value and then the pressure is kept constant (possibly by a testing machine), the counter-pressure of the sand against the piston first drops rapidly and then slowly to less than one-half the original reaction. In connection with this statement, it should be emphasized that there seem to be no records of conspicuous decompression phenomena except for cases in which the pile-driving was done with a steam hammer that could apply pressure very rapidly. The intensity of the decompression phenomena may be considerably increased by the presence of a layer or pocket of very loose sand a short distance beneath the point of the pile. Since the number of known conspicuous decompression phenomena on piles is exceedingly small compared to the number of cases in which the piles behaved normally, such a possibility could at least be considered.

Mr. Endersby's statement that the result of the tests with three different hammers was "a good test for consistency of the [*Engineering News*] formula as applied to various types of hammers", should be accepted with caution. The satisfactory agreement between the results appears to be mere chance; otherwise, it would always exist. To mention one example among several on record: When driving the piles for the Mariotis Lake Bridge, in Egypt, three different pile-drivers were used: A steam hammer (weight, R = 4 400 lb.; drop, h = 3.28 ft.); a hand-operated drop hammer (R = 1 320 lb.; h = 6.56 ft.); and a steam drop hammer (R = 4 400 lb.; h = 3.28 ft.). The points of all the piles penetrated the same stratum of sand.* The *Engineering News* formula furnished for the safe bearing capacity of these piles the average values of 66 800, 16 800, and 27 600 lb., respectively. If the *Engineering News* formula would furnish consistent results for piles driven into the same stratum with

* According to Mr. W. Stross, in *Beton und Eisen*, 1908, p. 113.

different types of hammers, these three values would have been fairly identical. Mr. Endersby's statements show how a formula can acquire a good reputation by an act of Providence rather than by inherent merits. Since engineers very seldom have a chance to watch different types of pile-drivers operating simultaneously on a very homogeneous material, they are apt to base their opinions on an inadequately small amount of actual evidence.

Mr. Paaswell justly emphasizes the fact that the New York Subway "is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems". The same was true for the Panama Canal in the field of stability of earth slopes. Unfortunately, very few results of lasting value were derived from the Panama experiment because of lack of funds available for purely scientific research. In New York, the unique opportunity for systematic research seems by no means to have been utilized to full advantage. The New York Subway remains a beautiful laboratory without equipment and without personnel. The profession should appreciate all the more the idealism of those few engineers engaged in subway construction who, by their own initiative, seriously endeavor to broaden technical knowledge on this subject by systematic experimentation.

Mr. Williams presents a theory based on three assumptions: (*a*) That settlement is merely due to compression of the underground; (*b*) that the total depth of soil compressed is limited by the depth at which the pressure intensity (per unit of area) drops below a certain limiting value; and (*c*), that the coefficient of compressibility, C, is independent of depth. Attention should be called to the fact that Assumption (*c*) could merely be considered approximately valid for materials with strong cohesion; for cohesionless materials, on conclusive, experimental evidence, it does not apply. Mr. Williams then computes the settlement of square areas with different widths, b. From Assumptions (*a*) to (*c*), it is evident that h (Fig. 18), increases in simple proportion with b. This fact is also expressed by Mr. Williams' formula, $h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$. By introducing the value of h into the terms preceding this formula, the same results are obtained as those expressed by Curve C_1, Fig 50. However, instead of retaining h, Mr. Williams ceases to consider it as a function of the width, b, without attempting to justify his step, thus transforming it into a constant. This operation leads to the formula, $W = A\,p + L\,f$, which contradicts his theory. Since his final formula does not represent the result of the theory, it could be accepted merely as an empirical equation and the writer considers that the empirical evidence is too fragmentary to warrant such a step.

The example quoted by Mr. Williams (the office building of the Michigan Central Railway Company in Detroit, Mich.) does not invalidate Curve C_1, Fig. 50. To conform with the assumptions of theory, the underground should be uniform to a depth equal to at least one and one-half times the width of the building. In contrast to this, from his own experience, the writer can state that the clay deposit of Detroit is far from being homogeneous. It consists of an irregular succession of layers of soft and stiff clay.

Furthermore, the observations quoted by Mr. Williams refer to a recently constructed building and settlements are not comparable as long as one of them still continues. For a building resting on clay foundations, the settlement twenty years after it was constructed is certainly not identical with that just after the building was completed. It may be many times more important. Thus, for instance, in Massachusetts, a short time after construction, the settlement of an important structure founded on plastic clay ranged between 0.1 and 1.1 in. One year later, the extreme settlements had increased to 0.6 and 2.8 in., respectively, and ten years later, they amounted to 1.0 in. and 7.6 in., respectively. In Chicago, conditions are not very much better.

Mr. Williams suggests reducing the discrepancy between the results of the loading test and the actual settlement by loading the soil around the test footing. To follow this procedure would mean an attempt to compensate for the effect of the time factor, which is absent, by adding a load factor. The importance of the time factor on the relation between settlements of small and large footings is exceedingly different even for the same clay, depending on the water content. Hence, it seems rather difficult to establish a reliable rule for computing the intensity of the compensating load which should be applied around the footing. Instead of trying to reduce the discrepancy between test results and the settlement of the proposed structure by introducing into his arguments one more element of uncertainty, the writer takes it for granted that this discrepancy (together with the possible effects of the lack of uniformity of the underground) represents inevitable defects of the load test method. Hence, when attempting to evaluate the probable settlement of a proposed structure, he bases his judgment not only on test results and his knowledge of soil physics, but also on past experience and on what the test borings show about the nature of the deeper strata. Test results and physical knowledge are used merely for establishing connection between past experience and the new situation. In no case would he dare to rely solely on a test result.

Mr. Williams states that the writer neglected the effect of the deflection of the foundation on the pressure distribution over the base of the foundation. For foundations on well compacted sand, the pressure-relieving effect of the deflections is very important indeed, but for soft soils, this is insignificant. This fact has been known for more than forty years. Since most raft foundations are supported by soft soils, designers do not often have an opportunity to take advantage of the pressure-relieving effect of deflection. The question concerning the distribution of pressures over the base of raft foundations is still in a controversial state, and the discussion by Mr. White of the same topic seems to indicate that engineers are by no means unanimous in assuming uniform pressure distribution to be safe.

Dr. Chatley calls attention to the slowness with which hydraulic fills, consisting of fine alluvium, expel their excess water. During 1928, under the writer's supervision, Glennon Gilboy, Jun. Am. Soc. C. E., made an exhaustive study of the process of consolidation of an hydraulic-fill dam. For this purpose, a shaft was sunk through the core of the dam and undisturbed samples were secured at various depths below the crest. The results of these investigations represent an important contribution toward present knowledge of the consolidation process; they conform with Dr. Chatley's observations and with the conclusions previously drawn by the writer from theory.

Dr. Chatley's conception of the part that the molecular bond plays in the physics of granular masses is fundamentally sound, although it is still doubtful whether the molecules of two adjoining particles interact directly or whether the interaction merely takes place between the molecules of the skins of adsorped materials surrounding the particles.* The possibility undoubtedly exists that the degree of dispersion of a highly colloidal soil may gradually change. To date, very little is known about this process. The writer is inclined to believe that it is often associated with chemical changes and that it plays a very important part in Nature.

The most promising methods for the design of sheet-piling seem to be those based on the assumption of a circular sliding surface. These methods were developed by K. E. Petterson, Sven Hultin, and W. Fellenius, in Sweden.†

In 1920 the writer was at least as enthusiastic about colloids and about the possible benefits to be derived from colloidal chemistry for foundation and earth-work engineering as Mr. Holmes is to-day. However, during the succeeding years of struggle with the complex material called soil, this enthusiasm has dwindled to practically nothing. Some of the reasons of the progressive disappointment can be explained by means of Mr. Holmes' own data. He admits that "extracting the colloids from a soil to determine the amount of colloidal material is a long and laborious process". Therefore, he proposes a simplified method of his own. According to Fig. 20, an ignition loss of 3% may mean anything between 6% and 29% of colloid content.

Furthermore, since the physical properties of soils with equal "colloid content" are apt to be widely different, Fig. 20 is certainly not a very convincing argument. The following data from tests on a soil in Spanish Honduras may serve as an illustration. The soil had a clay content of 70% and, in its natural state, a water content of 33 per cent. The coefficient of permeability of this soil was found to be: (*a*) Undisturbed sample under primeval forest with 33% of water, $k = 2.6$ cm. per min.; (*b*) the same soil dried, powdered, and compacted, $k = 0.0024$ cm. per min.; (*c*) the same soil with the same water content as in Case (*a*), but after completely destroying its original structure by kneading, $k = 0.00000085$ cm. per min. In addition, by increasing the water content of Sample (*c*), from 20% to 40% the coefficient of permeability was increased from 0.00000010 to 0.00000250.

Hence, even if a rapid and reliable method should be found for determining the percentage of colloids, it may have but little practical value. The quality of a dam construction material depends essentially on its permeability, cohesion, and internal friction. No simple and universally valid relation exists between the colloid percentage and any one of these important physical properties. The situation becomes still more involved considering that

* "Mechanics of Adsorption and the Swelling of Gels," by Charles Terzaghi, M. Am. Soc. C. E., Fourth National Colloid Symposium, 1926.

† "Erdstatische Aufgaben," W. Fellenius, Berlin, 1927; "Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes," H. Krey, Berlin, 1926.

both the permeability and the cohesion of any soil may vary between extremely wide limits, depending only on the history and not on the substance of the soil deposit. Nothing except a refined chemical analysis could disclose the nature of the admixtures. Finally, some of the most excellent dam construction materials that the writer has ever encountered, which were splendidly fit for constructing earth dams without a core-wall, contained almost no colloids.

Considering these and many other facts, the writer cannot be blamed for his skeptical attitude toward those who hope to derive important practical benefits from colloidal research. He prefers to obtain data directly from the results of appropriate experiments. Nevertheless, any advance in the field of colloid chemistry is of outstanding interest to every one dealing with soils and with earthwork engineering. This interest, however, is at present of a purely scientific nature.

Mr. Feld thinks that the writer's effort to establish a more accurate interpretation of actual construction experience is a desire to "suddenly substitute a new 'science of foundations'." for precedent and experience. No such intention ever entered the writer's mind. The need is, not for a latent state of war, but a more intimate co-operation between field and laboratory. In establishing such co-operation, the absence of any authoritative nomenclature certainly is a serious handicap; but it is by no means the principal stumbling block. Far more detrimental is the widespread lack of training in abstract reasoning and the tendency to reject theory completely unless it leads, overnight, to conclusive results.

The reason the writer neglected to deal with the effect of the shape of the area on settlement is obvious. At the time when the paper was written, he did not know anything about it. Even in the case of the bearing of solid against solid within the elastic limits, the shape of the area has a very important effect, but all efforts made to solve the problem theoretically thus far have failed. However, since January, 1927, experiments have been conducted at the Massachusetts Institute of Technology, for the purpose of getting at least preliminary information on this controversial subject. The results obtained are interesting enough. They suggest that the effect of the shape of the area on both the equivalent settlement and on the ultimate bearing capacity may be very different, depending on the degree of cohesion.

For example, assume two loaded areas, one with a circular shape, of which the diameter is d, and the area, f; and the other one rectangular with the width, d, and a length several times larger than d, so that the area is F. For a material with dominating cohesion (Plasticine), the equivalent settlement for f was found to be smaller than that for F. On the other hand, the ultimate bearing capacity for f was found to be considerably greater than that for F. For perfectly cohesionless materials, the opposite relation was found to exist. The tests were repeated several times, and the results were remarkably consistent. Here, again, it seems a fact that the influence of the shape of the area may be very different, depending on whether the bearing capacity of the material is essentially due to cohesion or to internal friction.

The "sympathetic" settlement of adjoining footings may have very different causes. In the case illustrated by the writer's Fig. 11, a fence post located at a distance of about 300 ft. from Building (*b*), had settled several inches. In this case, the sympathetic settlement was due to the presence of a layer of sand beneath a bed of unconsolidated clay. Another type of sympathetic settlement (settlement of the surface of the ground in the vicinity of a concentrated load) has been treated theoretically by the late Professor A. Föppl in "Vorlesungen über Technische Mechanik", which also contains the results of the tests performed in order to check the theory.

Mr. Housel presents a series of very interesting and valuable test results. However, from these results, he feels justified in concluding that the statements presented by the writer concerning the relation between the bearing capacity and the diameter of round plates, is false, and that they lead to absurd results. A similar opinion is known to be held by many other engineers.* The writer, therefore, may be justified in examining the arguments presented by Mr. Housel in somewhat greater detail.

At the outset of this closing discussion, the writer quoted the assumptions on which Fig. 50 was based. It also was shown that Laws (*A*) and (*B*), derived from these assumptions, are as unquestionable as the laws of applied mechanics, provided the assumptions agree with conditions existing in Nature. Therefore, if a discrepancy is found between theory and practice, it must be due to some factor not taken care of by theory. In the case of foundations resting on clay, an apparent discrepancy was found to be caused by the low permeability of the material which retards the settlement of a large footing much more than that of a small bearing-block. The advantages to be derived from such knowledge are manifold.

The first lesson learned is the intimate relationship that exists between the low permeability of the underground and the progressive settlement of the buildings which it supports. Furthermore, it is important to understand that a settlement of identical foundations on identical clays may be very different, depending on whether or not the clay contains streaks or layers of permeable sand through which part of the excess water of the clay may freely drain. It is also important to prevent the mistake of applying to a fairly permeable system of strata the experience previously obtained with foundations on very slightly permeable materials, such as plastic, homogeneous clays. That is what the writer calls a "scientific approach" to a soil problem; an attempt to uncover the responsible factors by isolating the variables and systematically determining their relative importance.

However, Mr. Housel selected another and apparently simpler method of dealing with his subject. Instead of attempting to investigate whether or not the statements quoted by the writer were consistent with the accepted laws of mechanics, he introduced into his discussion rather vaguely defined conceptions such as "shearing resistance" per unit per length of perimeter and "pressure bulb". These conceptions certainly are convenient for a general description of the conditions which exist in the underground of a loaded area; but, if it comes to rigorous analysis, they cease to exist. In applied mechanics, the term, "shearing resistance", refers exclusively to a unit of area, not to a unit

* *Engineering News-Record,* March 29, 1928, p. 520.

of length, and the pressure bulb is not referred to at all, except inasmuch as it indicates vaguely what is rigorously expressed by the formulas of Hertz and Boussinesq.

Considering these facts, it is very difficult to translate into terms of applied mechanics such statements as "the important consideration * * * is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb."

Speaking of his test results, Mr. Housel admits that the results agree with conditions as defined by Curves C and C_1, Fig. 50. Yet, he rejects them because "* * * when the principle is applied to the much larger areas used in practice that the fallacy of this assumption becomes apparent". Then, without preceding analysis, Mr. Housel supersedes the term he rejects by a term of his own conception (Equation (11)), and simply claims that his term represents a satisfactory solution, valid for all soils irrespective of their physical properties. Nowhere in his discussion does he hint at the possibility that a difference in drainage conditions may have a far greater influence on settlements than a difference in compressibility or in internal friction.

Mr. Bailey touches the human side of foundation engineering by quoting two cases of failure which occurred in spite of thorough preliminary work. The question arises: Will it ever be possible to avoid entirely such tragic incidents? The writer does not believe so. At the same time, from his personal experience with foundation failures, he is convinced that more than 90% of the accidents which occur nowadays could be avoided by proper application of the results of soil research to the design of foundations. The others will remain inevitable, very likely forever, on account of the fragmentary character of the information furnished by test borings. By transforming this fragmentary information into complete geological profiles, the engineer automatically introduces into his forecast a personal element and with it an element of uncertainty. Thus, on construction work with which the writer was connected, careful test borings were made and all of them indicated the presence of a continuous bed of very stiff clay extending to a considerable depth below the base of the proposed raft foundation. Immediately before the reinforcement for the concrete slab was placed, water broke through the bottom of the pit, carrying large quantities of very fine sand. Subsequent investigation by supplementary boring showed that there was a narrow belt of quicksand enclosed within the clay bed at a very shallow depth below the bottom of the excavation pit and by mere chance the test borings missed it.

The risk caused by the fragmentary character of test-boring records will always continue to exist. In connection with dam foundations this inevitable risk is very much greater and very much more serious. The ultimate aim of the science of foundations merely consists in reducing the present intolerably wide margin between the factor of safety of the most and of the least solid foundations.

Mr. Condron describes an instructive case of foundation experience in connection with the Chicago blue clay. According to the test borings, the stiffness or the cohesion of this clay increased considerably with the depth.

Since the cohesion of the clay varied with the depth, and since, in addition, the clay represents a very slightly permeable material, Law (*A*) is not directly applicable to Mr. Condron's case, and the interpretation of the test results represents a rather complicated problem of soil physics.

Mr. Condron's method of handling the problem can be considered a typical example of a very conscientious and conservative approach along the lines of current practice. In order to determine wherein this practice may fall short of what is desirable, engineers may ask the question: What arguments led Mr. Condron to conclude that 3 000 lb. per sq. ft. is a safe load for the soil which he tested?

His most important argument undoubtedly was that the neighboring buildings, with foundations designed for a soil pressure of 4 000 lb. per sq. ft., showed no evidence of unequal settlement. After Mr. Condron had verified this fact, he made a loading test, with the soil around the loaded area remaining in place. From the favorable outcome of this test, plus the evidence furnished by the neighboring buildings, he reached his conclusion.

If the procedure for determining the admissible soil pressure is so simple, then why does the engineer need any of the more refined soil studies? To answer this rather vital question, suppose that an engineer has to construct a duplicate of Mr. Condron's building. Before preparing the plans for the foundation, he makes exactly the same test as Mr. Condron did, except to assume that there are no neighboring buildings the behavior of which would inform him about the quality of the underground. Furthermore, when making his test borings, suppose that the engineer discovers that, to a depth of 10 ft. below the base of the proposed footings, the underground is the same as that found by Mr. Condron. Below that depth conditions differ in one of three possible ways, as follows:

(*a*) Instead of homogeneous stiff blue clay, there is a homogeneous soft blue clay.

(*b*) Instead of homogeneous stiff blue clay, there is a stiff blue clay which contains several layers of dense but very permeable, water-bearing sand.

(*c*) Instead of homogeneous stiff blue clay, there is a "varved" clay, consisting of a succession of almost impermeable and fairly permeable layers about 1 in. thick.

Since, at a depth of 10 ft. below its base, the soil pressure exerted by the loaded 6-ft. square test footing is already negligible, the outcome of the loading test will be almost exactly identical with that obtained by Mr. Condron. Yet, the major part of the pressure bulbs of the full-sized footings will be in a material altogether different from those in the case cited by Mr. Condron. As a consequence, the ratio between the settlement of the test footing and of the full-sized footings will also be altogether different.

In Case (*a*), the settlement due to lateral yield of the soft clay will be much more important than it was when the deeper part of the stratum consisted of stiff material. In Case (*b*), the excess water in the clay beneath the full-sized footings can freely drain into the sand out of the clay, while through the homogeneous stiff clay beneath Mr. Condron's building, it could not percolate except very slowly. In Case (c), the clay can discharge its excess water still more freely, and the settlements due to gradual consolidation are apt to be very important although the material gives the impression of being very stiff. Whenever, in practice, the writer encounters "varved clay", he knows there is trouble ahead, and experience has invariably confirmed his anticipation.

The engineer who pays little or no attention to the manifold characteristics of soils would be apt to trust the outcome of the loading test, and would consider Cases (a) to (c) to be identical with the one described by Mr. Condron. Hence, he would also consider 3 000 lb. per sq. ft. a safe load and to his surprise the result of his decision would be an excessive settlement of the structure. The task of developing a "science of foundations" consists in preventing such errors by painstaking analysis of the relative weight of each individual factor which could possibly influence the behavior of the proposed structure. Without such analysis, the profession would have always as many different rules for the interpretation of the results of loading tests as there are engineers who keep a record of their observations.

Mr. White's experience, derived mostly from construction work in New York, covers some of the most difficult fields of foundation engineering and, whenever he chooses to disagree with any of the writer's statements, the writer always feels compelled to "think it over twice" before he replies. The two points wherein Mr. White's opinions seem to differ from those of the writer are the value of loading tests and the relation between the bearing capacity and the density of soils.

The principal arguments against placing too much confidence in loading tests were explained in general at the beginning of this closing discussion and by means of an example in the writer's comment on Mr. Condron's discussion. The writer uses loading tests merely as a means of investigating the character of individual strata, but he never would dare to pass judgment on the probable settlements of a full-sized structure without careful consideration, not only of the results of the loading tests, but also of the influence of the characteristics of those strata between the elevation of the base of the pressure bulb of the test footing and that of the full-sized footing. The cases in which the soil is of a perfectly uniform character to a depth at least equal to the width of the building are very rare.

The relative density of a soil or of a rock is undoubtedly a factor of importance, but unfortunately the relation between the behavior of this soil and its density are by no means simple enough to furnish reliable information on the bearing capacity of the material. Thus, when experimenting with clays, the writer had an opportunity to compare two blue clays from two different localities. One of these clays, with a water content of 35% (volume of voids, 48%) was almost liquid, while the other with exactly the same water content and the same volume of voids was found to be stiff and had a very appreciable consistency. Similar although less conspicuous variations were found to exist when experimenting with sands.

This leads to a discussion of the effect of the shape of the grains. Mr. White is inclined to assume that the large voids ratio may have the same effect on the mechanical properties of a sand as the mica content. As a matter of fact, the compressibility of the same sand increases rapidly with its volume of voids. There is, however, one fundamental difference between pure sand and sand-mica mixtures which cannot be explained by Mr. White's assumption. This difference concerns the elasticity of the two materials as illustrated by the shape of the rebound curves. Thus, when experimenting with very fine quartz dust, it was found that the compression curve of this material was almost as steep as that of a sand-mica mixture, but the rebound curve was almost as flat as that of an ordinary sand, while the rebound curves of the equivalent sand-mica mixture rose very sharply. Since the rebound curves of the clays are as steep as those of sand-mica mixtures, the writer concluded that the clays and silts owe their mechanical properties essentially to the presence in the material of a high percentage of scale-like particles. At the time when this statement was published, it was presented as a hypothesis. Since then, Professor Goldschmidt, in Oslo, Norway, has succeeded in measuring the actual quantity of scale-like particles present in typical Norwegian clays,* and the data which he obtained checked so well with those derived from the sand-mica experiments performed at Massachusetts Institute of Technology that there is little doubt about the hypothesis being fundamentally sound.

From a practical point of view, the principal difference between very fine sand and an equally fine sand-mica mixture is that the volume of voids in the former is always very much smaller than that of the latter, provided both were deposited by identical geological processes. Sand-mica mixtures with macroscopic grains and with a high percentage of mica have not yet been found in Nature. The so-called "micaceous sands" as a rule contain only a very small fraction of mica by weight. The micaceous appearance is due to the fact that a single sand grain weighs twenty or more times that of a mica flake of equal size.

Mr. White is inclined to doubt the validity of the writer's statements concerning the effect of the depth of a foundation on the settlement for perfectly cohesionless material. His doubts are unquestionably based on what he has observed in actual practice where he has had almost no chance to study a material conforming with the condition of perfect lack of cohesion.

Mr. White's examples of the general character of the settlement of whole buildings (Fig. 31) leave little doubt that the centers of the outside walls settled more than the corners, but it is still an open question whether the centers of these buildings settled more than their perimeters. In order to insure uniform settlements, some engineers in Texas follow the practice of keeping the soil pressures for the outer footings 12 to 20% less than those for the inner footings, and their experience seems to confirm the soundness of their practice. It should also be considered that the relation between the amount of settlement of the center and of the perimeter of a building, at equal

* Abstract by Charles Terzaghi, M. Am. Soc. C. E., in "Economic Geology," 1928.

soil pressure per unit of area may be very different, depending on the degree to which the cohesion of the underground dominates over internal friction.

The introductory remarks of Mr. Simham's discussion describe exactly what the writer has been "driving at" for many years. The writer also concedes that no laws can be derived from uncorrelated sets of experiments; but, at the same time, he seriously doubts whether any formula can be derived to cover all the requirements of actual practice. In dealing with foundation problems, the writer confines himself to what Mr. Simham calls "a careful diagnosis of every factor that influences the behavior of a soil", and in doing so he has almost invariably found that he must be satisfied with finding out to what extent his previous experience applies to new cases. All the rules which he derived, and very likely all those which he will derive in the future, are merely intended to be tools for use in this fundamentally important diagnosis.

Equations (12) to (21), inclusive, represent one possible approach among many to a mathematical description of what engineers may observe. However, their value will depend on the degree to which Mr. Simham succeeds in taking care of the time factor, which is the stumbling block in the path of all similar attempts.

Mr. Marston describes an example for the application of soil physics to the study of materials for hydraulic-fill dams. Since this investigation was terminated, much progress has been made along similar lines by the work performed in the writer's laboratory. In pursuing this work, the thorough investigation of perfectly undisturbed materials extracted from the core of an existing hydraulic-fill dam proved to be of inestimable assistance and threw much light on the disputed problems. (See, also, the writer's comments on Dr. Chatley's discussion).

Mr. Shaw describes interesting experiences with New Orleans muck soils. Since the internal friction of under-drained muck soils is extremely low, their bearing capacity depends essentially on their cohesion. At the same time, that they are fairly permeable is proved by the fact that it is possible to drain them with open ditches. For soils of such a character the rule represented by Curves C and C_1, Fig. 50, has at least approximate validity. This statement agrees with Mr. Shaw's experiences.

Furthermore, previously published theoretical investigations of the writer have led to the following conclusions regarding the ultimate bearing capacity of medium permeable and saturated silts and muck soils. Select on the surface of such a soil two equal areas and load both of them at a different time rate until failure occurs by an "out-and-up" flow of the loaded soil. The more slowly the load is applied, the more chance the soil has to expel its excess water during load application. Therefore, ultimate failure of the area which was loaded very slowly will occur at a higher load per unit of area than that on which the load was rapidly applied. This theoretical conclusion is substantiated by the success of Mr. Shaw's method of levee construction by successive application of wide and thin layers. The compacting effect produced by load application is illustrated by the writer's Fig. 4.

Mr. Shaw's observations concerning the difference between the effect on soft soils of concentrated and of distributed loads call attention to an important fact, closely related to the influence of the depth of foundation on the ultimate bearing capacity of the soil. Suppose the area, *a-a* (Fig. 51), is uniformly loaded, while on both sides of *a-a*, the load decreases from its value at *a* toward zero. For this case, both theory and experience lead to the conclusion that the wider the area over which the load spreads, the greater is the load, p, p_1, p_2, etc., at which failure occurs.

The effect of the flaring out of the load becomes still more conspicuous if the thickness of the soft layer is very much less than the width of the base of the fill (Fig. 52). In this case, the soft layer is "trapped" between the base of the load and the surface of the stiff layer. If the fill has no tensile strength, the limited depth of the soft layer and the friction along the top surface of the firmer ground introduce factors of resistance absent in deposits of greater depth; but if the fill has tensile strength, an arching effect, similar to that which takes place in grain bins, still further reduces the tendency of the soft material to flow. (See Fig. 52.) From his experiences with the construction of fills on peat deposits in Michigan, Mr. V. R. Burton* concluded that in placing a fill with a width ranging between 24 and 30 ft. on the surface of a peat deposit, the character of the resulting subsidence was very different, for varying thicknesses of the deposit. If the thickness was less than about 20 ft., the fill merely produced a trough-like depression in the peat deposit. On the other hand, if the thickness of the deposit exceeded about 26 ft., the fills penetrated as far as the bottom of the deposit, thus completely displacing the peat beneath the center line of the fill. Examples of the sudden subsidence of fills placed on soft muck soils can be found in the report of the Swedish Geo-Technical Commission.† The observations made by the Swedish engineers in connection with large-scale subsidence failures led them to base their stability computations for such soils on the assumption of circular sliding curves.

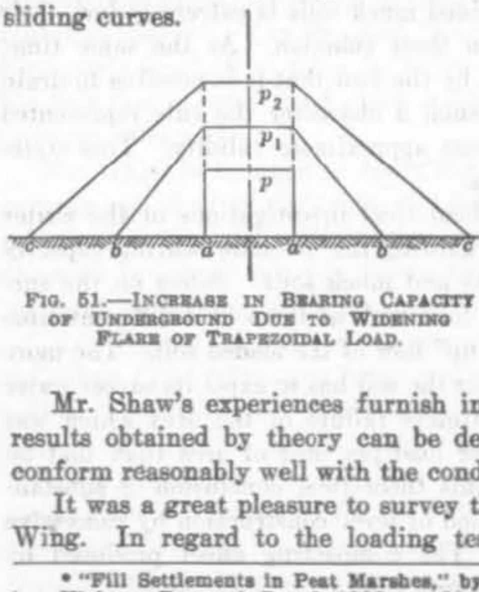


FIG. 51.—INCREASE IN BEARING CAPACITY OF UNDERGROUND DUE TO WIDENING FLARE OF TRAPEZOIDAL LOAD.

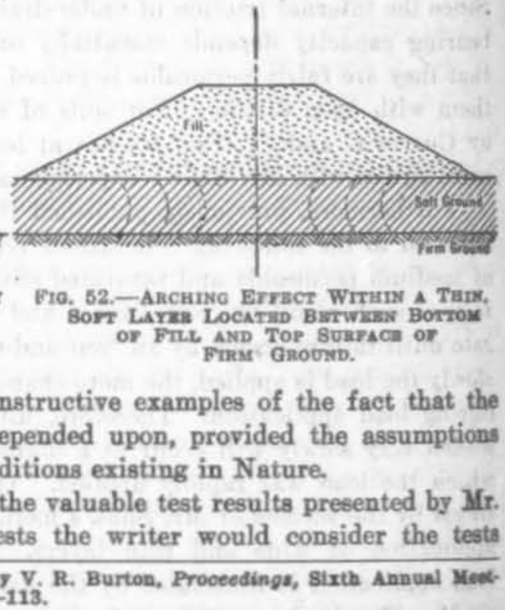


FIG. 52.—ARCHING EFFECT WITHIN A THIN, SOFT LAYER LOCATED BETWEEN BOTTOM OF FILL AND TOP SURFACE OF FIRM GROUND.

Mr. Shaw's experiences furnish instructive examples of the fact that the results obtained by theory can be depended upon, provided the assumptions conform reasonably well with the conditions existing in Nature.

It was a great pleasure to survey the valuable test results presented by Mr. Wing. In regard to the loading tests the writer would consider the tests

* "Fill Settlements in Peat Marshes," by V. R. Burton, *Proceedings*, Sixth Annual Meeting, Highway Research Board, 1926, pp. 93–113.

† Statens Jarnvägars Geotekniska Comm., 1914–22. An English abstract and comments on the report by Charles Terzaghi, M. Am. Soc. C. E., are filed in the Carnegie Library, Pittsburgh, Pa.

finished as soon as the settlement becomes equal to about one-fifth the diameter of the loaded area. Beyond this limit the process of settlement passes into one of penetration. When making the tests oil-jacks are always preferable to screw-jacks.

The writer uses as a criterion for the bearing quality of sand deposits the settlement under a load of 500 lb. per sq. ft., provided the loaded area is at least equal to 1 sq. ft. Under such a unit load, a well-compacted sand deposit does not settle more than about 0.001 ft. The loosest deposit that ever came under the writer's observation settled under the same load through a distance of ¼ in. From the character of the settlement curves for the sand (Fig. 44), the writer suspects that the sand was loosened by the upward current of water, as the pit was being pumped dry. On account of this possibility, the writer keeps the water out of a test pit by pumping, uncovers two opposite vertical sides of the pit, and makes the loading test in a horizontal direction, by inserting a hydraulic jack between two bearing plates, applied to the sides of the pit. The bearing plates should have a clearance, all around the rim, equal to at least twice the diameter of the plate. Experience has shown that the results of such tests differ but little from those made on horizontal plates, and one is sure that the loaded material is in an undisturbed state.

The results of mechanical analysis should preferably be presented in graphical form with the grain size plotted on a logarithmic scale.* The limits of consistency of the clay, quoted by Mr. Wing, remove any doubt concerning the type of clay that he encountered. Experience has shown that, for natural clay deposits, both the limits of consistency and the water content vary within rather wide limits. Therefore, it is advisable to perform these tests for each drill hole on several samples. Instead of determining the volume change associated with a change of the moisture content, it is preferable to determine the shrinkage limit (from dry weight and from the volume of the dried sample, assuming for the dried substance a specific gravity of 2.7).†

The settlement diagram (Fig. 45), for the clay indicates that the loaded area was too big for the size of the test pit. The abnormal shape of the curve seems essentially to be due to arching action within the clay, between the bearing plate and the walls of the pit. Similar phenomena were observed by the writer on a small scale, when loading clay specimens that were contained in a rigid box. The clay beneath the bearing plate undoubtedly compacted, but this fact alone does not account for the characteristics of the curve. The width of the pit should be equal at least to five times the diameter of the loaded area.

Loading tests on both sands and clays should always be made at least in duplicate on what appears to be the stiffest and the softest spot of the underground. The writer selects these spots after having passed over the ground with a sounding rod. The observations to be performed on the diversion dam are most promising, particularly if the settlement observations extend over a longer period and if, in addition, bench-marks are established on the unoccupied ground at a distance equal to one-half the width of the structure.

* "Physical Differences Between Sand and Clay," by Charles Terzaghi, M. Am. Soc. C. E., *Engineering News-Record*, December 3, 1925, Fig. 2.

† "Present Status of Subgrade Testing," by C. A. Hogentogler, Charles Terzaghi, and A. M. Wintermayr, *Public Roads*, March, 1928.

In his very thorough and able discussion, Professor Krynine quotes certain statements made by Dr. Ing. Koegler concerning the relation between settlement and size of area, and finds them to be at variance with those of the writer, as far as soils with great cohesion are concerned. It should be emphasized that Dr. Koegler derived his conclusions exclusively from tests performed with cohesionless sand. For this reason they conform to Curves C and C_2, Fig. 50. For the same reason they certainly do not apply to soils in general, irrespective of their physical characteristics, and Dr. Koegler's generalized statements must be received with reserve until more convincing arguments are presented for substantiating their validity.

Professor Krynine's suggestion to consider, in the computation of a raft foundation, the presence in the ground of hard and soft spots is theoretically sound. However, in practice, it seems very difficult to follow the advice. First, loading tests would be required, not only all over the area, but also at different elevations below the ground surface. Furthermore, since engineers do not yet know the distribution of the soil reactions for perfectly homogeneous foundations, and the importance of the effect of the soil characteristics on this pressure distribution, there is little hope for successfully introducing further refinements into computations.

The doubts expressed regarding the writer's comment on the settlement of the hardpan caissons at the Chicago Union Terminal, were apparently due to the fact that Professor Krynine was not familiar with the details of the tests. The frictional resistance acting over the surface of the caissons was determined by undermining the caissons and loading them until they slipped. The unit loads quoted by the writer obviously represented the top load minus the relieving effect of side friction.

Another objection by Professor Krynine concerns the shape of the curve which represents the distribution of the soil reactions over the base of a loaded slab. When the writer stated that these curves have a parabolic shape, he obviously meant an approximately parabolic one. At the rudimentary state of present knowledge a more specific statement concerning this shape would be premature. In addition, the difference between the right and left sides of Fig. 40 is so slight compared with the importance of the errors associated with Professor Krynine's theoretical assumptions that it seems reasonable to disregard it. Very much more important is the question as to whether the parabolic law may not lose its validity altogether if applied to very cohesive soils. For such soils the pressure distribution is apt to be similar to that over the base of a piston which acts on the plane surface of a homogeneous, elastic solid. The writer doubts seriously whether this distribution has any resemblance at all with either one of the distributions represented by Fig. 40.

Professor Krynine's comments on Fig. 5 are fully justified. The writer agrees that the two lines will be neither straight nor parallel. The reason he failed to show their true shape in the diagram was simple enough. He does not yet know this shape.

The writer became interested in the subject of "entrapped air" for the first time when certain inconsistencies were found to exist between the results of compression tests on clays that were dried before testing and on others that were used as they were taken from the drill hole. The inconsistencies were caused by the fact that those samples which were prepared by mixing the dried specimen with water contained an appreciable quantity of air, while the others contained practically none. In order to determine the importance of the influence of the air on the test results, Mr. A. Casagrande performed some tests in the writer's laboratory. Fig. 53 shows a typical test result. One of the two samples merely contained air and water. The other was prepared from dry powder and, in addition, air was injected into the specimen by a drop counter in such quantities that its voids ratio increased by almost 50 per cent.

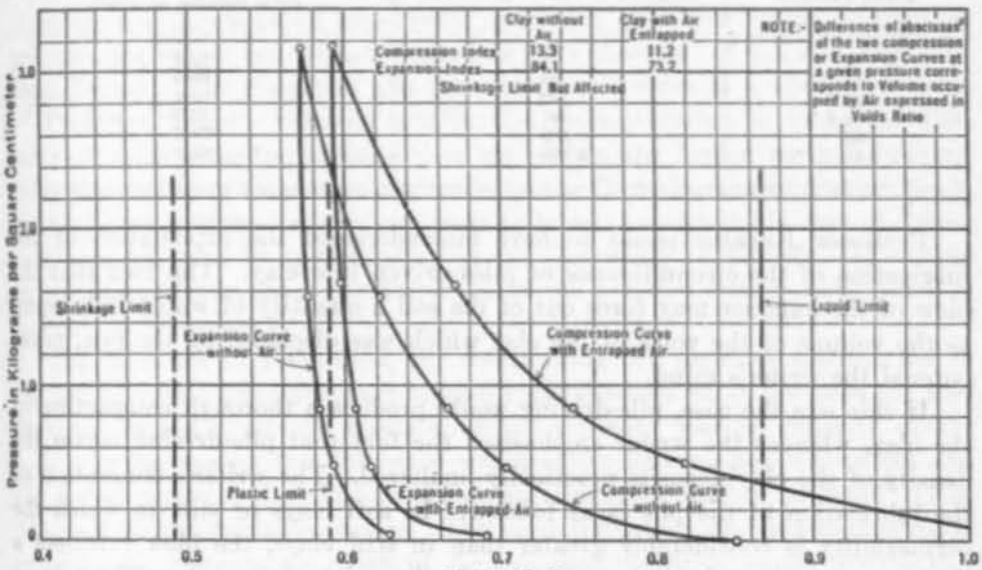


FIG. 53.—EFFECT OF AIR CONTENT ON ELASTIC PROPERTIES OF CLAY.

The most conspicuous effect of the air content was the difference in shape of the time-compression curves of the samples. This is caused by the fact that the compression of the air takes place almost instantaneously, similar to the compression under pressure of the air bubble in a spirit-level, while the compression of the clay occurs gradually, according to the laws of consolidation of compressible capillary systems. By this very reliable criterion, and by the results of numerous air-content determinations, the writer has learned that neither clays nor silts seem to contain in Nature a measurable amount of air, provided they are beneath the zone of temporary desiccation. For this reason the writer did not consider it necessary to mention the air factor in connection with a paper on foundations. It plays an important part merely in connection with sub-grade studies for roads. The intermittent wetting of the sub-grade of hard surface roads occurs almost entirely by capillary action, whereby a considerable quantity of air remains in the soil.

Some experiments on the air content of fine sands and dust, wetted by capillary action, were conducted in 1926 by Mr. C. V. Arellano, under the supervision of the writer.* The test results seemed to indicate that the air

* "Relation between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., 1926.

content decreases considerably with time. One of the samples tested was a fine, reddish, micaceous dust; effective size, 0.028; uniformity coefficient, 3.6; specific gravity of grains, 2.674; voids ratio, 0.698; and coefficient of permeability, 9.75×10^{-5} cm. per sec. The results are shown in Table 12.

TABLE 12.—AVERAGE DEGREE OF SATURATION OF FINE DUST WETTED BY CAPILLARY RISE.

Capillary rise, in centimeters.	Time elapsed since water started to rise, in minutes.	Percentage saturation, or the ratio between total volume of water and total volume of voids.
5	9	75.2
10	38	69.9
15.7	100	70.1
32.9	487	72.2
40	800	73.0
40	3 800	82.7

Professor Krynine seems to have misunderstood the explanation of the lubrication of the circumference of piles driven into clay. The idea that the blow of the hammer may force out of the soil a quantity of water fully equal to the volume of the voids of the clay which was displaced by the pile, never entered the writer's mind.

If this was the case, pile-driving would produce a thorough compacting of the clay, whereas the writer emphasized the fact that pile-driving leaves the density of the clay deposits practically unaltered. The writer's conception of the lubrication of the pile is as follows: In soft clays or silts in which the permeability is considerably greater than in stiff clays, the blow produces a slight compression of the material next to the point of the pile. The volume of water pressed out of the material represents a small fraction only of the volume displaced. Lubrication is caused by this small quantity escaping between the soil and the surface of the pile. The absorption of the film during the period between driving and re-driving is due to subsequent expansion of the slightly compressed clay which surrounds the pile. This process of absorption by no means requires (as Professor Krynine assumes) the presence of entrapped air, because it also occurs in the laboratory under test conditions which completely exclude the presence of air in the clay.

There is also another factor that is apt to cause a gradual increase of the "skin friction". If certain clays and organic muds are moulded into cylindrical shapes and tested at different times after moulding, one is apt to find that the shearing strength increases considerably as the time interval becomes greater. This increase seems to be due to the gradual development of a molecular bond between the individual particles across the layers of absorbed material. If piles are driven into such materials, the skin friction should also increase as time goes on.

Professor Krynine emphasizes the leaning of Russian engineers toward the "dry substance theory". On account of this leaning, Russian engineers seem to be inclined to speak of the "bearing capacity of soils" in general without due regard to their extremely manifold physical characteristics. This, at least, is the impression the writer has obtained from some recent Russian publications on this subject.* More than a half century ago, a prominent French engineer expressed his experiences with earth slides in a single sentence which may be translated: "The fear of frost and of clay marks the beginning of wisdom". In a similar fashion the writer is inclined to summarize his experiences in dealing with the bearing-capacity problem in the sentence: "To recognize the full importance of the permeability factor marks the beginning of wisdom in foundation engineering". Since the permeability factor represents by far the most obstinate stumbling block in the path of any attempt to bridge the gap between small-scale tests with clays and actual foundation practice, the writer cannot help regarding "dry theories" of soil behavior with deep distrust. The history of pile formulas should serve as a solemn warning.

Professor Enger's valuable contribution calls attention to the great difficulty of measuring the stresses that act within the loaded earth. Judging from the results of measurements made by the U. S. Bureau of Public Roads the Goldbeck pressure cells do not seem to be any more reliable than the pressure capsules used by investigators at the University of Illinois. It will be interesting to learn about the properties of the pressure cells recently used by Koegler in Germany.†

The formulas of Boussinesq for determining the pressure distribution in the soil beneath loaded areas are at least approximately valid, provided the load does not exceed a certain fraction of the ultimate bearing capacity of the soil. For expressing the difference which exists between the theoretical and the actual pressure distribution, Professor Enger's "lines of pressure" (Fig. 48), will render valuable service. The most important problem that remains to be solved concerns the effect of the soil characteristics on the distribution of the pressures over the base of the loaded area.

The objection of Professor Enger against the use of remixed material does not seem to be quite justified. In the example which he cites, the compacting of the clay was performed by the capillary pressure at a time when the water-table was at a lower elevation. Once compacted, the clay takes up only a small fraction of the quantity of water which it previously lost. The process can be reproduced very easily in the laboratory by first compacting the clay under external or capillary pressure (applying the pressure under water or exposing the surface of the clay to the air) and then allowing it to expand very slowly under water.

In Nature one even encounters layers of very stiff clay beneath the bottom of lakes, between layers consisting of the same material in a very soft state. The history of these "fossil crusts" is as follows:‡ At a period when the surface of the older clay deposit was exposed to the air, the top layer of this deposit partly dried out (compression due to capillary pressure). At a later date, the surface of the older deposit was flooded, and a new soft clay deposit was formed on top of it. Due to the very imperfect volume elasticity of the

* Brief abstract of chapter on "Bearing Capacity," by Mr. N. Ivanoff, in Textbook on Soils and Top Soils, of the Russian Highway Research Bureau, Leningrad, 1926.

† "Ueber Tragfähigkeit von Sandschüttungen," *Bautechnik*, 1927.

‡ Statens Jarnvägars Geotekniska Comm., 1914–22, English abstract and comments by Charles Terzaghi, M. Am. Soc. C. E., filed in Carnegie Library, Pittsburgh, Pa.

clay, the former crust retained the greater part of its stiff consistency (see, for instance, the re-saturation or swelling curves in Fig. 53).

Since the paper was published, much headway has been made in developing methods for securing undisturbed samples of clay.* Tests are being planned to investigate the relative importance of the disturbance caused by different sampling tools. There seems also to be no difficulty in securing undisturbed clay samples from depths as great as 100 ft.

The effect of vibrations on the settlement of structures resting on clay was brought to the writer's attention under rather peculiar circumstances. Loading tests were made on a small scale on glacial clay, in his laboratory, and very sensitive instruments were used for measuring the settlements. All the settlement curves showed distinct breaks at points corresponding to 5:00 P. M. and 9:00 A. M., the 9:00 A. M. to 5:00 P. M. sections being considerably steeper than the 5:00 P. M. to 9:00 A. M. sections. The cause of this phenomenon was found to be that between 9:00 A. M. and 5:00 P. M. the different motors located in the Massachusetts Institute of Technology Laboratories are operating, while at 5:00 P. M., they are shut down.

The tests suggested by Mr. Jorgensen will be particularly valuable if they are made in triplicate, using identical bearing plates, resting on identical clay water mixtures, so that one of the bearing plates consists of cast iron (impermeable), one of concrete (medium permeable), and one of concrete separated from the clay by a sheet of filter paper and a thin layer of sand (permeable). The diameter of the bearing plates should not be more than one-fifth that of the container. The exposed surface of the clay should be covered with a paraffined rubber sheet in order to prevent evaporation, and the settlements should be measured with delicate instruments to detect the difference in the speed of settlement of the three plates. During the tests, the load should be kept constant, and should be equal to the pressure required to produce an initial settlement of the cast-iron bearing plate equal to about 0.05 or 0.1 times its diameter. In each of the three cases the load should be applied in equal increments and in equal time intervals. In order to obtain the maximum benefit from the tests, the base of the bearing plates should be equipped with annular oil-pressure cells, for investigating the distribution of the soil reaction over the case.

The procedure proposed by Mr. Jorgensen for load-moisture tests is practically identical with that of the writer. Since the shape of the re-compression curves, *c d* (Fig. 49), is only slightly influenced by the intensity of the pressure at which swelling was begun, no great care is necessary in performing the test.

In conclusion, the writer wishes to express his gratitude to the discussors for their valuable contributions. The gaps in the presentation of the subject are still numerous. Yet, the arguments brought forth offered a unique opportunity for expanding the paper on those points which were still in doubt in the minds of the readers.

* "Relation Between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., Cambridge, Mass., 1926.

The results of the discussion may be summarized, as follows:

1.—The *Engineering News Formula* received all the comment it deserves and few engineers seem to be willing to defend it.

2.—The attitude of the different engineers toward the writer's theoretical statements represented by Fig. 50 was far less unanimous, yet it leads to very interesting conclusions because no obvious contradiction between theory and practice was found to exist by those who reported experiences with fairly permeable materials (Messrs. White, with friable limestone; Shaw, with muck; and Wing, with sand and gravel). Without exception, all the examples which were quoted as arguments against the validity of the writer's "rules" referred to clays. The reason thereof is obvious. The compression of clay under load proceeds very slowly. Large footings or raft foundations require decades or even hundreds of years for complete settlement, while the observations on which the arguments were based covered, at the very best, a period of a couple of years of the settlement records. In other words, the bill for the excessive loading of a sand or silt foundation should be paid without delay by the first generation, while that for treating the clay in a similar manner should be charged against the second or third generation.

It is amazing to realize that the Engineering Profession is facing the problem of progressive subsidence of clay foundations in almost every part of the globe, and yet is still completely ignoring the time element when it comes to approaching the settlement problem from the theoretical point of view. If the discussion had not brought forth anything but striking illustrations of this one fact, the labor involved could be considered as well invested.