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PART I

The University of Texas
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NOTICE

In order to make available as many papers as possible to those attending the Conference, Part I of the Proceedings is issued in advance and will be followed by Part II published after the meeting

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PART I

SHEAR FAILURE OF SOILS

by

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In 1773 a theory for the shearing strength of soils as applied to earth pressure on retaining walls was published by Charles Auguste de Coulomb.(1)* In this theory he assumed that shearing strength can be divided into two components, one of which depends directly on the normal pressure acting on the plane of shear while the other is independent of normal pressure. Expressed mathematically the theory becomes the now familiar Coulomb equation or formula

$$S = c + \sigma \tan\phi \qquad (1)$$

in which S = shearing strength per unit area.

c = The component of shearing strength per unit area which is independent of normal pressure, commonly called the "cohesion" or "unit cohesion".

σ = normal stress per unit area on the plane of shear.

$\tan\phi$ = coefficient of proportionality between normal stress and that portion of the shearing strength which depends directly on normal stress, sometimes called the "coefficient of friction".

ϕ = the angle of internal friction for the soil.

When c = 0 in equation (1) the soil is called "cohesionless", a common condition for sands and gravels. Other soils are cohesive and, according to

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* Numbers in parentheses refer to references in bibliography at end of this paper.

Coulomb, their shearing strength is expressed by equation (1) with c having a positive value. Later writers have interpreted ϕ to be the angle of repose of the soil which it is not and which was not intended by Coulomb.

In the one hundred and sixty-nine years since the publication of Coulomb's theory, and particularly in the last twenty years, much research and many papers have been devoted to the shearing strength of soils. In this mass of material there are many significant results but none of them compare in importance with Coulomb's conclusions expressed by equation (1). The evidence for this statement is found in the fact that most existing theories for bearing capacities of footings, earth pressures on walls and abutments, and stability of cuts, fills, and earth dams are based on equation (1). The most significant recent modifications have been concerned with the effects of pore water pressures on σ and the effects of previous stress history of the soil on c in the basic Coulomb equation.

The importance of the applications of equation (1) cannot be denied. Bearing capacities, earth pressures, and stability analyses include the most economically important problems in earthwork engineering. Safety of engineering structures and economic utilization of materials and labor require not only confirmation of pressure and stability theories by field observations but also careful checking in the field and in the laboratory of the shearing properties of soils used in these theories.

Coulomb's equation is limited to the shearing <u>strength</u> of soil at the point of failure. It does not apply to shearing <u>resistances</u> less than the maximum strength. In spite of many hypotheses for the relative magnitudes of the cohesive and frictional components of shearing resistance before failure occurs, no definite evidence is yet available for a division except at failure. Therefore Coulomb's equation is the statement of a theory for the failure characteristics or the limiting conditions of stress in soil. Soil is a natural material differing from other engineering materials only in that it is used as found in nature, its degree of resistance to stress is less, and it usually consists of a mixture of mineral grains and water, each of which resists stress in a different way. The limiting conditions of stress for other engineering materials have been studied extensively. These conditions, with the fundamental concepts of applied mechanics which can be applied to failure conditions, form a rational basis for analyzing the limiting conditions of stress in soils.

REVIEW OF PRINCIPLES OF FAILURE

Elementary applied mechanics has demonstrated that the stresses in any material in static equilibrium must fulfill certain conditions which are independent of the material and of the magnitudes of the stresses. This theory can be reviewed by considering a static condition of plane stress (stresses in two dimensions only) at a point in a solid. If the stresses acting on any two perpendicular planes passing through the point in question can be determined, the stresses acting on any plane through this point and inclined to the assumed coordinate planes can be calculated from the equations of statics. The results are shown in Fig. 1. From equation (4) for $\tau = 0$, it can be seen that on two perpendicular planes passing through the point the shearing stress is zero. These planes are called <u>principal planes</u> and the corresponding normal stresses acting on these planes are calles <u>principal</u> stresses.

If the coordinate axes for the prismatic element in Fig. 1 are rotated to a position such that they coincide with the principal planes, equations (2) and (3) in Fig. 1 are simplified because no shearing stress will now exist on the coordinate planes. The simplified equations are shown in Fig. 2, with the normal stresses acting on the principal planes designated by σ_1 and σ_3. Equations (5) and (6) can be illustrated graphically by a method developed by Otto Mohr(2) which is now known as Mohr's stress diagram or Mohr's circle of stress. In this method values of σ are plotted along a horizontal axis and values of τ perpendicular to this axis. Then, if σ_1 and σ_3 plotted on the horizontal axis from an origin O, determine the diameter of a circle, its radius will be $\frac{1}{2}(\sigma_1 - \sigma_3)$. Also the distance from the origin to the center of the circle will be $\frac{1}{2}(\sigma_1 + \sigma_3)$. Thus the distances marked σ and τ on Fig. 2 are graphical representatives of equations (5) and (6).

All combinations of normal and shearing stress acting on any plane passing through the point, in the case of plane stress, will be represented by points on the circumference of the circle passing through points C and D. Thus of all the normal stresses acting on planes passing through the point σ_1 will be the largest and σ_3 the smallest. σ_1 is therefore known as the major principal stress and σ_3 is known as the minor principal stress. Every other plane passing through the point will have acting on it not only normal stress but also shearing stress.

If stresses in three dimensions are considered, it is found that on a third principal plane, which is perpendicular to the other two planes, there will be zero shearing stress. The normal stress acting on this plane is known as the intermediate principal stress, designated by σ_2, and may have any value between σ_1 and σ_3. Statical analysis in three dimensions, developed by Westergaard(3), has demonstrated that Mohr's stress diagram for a three-dimensional condition of stress consists of three circles with diameters ($\sigma_1 - \sigma_3$), ($\sigma_1 - \sigma_2$), and ($\sigma_2 - \sigma_3$) on the horizontal axis. Every combination of stress on any plane passing through a point in a static solid is represented on the stress diagram by a point which falls within the area bounded by the three stress circles, as shown in Fig. 3.

If the intermediate principal stress is equal to either the major or the minor principal stress, all possible combinations of normal and shearing stress will again be represented by points on the circumference of a circle defined by the major and minor principal stresses as shown in Fig. 2. In most cases in materials testing, either the major or the minor principal stress is the result of applied load while the intermediate and the remaining principal stress, on the basis of externally applied loads, are equal to zero. In the application of Soil Mechanics to the solution of various problems, the intermediate principal stress can frequently be assumed equal to the major or minor principal stress. In other problems in which axial symmetry does not exist and in which plane stress cannot be assumed, the intermediate principal stress may fall anywhere between the major and minor principal stresses.

Mohr's stress diagram has several advantages as means of investigating and showing graphically the limiting conditions of stress in materials. It is dependent only on the existance of static equilibrium. If the true internal principal stresses in a material at the point of failure are known, the corresponding Mohr's stress circle represents definitely the limiting stress conditions for the particular type of loading. Other types of loading yield other limiting stress circles and the envelope of all such circles, known as the line of rupture, includes all possible stress combinations which the material can sustain. Such a diagram is shown in Fig. 4, with compressive and tensile normal stresses plotted to the right and left respectively of the axis for zero externally applied normal stress.

The properties of this diagram should be noted well. Normal stresses are plotted on the horizontal axis. In the case of hydrostatic pressure only normal stresses exist. Since there are no shearing stresses, all three principal stresses must be equal and the Mohr's stress circle is a point on the horizontal axis. Conversely, in a material which has no resistance to shearing forces at rest (a liquid) the stresses in all directions must be equal. If, in a case of static equilibrium, normal stresses in various directions have different magnitudes, shearing stresses must exist. The magnitude of the maximum difference between the major and minor principal stresses will depend on the ability of the material to resist shearing stresses, in other words, on its shearing strength. Tests by Bridgman(4,5,6) on materials under hydrostatic pressures of 200,000 to 300,000 pounds per square inch have shown that hydrostatic pressures far in excess of normal compressive strengths do not cause rupture or plastic yielding. In fact Bridgman found that the volume change of most materials under large hydrostatic pressures is almost perfectly elastic. Therefore a limit on the compression side of the normal stress axis is not probable.

Testing materials under hydrostatic tension is not feasible, although Bridgman found that, in an accidental sudden release of a very large hydrostatic pressure, the suddenly applied rebound hydrostatic tension reduced a glass sphere to an "impalpable powder". There are many other test indications that a definite limit exists for all materials on the tension side of the normal stress axis. The stress represented by the distance between this hypothetical but probable tensile limit and the origin for externally applied normal stresses may be called the "intrinsic pressure" of the material.* It depends on the physical and chemical constitution of the material, its crystalline structure and intermolecular attractions.

In plotting normal stresses on Mohr's stress diagram two points of view may be taken. First, the origin for normal stresses may be taken at the point where externally applied stress is zero, compressive stresses being plotted to the right of the origin and tensile stresses to the left of the origin. Or second, in the same plot it may be considered that all stresses in the material are compressive and that the origin for externally applied stress is the intrinsic pressure, assuming it to be equal in all directions. From the second point of view tension will result only in a reduction of the intrinsic pressure and the actual stress will be compression of a smaller magnitude. Thus, if a cylinder is loaded in compression in the z-direction, the major principal stress will be equal to the applied stress plus the intrinsic pressure. The intermediate and minor principal stresses in the x- and y-directions will be equal to the intrinsic pressure

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* In connection with the resistance of metals to failure the value referred to here as "intrinsic pressure" is called the "cohesive strength" or "technical cohesive strength" as opposed to the theoretical cohesive strength of pure metal crystals. It is defined as the ultimate resistance of the metal to triaxial or hydrostatic tension and has been approximated by static tension tests on notched specimens in which the notches vary in depth and angle. The stresses in the metal immediately inside the base of the notch approach hydrostatic tension as the sharpness and relative depth of notch increases. (See W. Kuntze, "Kohasionsfestigkeit", Mittelungen der deuschen Materialprufugsanstalten, Sonderheft XX, Julius Springer, Berlin, 1932, and "Symposium on Significance of the Tension Test of Metals in Relation to Design", pp. 501-609, Proc. Am. Soc. T. M., v.40, 1940, particularly p. 515 of paper by C. W. MacGregor and p. 539 of paper by F. B. Seely.) In connection with failure of soils the term "intrinsic pressure" is preferable to avoid confusion with the well established usage of cohesion in Coulomb's equation as the shearing strength under zero normal pressure.

minus the stresses created by lateral deformation of the cylinder as it decreases in length. If this same material is loaded in tension in the z-direction, the major and intermediate principal stresses will be equal to the intrinsic pressure plus the stresses due to lateral deformation while the minor principal stress will be equal to the intrinsic pressure minus the applied stress. Thus two extreme conditions in the variation of the intermediate principal stress are obtained. Tests by von Kármán(7), Böker(8), and Richart, Brandtzaeg, and Brown(9) on marble and concrete indicate a difference of approximately 15% between lines of rupture for these two conditions, neglecting the stresses due to lateral deformation. Their data are insufficient to evaluate the stresses due to lateral deformations but it is easily possible that, if true internal stresses were known, a single line of rupture would be obtained.

Existing theories for limiting conditions of stress in materials have been applied primarily to tests on metals. They can be grouped as follows: maximum stress theories (Rankine's theory and variations) which assume that failure results from normal stresses exceeding a limiting value; maximum strain theories (St. Venant's theory and variations) which assume that failure results from longitudinal or shearing strains exceeding a limiting value; maximum energy theories which assume that some function of the energy of distortion limits the stresses which material can carry; and shear theories (including Coulomb's theory, Guest's maximum shear theory, and Mohr's general shear theory) which assume that failure is governed by limiting values of shearing stress or of a combination of shearing stress and normal stress. These theories, as applied to metals, have been discussed in detail by Westergaard(10) and Marin(11) and some of their implications have been analyzed by Gensamer.[12]

The maximum stress theories do not take into account Bridgman's tests at large hydrostatic pressures. Maximum strain theories cannot be applied to rupture because many tests have shown that strain at rupture in a given material varies widely with the loading conditions. Maximum energy of distortion theories were developed to account for observed deviations from the other theories of the results of torsion and combined torsion and tension tests. Mohr's general shear theory, which conforms best with existing test results for all materials, has always been applied to stresses at failure due to external loads, neglecting completely the internal stresses caused by deformations prior to failure.

The theory which is most generally applicable to all materials is a modification of Mohr's general shear theory. It has, in effect, been presented in the preceding pages. The two basic modifications of Mohr's theory are as follows:

(1) The failure stresses plotted in Mohr's rupture diagram must include internal stresses due to lateral deformation prior to failure (modification proposed by the author).

(2) Failure is governed both by a limiting combination of normal and shear stresses, as proposed by Mohr, and by a limiting tensile stress or tensile strain. (Modification proposed by Böker and Brandtzaeg.)

The first of these modifications explains simply and directly the observed deviations between the results of torsion tests on steel and Mohr's theory, in which only stresses due to external loads are considered. Sufficient test data are not yet available to prove that this modification accounts completely for observed discrepancies caused by the two extreme conditions of the intermediate principal stress in tension and biaxial compression tests as compared with compression and triaxial compression tests.

The second modification, which includes the intrinsic pressure hypothesis described in the preceding pages, accounts for many observed phenomena in the failure of brittle materials like cast iron, natural stone, concrete, and natural cemented sands. It is a common observation that vertical tension cracks appear on the circumferential surface of a concrete cylinder immediately prior to failure in simple compression. These cracks are caused by the circumferential strains and stresses which accompany the increase in diameter of the cylinder as it decreases in length. Their result is brittle failure at approximately one-half per cent strain. Tests on marble, sandstone, and concrete (references 7, 8, and 9) show that these materials, which are brittle in simple compression, can sustain axial strains of three to seven per cent in triaxial compression without disintegration. Concrete specimens in these tests bulged laterally without rupture and, when retested in simple compression, sustained an average of sixty-nine per cent of the maximum load carried by undeformed specimens. The reason for the change from brittle to plastic behavior in triaxial compression is found in the lateral pressure. This pressure, added algebraically to the radial and circumferential tensions resulting from deformation, prevents internal lateral stresses from reaching the limiting tensile strength.

Precisely the same phenomena are observed in compression and triaxial compression tests on natural clays and on soil mixtures. Many natural clays which fail by rupture along definite shear planes in unconfined compression tests become progressively more plastic in triaxial compression with increasing lateral pressures and decreasing rates of deformation. The results of triaxial compression tests on sand and gravel mixtures are almost duplicates, at a different scale, of the results of similar tests on concretes. Therefore the modified general shear theory seems well adapted to the study of soil test results. Coulomb's theory for the shearing strength of soils is a special case od the modified general shear theory in that equation (1) represents a rupture line which is a straight line.

RESULTS OF TESTS ON SOILS AND OTHER ENGINEERING MATERIALS

The extent of applicability to soils of the Coulomb theory, as a special case of the modified general shear theory, depends on how well it fits test results and field behavior of soils. Other common engineering materials which exhibit failure phenomena similar to soils are less complex in character, are familiar to all engineers, and have been thoroughly investigated. Therefore a review of results of tests on such materials provides a close parallel to soil test results and a preliminary survey of the applicability of Coulomb's theory and the modified general shear theory to all engineering materials. These test results are presented in terms of stresses due to externally applied loads because the data are not sufficiently complete to evaluate the additional stresses caused by deformations prior to failure.

The most complete tests available on brittle materials have been previously mentioned (references 7, 8, 9). Fig. 5 and 6 show the results of tests on marble and sandstone by von Kármán[7] and on marble by Böker[8]. Fig. 7 shows the results of tests on three concrete mixes by Richart, Brandtzaeg, and Brown[9]. Each circle of stress in these figures represents the stress conditions at failure for one test or, in the case of Fig. 7, the average of four identical tests. The effect of correcting the circles of stress for the additional stresses caused by deformations prior to failure would be, for each test, a decrease in the minor principal stress and an increase in the diameter of the stress circle. The changes in diameter would be neither equal nor proportional to the diameters but would depend on the lateral strains and the relation between strain and decreasing compressive stress or tensile stress. For unconfined compression tests and tests under small lateral pressures they would be limited by the intrinsic pressure. Thus the shape, position and inclination of the line of rupture for each group of tests would be affected. The results of the tests on concrete indicate that Coulomb's equation provides a reasonable approximation for the stress conditions at failure. Coulomb's equation cannot be applied to the results of the tests on marble and sandstone as presented in Fig. 5 and 6 but the results do not conflict with the modified general shear theory.

While the resistance of steel to applied stresses is unlike that of soil, it is a material familiar to all engineers. Fig. 8 shows results of tests on steel by Seely and Putnam(13) in which stresses at the yield point rather than at rupture are plotted. The full line stress circles are plotted in terms of externally applied stresses showing how the results of torsion tests do not conform with Mohr's theory. The same results are shown by dash lines with the tension and compression test results corrected for lateral stresses due to deformation. The torsion test results then coincide exactly with the compression and tension test results, which again substantiates the modified general shear theory. The shape of the rupture line for steel beyond the limits shown is not known.

Turning to the results of tests on soil, Fig. 9 and 10 show lines of rupture determined by tests performed under the author's direction on loose and dense gravel. Fig. 11 shows the results of tests on angular and found grained sands by Watson.(14) In reference to the curved lines of rupture Watson attributes the curvature to anisotropy caused by orientation and interlocking of angular grains, pointing out that it is greatest in dense angular sand and least in round grained material. Böker and Brandtzaeg have also studied carefully the effects of anisotropy on the limiting conditions of stress. The marked similarity between the results of tests on granular soils shown in Fig. 9, 10, and 11 and the results of tests on concrete shown in Fig. 7 should be noted. Differences appear only in the cohesive effect of the cement in providing shearing strength under zero normal pressure and in the absolute magnitudes of the stresses.

In Fig. 12 the results of tests on a clay typical of those used in earth dam and stabilized road construction are shown. This soil was compacted in cylindrical molds to three different densities and tested in a relatively dry condition. Fig. 13 shows the results of tests on the same material mixed with varying percentages of a bitumen and tested relatively dry and at a relatively low density. Both sets of results are almost identical to the results of the tests on granular soils. The important differences between the properties of clay and granular materials appear only when the voids of the clay are filled with water. Therefore the limiting conditions of stress for natural clays and saturated soils which cannot drain freely must be analyzed by separating the effects of applied stress on mineral grain structure of the soil and on the water which fills its voids.

RESULTS OF TESTS ON SATURATED SOILS

At the Fourth Texas Conference on Soil Mechanics and Foundation Engineering last year, Van Auken(15) presented the results of tests on saturated cohesionless soils in which pressures in the water filling the soil voids were varied in arbitrary ways and were measured. This work developed from an analysis by Casagrande(16) of the possible effects of large pore water pressures on the shearing resistance of cohesionless soils. Pressure in static water is equal in all directions. If water in the voids of a saturated, stressed mass of soil is under pressure, the effective normal stresses carried by the mineral grains (see reference 17) will be equal to the stresses due to externally applied loads reduced equally in all directions by the amount of the pore water pressure. In terms of the limiting circle of stress for a specific loading condition, equal reduction of all normal stresses means moving the stress circle for externally applied stresses toward the origin by the amount of the stress carried by the water. Failure occurs when the stress circle corrected for the stresses carried by the water becomes tangent to the line of rupture for the material determined by tests in which pressures in pore water do not exist. Van Auken's tests and others have substantiated the conclusion that water filling the voids of cohesionless soil has no other effect on its shearing strength. Therefore, if pore water pressures in saturated cohesionless soils can be measured or determined by some other means, test results can be corrected to give the true limiting conditions of stress. Similarly the limiting conditions of stress can be used in design analyses if the pressures in the pore water can be anticipated. The conclusion is, therefore, that Coulomb's theory applied to cohesionless soils is not affected by saturation, provided corrections are made for pore water pressures.

An analogous situation exists with respect to saturation of clays but, due to the impervious character of most cohesive soils, pore water pressures have not been measured successfully and are doubly important because of the tendency of such soils to compress and transfer stress to pore water under normal as well as shearing stresses. Fig. 14 shows the results of two sets of consolidated quick triaxial compression tests on compacted clay used for earth dam construction. The first set of tests, shown by dash lines, was performed on specimens 85% saturated. The second set, shown by full lines, was performed on saturated specimens. The observed reduction in strength is due partly to the stresses in the pore water, for which no correction was made, and partly to the elimination of capillary forces by saturation. Similar results were shown by Van Auken(18) for compacted soils used in the Denison Dam.

The results of tests on a natural saturated clay are shown in Fig. 15. These tests on undisturbed samples, each set of tests being performed at a different rate of strain. The uncorrected results are shown by the full line circles and lines of rupture in Fig. 15. The results show that the slower rates of strain, which permit more water to flow out of the consolidating clay and thus reduce pore water pressures, produce larger resistances to stress. These test results have been corrected approximately for pore water pressures by comparing the triaxial compression test void ratios with the void ratio-pressure relationships obtained from parallel consolidation tests. This approximate method of correction was devised by H. H. Ku(19) under the author's supervision. The corrected results are shown by dash line circles and lines of rupture in Fig. 15. The average corrected line of rupture for all five sets of tests is shown by straight dot-dash lines on the diagram for each set of tests in Fig. 15. The results have not been corrected for the radial and circumferential tensions caused by deformations of the specimens prior to failure.

If results of the type shown in Fig. 15 are to be applied to problems involving the shearing resistance of natural undisturbed clays, another complication enters. The average natural void ratio of this clay is 0.759. The void ratios at maximum stress corresponding to each test circle of stress are shown in Fig. 15. Table 1 shows complete average data for the quick and slow tests in this series. Both the void ratios at one per cent strain and at maximum stress in these consolidated triaxial compression tests are decidedly smaller than the average initial void ratio. Some of the increase in strength with increasing lateral pressure is caused by the lateral stress and some by the decrease in void ratio. At present it is impossible to separate the effects of these two factors.

If the changes in void ratio are neglected, the results in Fig. 15 show that Coulomb's equation describes the shearing strength of natural clay with practical accuracy. Because of the changes in void ratio under test and because of inevitable pore water pressures in large masses of natural clay, Terzaghi has recommended repeatedly that Coulomb's equation be applied to clays assuming $\phi = 0$. This recommendation assumes that all normal stresses in clay due to applied loads are transferred to the pore water and that the line of rupture is parallel to the horizontal axis. The shearing strength of the clay is thus determined by the results of an unconfined compression test on a good undisturbed sample. Terzaghi's recommendation has been confirmed by field observations.

This discussion has not considered the effects of previous stress history on the shearing strength of clays or the very important influences of sample disturbance on laboratory determinations of natural shearing strengths (see reference 20). The resistances of materials to shearing stresses less than their shearing strengths and the relations between stresses and deformations have not been included. The results discussed can be applied only to problems which are dependent on the assumption of rupture or failure of materials.

CONCLUSIONS

The test results presented herein show that three theories for the failure of engineering materials deserve serious consideration. These are: (1) Coulomb's theory of friction and cohesion; (2) Mohr's general shear theory; and (3) the modified general shear theory. Coulomb's theory is a special case of Mohr's theory and both are included in the modified general shear theory. Present indications are that failure of all engineering materials of importance to Civil Engineers can be explained by the modified general shear theory. For application to practical problems in earthwork engineering Coulomb's equation can be used to express the results of tests on soils, provided the effects of volume changes and pore water pressures are fully evaluated both in the tests and in the applications, and provided the effects of lateral deformations and limiting intrinsic pressures are understood and considered in the analysis of laboratory tests and field applications. With the above limitations the designing engineer can take full advantage of the increased strength of soils under conditions of combined compressive stresses.

TABLE 1

AVERAGE RESULTS OF TRIAXIAL COMPRESSION TESTS ON UNDISTURBED CHICAGO CLAY

Note: All specimens were prepared from one 12" cubical undisturbed sample cut out by hand in Chicago Subway excavation.
All specimens were fully consolidated before test.

Natural void ratio	0.762	0.762	0.754	0.755	0.743	0.758	0.769	0.765	0.758	0.777
Hydrostatic pressure	0	0	0.5	0.5	1.0	1.0	2.0	2.0	4.0	4.0
Type of test	Quick	Slow	Quick	Slow	Quick	Slow	Quick	Slow	Quick	Slow
*Deviator stress	0.267	0.275	0.540	0.672	0.722	0.768	1.10	0.90	1.62	1.63
Modulus of deformation	26.7	27.5	54.0	67.2	72.2	76.8	110	90	162	163
*Void ratio	0.762	0.762	0.708	0.685	0.687	0.690	0.635	0.645	0.570	0.575
*Poisson's ratio $= \mu$	----	----	0.46	0.31	0.47	0.31	0.48	0.32	0.47	0.28
*Stress in pore water	----	----	0.52	0.50	0.69	0.64	1.09	0.78	1.74	1.29
*Lateral soil stress $= \sigma_3$	----	----	-0.02	0.00	0.31	0.36	0.91	1.22	2.26	2.71
*Vertical soil stress $= \sigma_1$	----	----	0.52	0.67	1.03	1.15	2.01	2.12	3.88	4.34
Modulus of elasticity = E	26.7	27.5	54	67	75	90	114	134	176	282
Max. Deviator stress	0.46	0.42	0.73	0.92	1.26	1.50	1.62	2.64	3.13	5.31
Void ratio at max. stress	0.762	0.762	0.703	0.670	0.672	0.630	0.587	0.575	0.533	0.512

All stresses are given in kilograms per square centimeter.

* Indicates values from test results at 1.0 per cent strain.

Modulus of deformation = slope of deviator stress-axial strain curve.

Modulus of elasticity computed from $E = \frac{1}{\text{strain}} (\sigma_1 - 2\mu\sigma_3)$.

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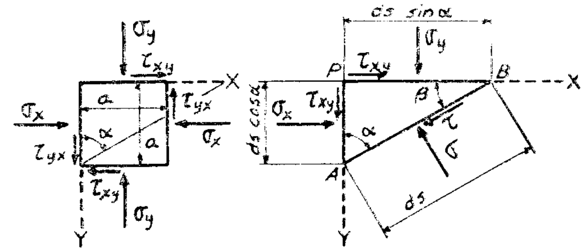


$\Sigma M = 0 \qquad a^2\, \tau_{xy} = a^2\, \tau_{yx} \qquad\qquad \tau_{xy} = \tau_{yx}$

$\Sigma H = 0 \qquad \sigma_x\, ds \cos\alpha + \tau_{xy} ds \sin\alpha - \sigma ds \cos\alpha - \tau ds \sin\alpha = 0$

$\Sigma V = 0 \qquad \sigma_y ds \sin\alpha + \tau_{xy} ds \cos\alpha - \sigma ds \sin\alpha - \tau ds \cos\alpha = 0$

SOLVING THE EQUILIBRIUM EQUATIONS FOR σ AND τ

$$\sigma = \sigma_x \cos^2\alpha + \sigma_y \sin^2\alpha + 2\, \tau_{xy} \sin\alpha \cos\alpha \qquad (2)$$

$$\tau = (\sigma_x - \sigma_y) \sin\alpha \cos\alpha + \tau_{xy}(\sin^2\alpha - \cos^2\alpha)$$

$$\tau = 1/2\, (\sigma_x - \sigma_y) \sin 2\alpha - \tau_{xy} \cos 2\alpha \qquad (3)$$

BY ROTATING THE COORDINATE PLANES AN ANGLE α' SUCH THAT

$$\tan 2\alpha' = \frac{2\tau_{xy}}{\sigma_x - \sigma_y} \qquad (4)$$

THE SHEARING STRESSES ON THE ROTATED COORDINATE PLANES WILL BECOME ZERO. THESE ARE THE PRINCIPAL PLANES.

FIG. 1 - STATIC SOLUTION FOR PLANE STRESS AT A POINT

$$\sigma = \sigma_1 \cos^2\alpha + \sigma_3 \sin^2\alpha$$

$$\sigma = 1/2\, (\sigma_1 + \sigma_3) + 1/2\, (\sigma_1 - \sigma_3) \cos 2\alpha \qquad (5)$$

$$\tau = 1/2\, (\sigma_1 - \sigma_3) \sin 2\alpha \qquad (6)$$

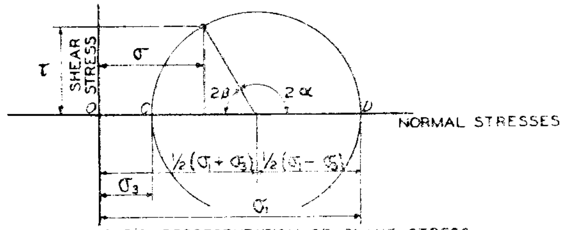


FIG. 2 - MOHR'S REPRESENTATION OF PLANE STRESS

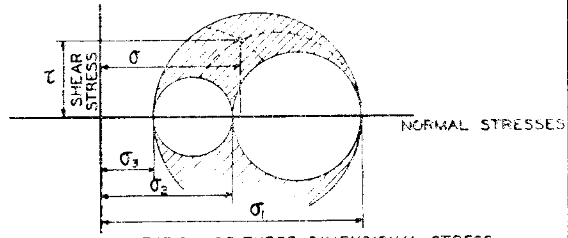


FIG. 3 - REPRESENTATION OF THREE DIMENSIONAL STRESS

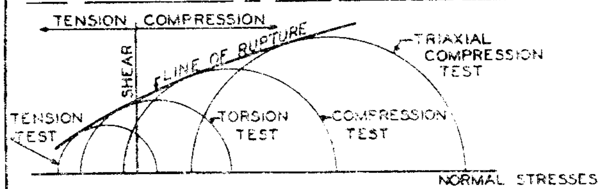


FIG. 4 - DIAGRAM FOR LIMITING CONDITIONS OF STRESS

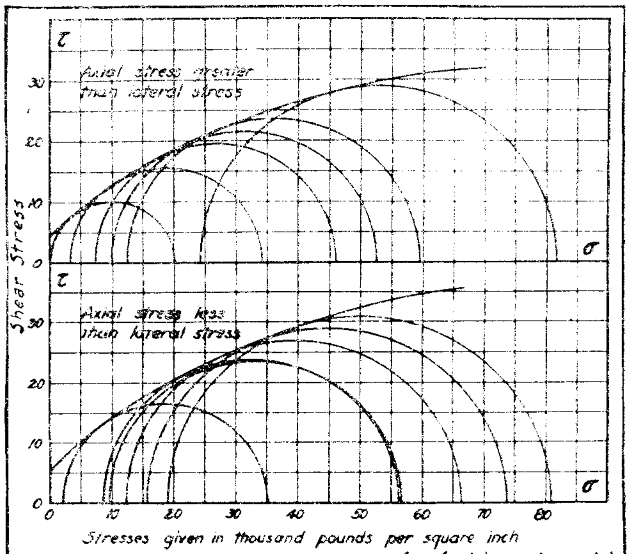


FIG. 5 - TESTS ON MARBLE BY VON KÁRMÁN (7) & BÖKER (8)

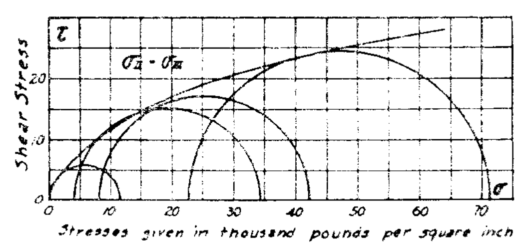


FIG. 6 - TESTS ON SANDSTONE BY VON KÁRMÁN (7)

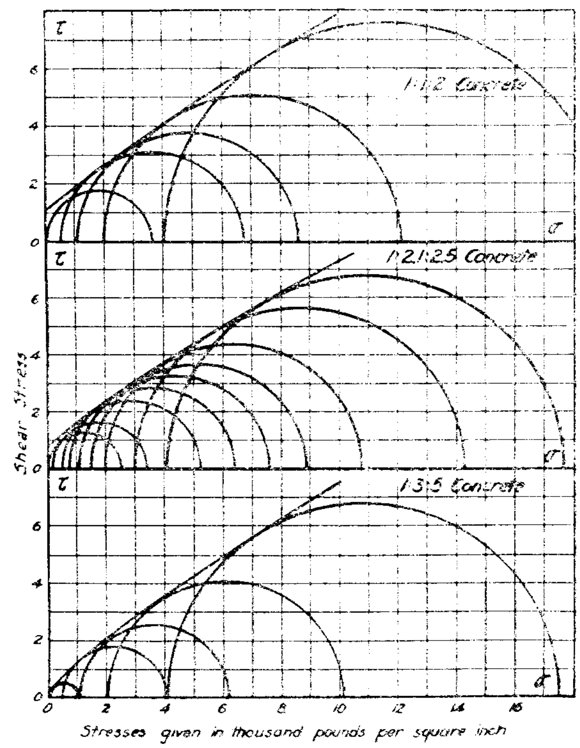
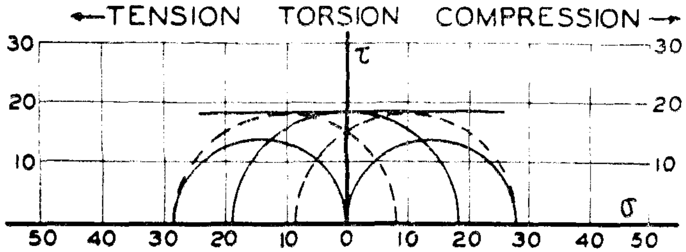
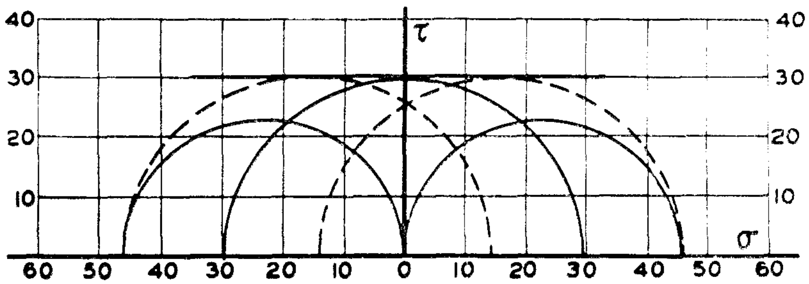


FIG. 7 - TESTS ON CONCRETE IN TRIAXIAL COMPRESSION BY RICHART, BRANDTZAEG & BROWN (9)



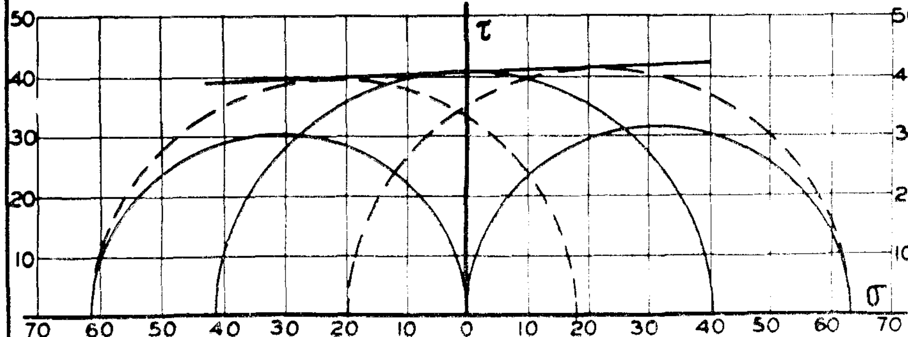
SOFT STEEL

STRESSES GIVEN IN THOUSANDS OF POUNDS PER SQ. IN.



MEDIUM STEEL

ALL STRESSES SHOWN AT YIELD POINT



VANADIUM STEEL

DASH LINE STRESS CIRCLES CORRECTED FOR LATERAL STRESSES DUE TO DEFORMATIONS.

FIG. 8 - TESTS ON STEELS BY SEELY & PUTNAM

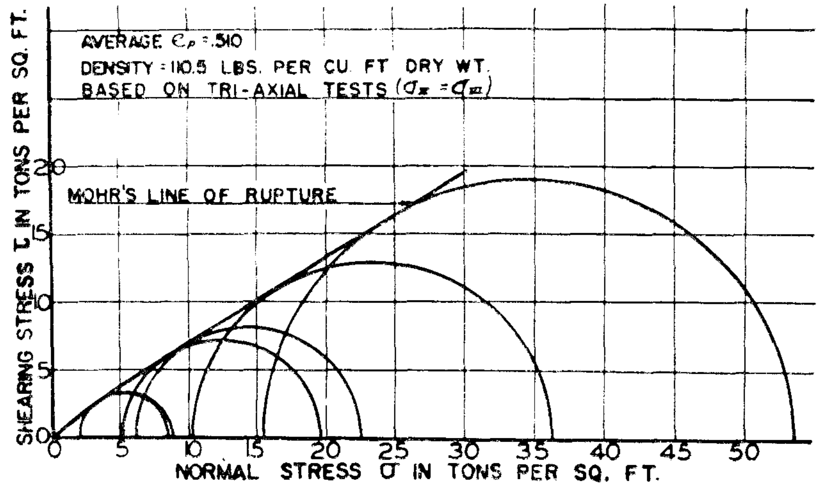


FIG. 9 - TESTS ON LOOSE PIT-RUN GRAVEL

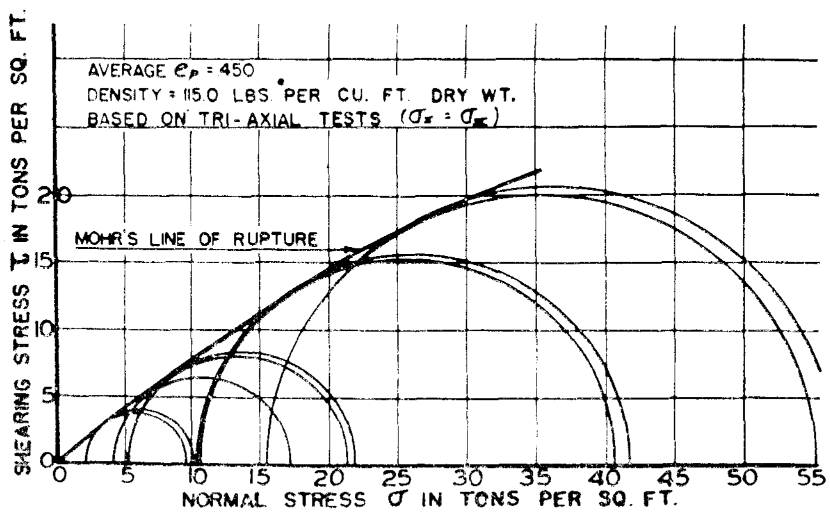


FIG. 10 - TESTS ON DENSE PIT-RUN GRAVEL

TESTS BY B.C. WILKAS & L.E. GREGG

PURDUE UNIVERSITY

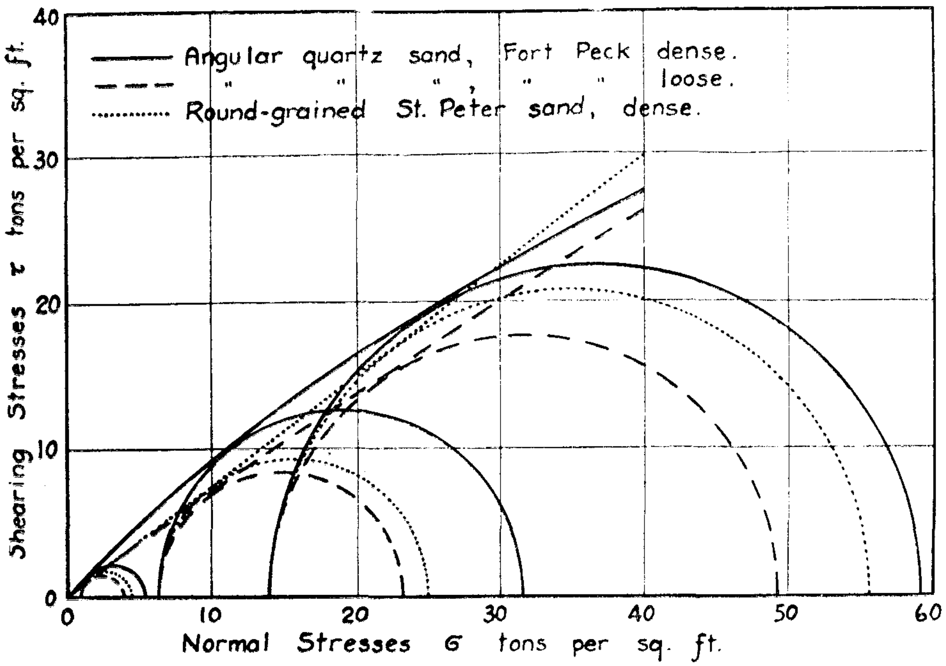


FIG. 11 STRESS CIRCLES FOR DIFFERENT SANDS.
TESTS BY J.D. WATSON (14)

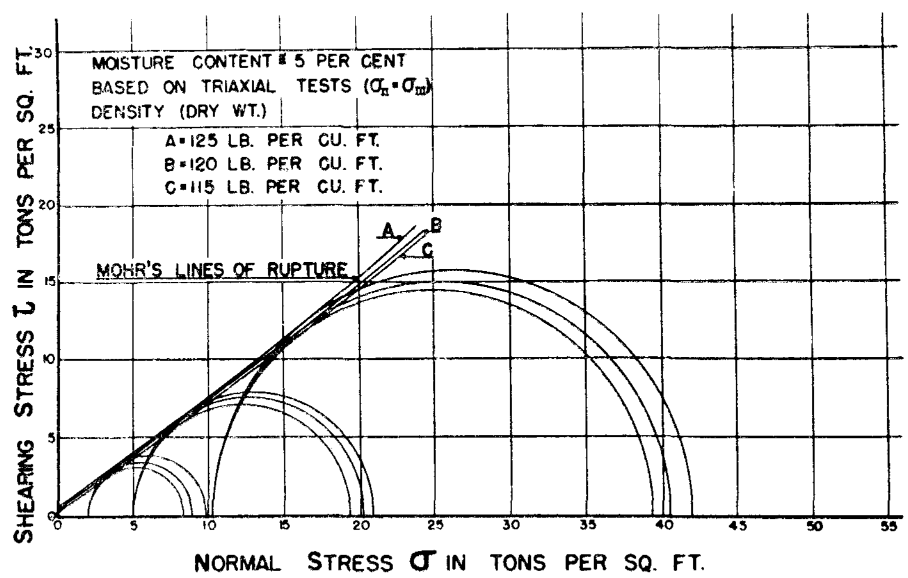


FIG. 12 - EFFECT OF DENSITY ON STRENGTH OF DAMP COMPACTED CLAY
TESTS BY L.E. GREGG, PURDUE UNIVERSITY

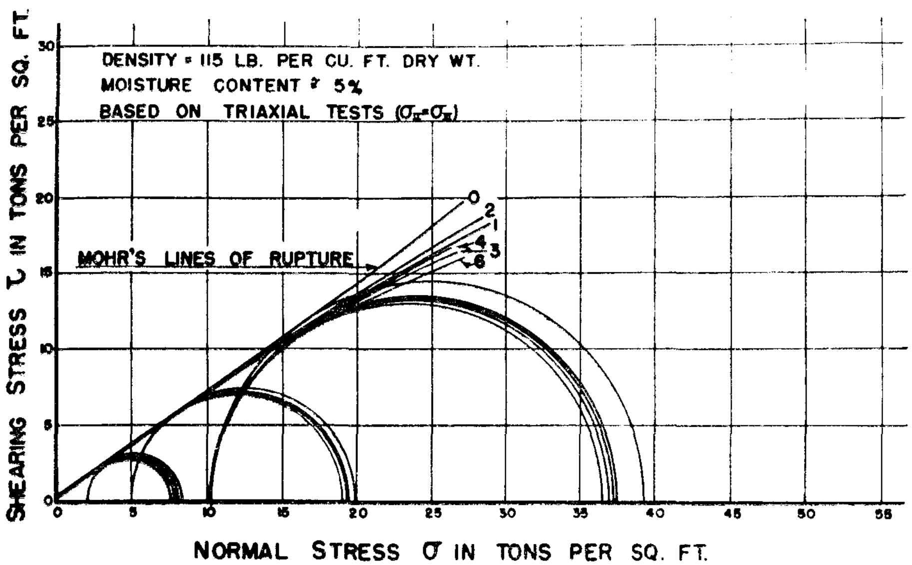


FIG. 13 - EFFECT OF BITUMINOUS ADMIXTURE ON STRENGTH OF DAMP COMPACTED CLAY
TESTS BY L.E. GREGG, PURDUE UNIVERSITY

TESTS ON CLAY FOR ROLLED-FILL EARTH DAM

SAMPLE NO.	13	5	10	11	3	8	7	6	2	14
DRY UNIT WT.	104.2	105.0	104.6	105.2	107.0	104.8	106.2	105.9	105.1	109.5
WATER CONTENT %	22.1	21.8	21.9	22.0	21.9	22.2	21.4	21.7	18.8	18.3
DEGREE OF SATURATION	97.6	97.0	96.7	98.5	100	98.5	98.5	98.9	84.5	92.0
TYPE OF FAILURE										

MOHR'S RUPTURE CIRCLES
FOR
QUICK-CONSOLIDATED TRIAXIAL TESTS
SATURATED-TYPE A-BORROW MATERIAL
—— COMPRESSION WITH LATERAL PRESSURE
--- COMPRESSION WITHOUT LATERAL PRESSURE
ALL SAMPLES INITIALLY 3"O x 7" HIGH
* --- VICKSBURG TESTS - UNSATURATED
(SEE PLATE 37 - U.S.W.E.S. REPORT)

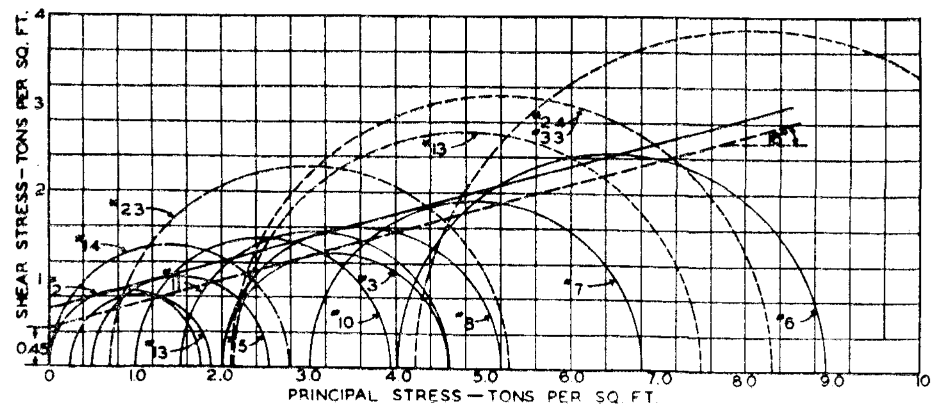
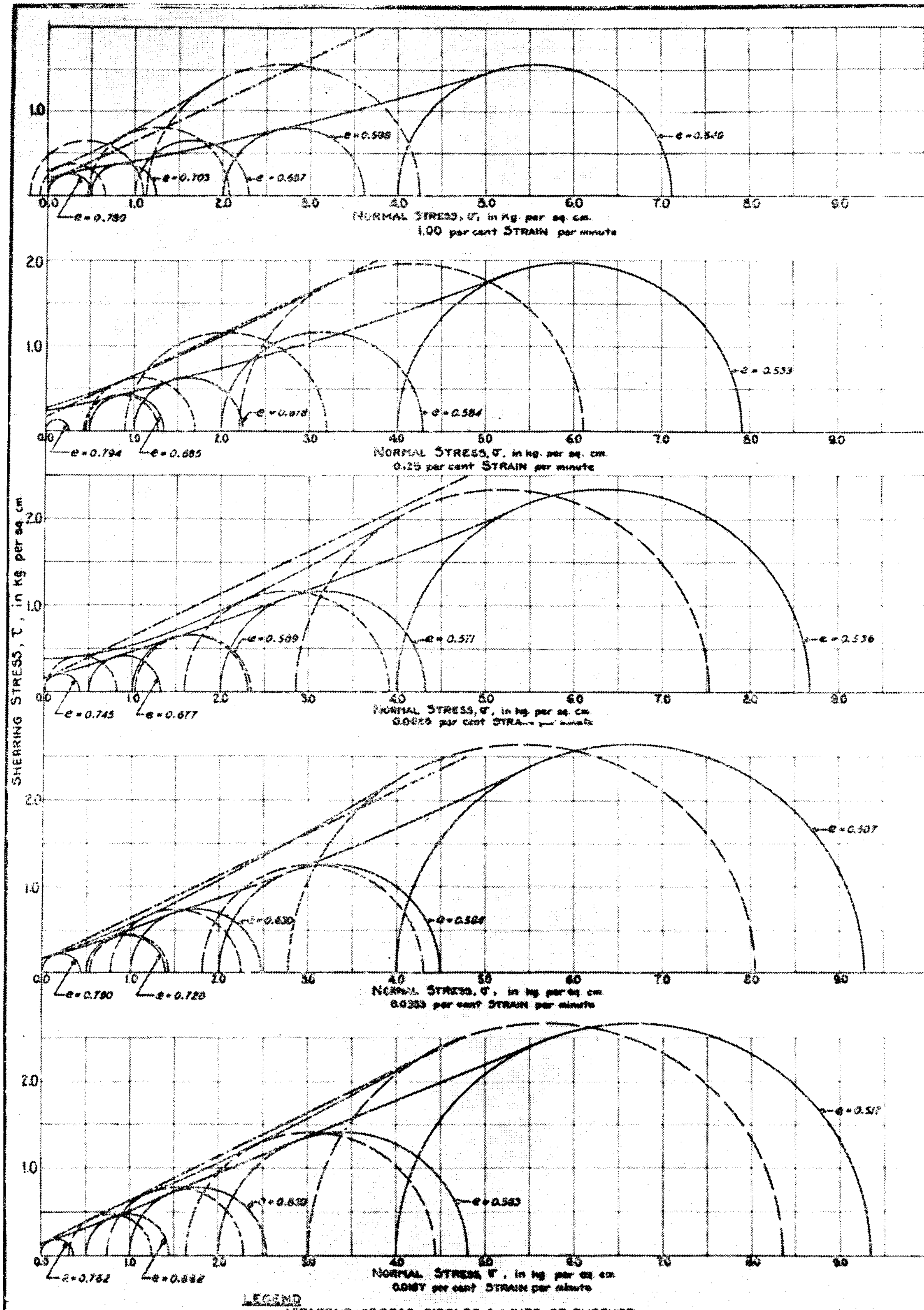


FIG. 14 - EFFECT OF SATURATION ON STRENGTH OF COMPACTED CLAY



LEGEND
— APPARENT STRESS CIRCLES & LINES OF RUPTURE.
— — STRESS CIRCLES & LINES OF RUPTURE CORRECTED FOR PORE WATER PRESSURE.
—·— AVERAGE CORRECTED LINE OF RUPTURE FOR ALL FIVE SERIES OF TESTS.
NOTE: ALL TESTS ON SPECIMENS CUT FROM ONE 12" CUBICAL SAMPLE WITH AVERAGE INITIAL VOID RATIO = 0.759. VOID RATIOS SHOWN ARE FOR MAXIMUM STRESS.

FIG. 15 - FAILURE DIAGRAMS - TESTS ON UNDISTURBED CHICAGO SOFT CLAY

LIMITATION OF THE FIELD LOADING TEST

by

Raymond F. Dawson
Associate Professor of Civil Engineering,
Testing Engineer and Assistant Director of the
Bureau of Engineering Research
The University of Texas
Austin, Texas

Engineers are continually asking "What is the bearing power of this soil?" and handbooks and building codes are full of recommended bearing capacities for different types of soils. Yet if you ask these same engineers how much load a 12", 50 lb., I-beam would carry, and give nothing as to the length of span, supports, or condition of loading, they would think it absurd. It is even more absurd to ask for the bearing power of a soil for in the case of the I-beam, you at least have a uniform material.

Soils are seldom uniform in either character of material, structure, or moisture content. If the I-beam was made up of a heterogeneous mass of steel, wood, concrete, and stone, you would certainly hesitate to apply your standard design formulae, yet such a conglomeration is nothing worse than is often encountered in soils. Still engineers who spend weeks and months designing a superstructure of steel or reinforced concrete will ask for a simple test to determine the bearing capacity of the soil and want the results in less than three days.

This desire for a simple test led to the establishing of the field loading test which consists of loading a relative small plate on the site of the foundation. Reams have been written about methods of making these tests, ingenious devices have been contrived to load and measure the settlements, but almost nothing has been said about analyzing the data and utilizing the results. Unless you gain some useful information from the field loading test, about the only excuse for conducting it is to impress an uninformed client with your "scientific" study of his problem and to give him the feeling that he is getting something in return for your fee. Personally, I believe that in proportion to the information obtained, there has been more money wasted on the field loading test than on any other test devised by engineers.

Field loading tests may be of value in a few limited cases and even then certain precautions must be taken in analyzing the data. In the case of cohesionless material like sand and gravel, the settlement is almost independent of the loaded area, and we may expect fairly reliable results from the field loading test if we are certain that the cohesionless material is not underlain by a bed of soft clay. However, settlement of a structure on sand and gravel is usually rather small and much of it takes place during construction so the field loading test may be of little actual value even thought the results are satisfactory.

If cohesive soils were elastic, homogeneous materials, it would be possible to work out a relation between the size of the loaded area and settlement using the theory of elasticity. But clays are not perfectly elastic or homogeneous, therefore, the settlements obtained by applying a ratio of this character will probably be very misleading. If soils were perfectly elastic and homogeneous, the ratio of settlement of two areas "A" and "a", loaded with equal unit loads would be $\sqrt{A/a}$. This value will be used in cases cited later to show its unreliability.

To an engineer who is thoroughly familiar with uniform soils in his locality, the field loading test can give considerable help and comfort by comparing the results with those obtained on the same soils under other buildings he has constructed. By keeping careful records and case histories of his foundations, an engineer can construct his own bearing tables that can be relied upon for similar structures in the same locality.

Settlement of a structure is caused by the consolidation or elastic deformation of the soil under it or by the soil flowing laterally or by a combination of these conditions. Consolidation results from the grains of soil being forced closer together which in turn reduces the voids and the total volume of the soil. Since the voids are usually almost completely filled with water, the grains cannot be forced closer together until the water is forced out as water is practically uncompressible. Therefore the rate of settlement depends upon how fast the water can be squeezed out which in turn depends on the size of the voids. Thus a sand will consolidate rapidly while a clay consolidates very slowly. This rate of consolidation affects the time necessary for complete consolidation in the field loading test. Laboratory tests on clay samples one inch thick usually require three to four hours to complete the primary consolidation. Pressures under a loaded area is effective to a depth equal to approximately twice its width and from the theory of consolidation, we know that the rates of compression of two layers of the same material under the same pressure conditions vary directly with the square of the thicknesses of the layers. From this we calculate that the field loading tests on a clay soil using a plate one foot square should take from two to three months to complete the primary consolidation under each load while a two foot square plate would take from nine to twelve months. Since loads are rarely ever left on the platform over a few days, they do not measure all of the consolidation of the soil. Because of the lack of refinement in the measuring device, settlement is not observed and the load is increased.

As stated before, the pressure beneath a footing is effective to a depth of $1\frac{1}{2}$ to 2 times the width of the structure and this pressure cannot be ignored until it drops below a value of approximately 0.2 tons per sq.ft. There are many examples where field loading tests show practically no settlement because of a firm surface stratum, but soft strata below have caused excessive settlement of the structures. Dr. Terzaghi(1) reports the following example: "To a depth of 25 feet beneath the base of the foundation, the soil consists of dense sand and gravel which, in turn, rests on a stratum of soft clay, 50 feet thick. At the base of the footings the soil pressure ranges between three and four tons per sq.ft. In a loading test which was performed on an area of one square foot, a soil pressure of four tons produced a settlement of not more than a fraction of an inch. Nevertheless during the forty years of its existence, the structure underwent unequal settlement ranging between 12 and 32 inches." Case D in the examples is also an excellent illustration of this type of settlement.

If the stresses induced in the soil mass excedes the shearing strength of the soil, it will fail by flowing laterally and we say that we have passed the ultimate bearing power of the soil. The possibility of shear failure depends on the depth and width of the foundations. The deeper the footing or the greater its width, the less is the danger of shear failure. Since field loading tests are made on small plates and usually on the surface of the ground or at the bottom of a pit with no surcharge of soil

(1) Proceedings Am. Soc. C. E., October, 1933, p. 1368.

next to the compression post, there is a greater possibility that the field loading test will exceed the ultimate bearing power of the soil than will the structure with its wider and deeper foundation. In a failure of this type, you may be able to see the bulging of the soil around the plate especially in the case of large settlements. Because of the rather frequent failures caused by lateral flow of the soil, the American Society of Civil Engineers, Special Committee on Bearing Value of Soils for Foundations, in a recommended procedure for making field loading tests published in 1920, suggested that burlap be used beneath the compression plate and tile to resist the natural tendency for the material to squeeze out from under the compression plate. Here we have a condition where the field loading test may settle excessively while the structure will have relative small settlement. Case E is an example of this character.

Summing up the relationship between the field loading test and settlement of the structure, we find that:

(1. The effect of size of loaded area varies all the way from 0 to the ratio $\sqrt{A/a}$ depending on the character of the soil.

(2. The ordinary field loading test does not measure the total amount of settlement likely to occur from the consolidation of the soil. To complete the primary consolidation under each load would usually require at least two months on a clay soil.

(3. The field loading test on a relative stiff upper stratum will not indicate the presence of a soft lower stratum that may cause detrimental settlements of the structure. One must have complete information relative to the character and condition of the soil for a depth equal to twice the width of the structure and if it is not uniform for the entire depth, there

can be no relation between results of the loading tests and the pressures developed under the foundation.

(4. Because of the size of plate and condition of loading, there is a much greater possibility of shear failure in the field loading test than under the foundation.

Thus we see that in materials where we have no constant relation between the size of loaded areas and settlements, we are measuring small amounts of consolidation and possibly large settlements due to shear failures in the case of the field loading tests and expecting them to balance with large consolidation settlements and no lateral flowing of the soil under the foundation. In 1891, Collingwood(2) stated: ". . . . and it is insisted upon by experimenters that absolutely certain results can only be reached when the area tested is approximately the same as that to be built upon." Completely disregarding these differences and this warning, engineers still insist on loading a one sq.ft. plate to determine the settlement of a 10,000 sq.ft. foundation.

<u>Examples showing the results of field loading tests compared to actual settlements.</u>

CASE A:

This is a record of the results in a sandy soil where settlement is almost independent of the loaded area. However, due to layers of clay and sandy clay, the structure has settled more than would occur had the foundation been on dense sand. "In 1922, the A.S.C.E.'s Special Committee on Bearing Value of Soils and Foundations, etc., published the results of loading tests performed on a building lot in San Francisco, California. A

(2) Engineering News, Vols. 25-26, February 14, 1891.

stratum of soft, clean, or sticky sand, with occasional layers of sandy clay and yellow clay was encountered. It was tested at six different points. Under a load of 4,800 pounds per sq.ft., the settlement of the bearing plate ranged between 0.04 and 0.17 inches, averaging 0.10 inches. For the past few years the lot has been occupied by a 22 story building, placed on a raft foundation, 218 by 252 feet, exerting a pressure of 4,800 pounds per sq.ft. (dead load only) on the soil. According to Equation (2), the settlement of the building should amount to 152 x 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 inches. During the first year, the settlement was quite rapid, but the rate has been less each succeeding year."(3)

CASE B:

This is the record of a structure in the coastal area of Texas. The soil consists of a layer of gray clay about 20 feet thick which contains a one foot stratum of sand. Under the gray clay is a tough red clay extending down more than 100 feet below the surface. Two field loading tests were made on this site, No. 1 using a plate with an area of 2 sq.ft. and No. 2 with an area of 4 sq.ft. The results of these tests are shown in Fig. No. 1. According to the theory of elasticity, Plate No. 2, (the larger one) should have settled 1.4 times as much as Plate No. 1. Actually Plate No. 1 settled more than Plate No. 2. This may have been caused by a very small amount of lateral flow under the smaller plate.

The building rests on a mat approximately 125 feet square and the total dead load on the soil is 5,100 pounds per sq.ft. Under this load,

(3) "The Science of Foundations," Dr. Karl Terzaghi, Trans. Am. Soc. C.E. Vol. 93, 1929, p. 275.

Plate No. 1 (2 sq.ft. area) settled 0.107 inches and Plate No. 2 (4 sq.ft area) settled only 0.062 inches. From the elastic theory, we would expect the structure to settle 9.5 inches using the results from Plate No. 1 and 3.9 inches from Plate No. 2. After about six years, the average settlement was about 3 inches and since then the settlement has been very small. Last year Dr. Terzaghi examined the settlement record of this structure and expressed the opinion that the settlement was essentially the result of elastic deformation rather than consolidation. The results obtained from Plate No. 2 confirm this opinion as they compare favorably with what has actually taken place. The smaller plate, however, indicated a settlement far in excess of what has actually occurred, and with a settlement of only 0.107 inches, we are in no position to say that there was a shear failure.

CASE C:

This is an isolated structure in the Gulf coastal place resting on a thick bed of joint clay. Borings show about 90 feet of tough red and yellow clay, underlain by 30 feet of red clay with sand streaks. Under this is 20 feet of sand followed by more clay with streaks of sand. The foundation of the building is a concrete mat approximately 125 feet square. The unit dead load on the soil is 4,700 lbs. per sq.ft.

Five field loading tests were made on the site and consolidation tests were made on undisturbed samples before construction. For a pressure of 4,700 lbs. per sq.ft. the field loading tests gave settlements ranging from 0.08 inches to 0.22 inches. The test giving the highest value failed under a load of 6,500 lbs. per sq.ft. in what was evidently a shear failure, therefore this test was not considered in the study. The other four tests gave an average settlement of 0.10 inches for a load of 4,700 lbs. per sq.ft.

Figs. 2a gives the results of a typical field loading test, and the predicted settlement of the plate from the consolidation test data. Curve (1) is the predicted total settlement of the plate if the soil were permitted to completely consolidate under each load while Curve (2) is the prediction corrected for the time the plate was actually loaded. Curve (2) is only about 10% higher than the average settlement of the plates for a load of 4,700 lbs. per sq.ft. These curves verify conclusion (2. which stated the ordinary field loading test does not measure all of the settlement likely to occur from the consolidation of the soil.

Using the ratio $\sqrt{A/a}$ and the average of the field loading test settlements, we would expect the structure to settle 9 inches. However, if the soil were permitted to completely consolidate under the field loading test, the calculated settlement would be 17 inches. From the laboratory consolidation test we predict a total settlement of 5.7 inches and to date the actual settlement has been slightly over 5 inches. Fig. 2b gives the results of the predicted settlement from the consolidation test compared with the actual settlement of the structure.

From Cases "B" and "C" we learn that we cannot depend upon the theory of elasticity to determine the ratio of settlement of two areas loaded with equal unit loads unless we multiply the ratio by some factor K, which may vary all the way from 0.33 to 0.83 for the same soil. Some engineers have suggested that a series of field loading tests be made on various size plates in order that K may be determined for the soil under consideration. There is the possibility that such a factor may be influenced by a slight amount of lateral flow under the smaller plates which would make such a factor utterly useless. In Case "B" any attempt to determine such a factor from the two field loading tests would result in predicting a settlement of practically 0 for the structure. Also such a factor will not show the influence of a deep soft stratum as illustrated in Case "D".

CASE D:

Case "D" is an oil tank in the Gulf coastal region which illustrates the effect of a soft stratum of soil below the surface. Fig. 3a gives the soil conditions under the foundation. On this chart has been plotted the pressure profiles for the center of the field loading plate and also the center of the foundation slab. These pressure profiles were computed by the Boussinesq equation which assumes a perfectly elastic, isotropic material. Since we have just pointed out that soils are not perfectly elastic or isotropic, we cannot therefore expect the charts to give an accurate picture of how the stresses vary in the soil. However, since in Fig. 3a we are comparing stress distributions in the same materials, they should give relative values. These pressure profiles are for the center of the loaded areas and are therefore maximum conditions. The average stresses in the soil should be somewhat less than those indicated by the curves.

From Fig. 3a we see that by the time we reach the soft blue muck, the pressure under the center of the field loading plate is about 0.1 ton per sq.ft. while under the foundation slab, it is above 0.9 tons per sq.ft. Due to this condition, we would expect the soft blue muck to have little effect on the settlement of the field loading plate while it would contribute the major portion of the settlement of the foundation slab.

Fig. 3d is the result of the field loading test on a 2' x 2' plate. The load on the soil from the tank and foundation amounts to 820 pounds per sq.ft. For this pressure the field loading test gives a settlement of 0.1 inch. Using this value and the ratio from the theory of elasticity, we would expect the slab to settle 0.9 inches. From Fig. 3c we find that the foundation has actually settled from 2 to 4 inches, and continues to subside at a uniform rate. The field loading test is certainly misleading under these conditions.

CASE E:

This is a situation where the field loading test exceeded the ultimate bearing capacity of the soil resulting in a large settlement of the plate. The structure is located in North Texas and the soil consists of a medium clay which extends well below twice the width of the footings. The foundation consists of a series of spot footings located so that there is no overlapping of stress except that from one adjoining footing as may be seen in Fig. No. 4b. At the site of the field loading test the footing is 12 feet below the surface. A rain shortly before the excavation was completed, softened the surface soil and made it appear so weak that a field loading test was ordered. This test was made on a 1 sq.ft. plate at the bottom of excavation as shown in Fig. 4d. Contrary to the usual procedure for field loading tests, the total load of 9,000 lbs. per sq.ft. was placed on the platform in as short a time as possible. The settlement of the plate is shown in Fig. 4c. Because of the large amount of settlement (3-7/16") and the observed bulge (Fig. 4d), it is evident that the ultimate bearing capacity of the soil was exceded.

Having determined the shearing strength (Fig. 4a) from undisturbed samples of soil taken only a few feet from the site of the loading test, these data were used in computing the critical loads for shallow footings using curves derived from the log-spiral method. These results are for a general failure of a strip load which we do not have in the case of the square bearing plate. However the general results for a strip loading 1 foot wide check very closely to the results obtained from a circular plate with one sq.ft. area, so the general case was used for the comparisons. Even though the calculations are in error by a considerable amount, it would not alter the conclusions.

The following table shows how increasing the width of the loading plate increases the ultimate bearing capacity of this soil when the plate is placed on the surface of the ground.

Width of Loading Strip in Feet	Bearing Capacity of Soil, Lbs./sq.ft.
1	3,750
2	3,950
4	4,350
8	5,140
10	5,530

Applying the conditions of the field loading test, we find the bearing capacity of the soil to be 3,750 lbs. per sq.ft. Since this value is below the unit load used in the test, we should expect a shear failure. In fact, the 10 foot wide footing would have failed under the test load had it been placed on the surface.

If the compression plate is below the surface of the ground, the weight of the soil above the loading plane will increase the ultimate bearing power. In the case under consideration, this surcharge has a greater influence on the bearing capacity than the width of the plate. The effect of the depth of surcharge surrounding the compression plate is as follows for a 1 sq.ft. area:

Depth of Surcharge in Feet	Bearing Capacity of Soil, Lbs./sq.ft.
0	3,750
1	4,410
2	5,070
4	6,390
8	9,030
10	10,350

Now it is interesting to observe Fig. 4b and d. The compression post was placed near the north wall of the pit and it will be noted that the flow of soil was on the opposite side of the plate from this wall, indicating that the weight of the surcharge on the north side may have assisted in preventing a shear failure even though it was approximately 2 feet from the post.

With a footing 10 feet wide and 12 feet below the surface, the calculated ultimate bearing capacity is 13,450 pounds per sq.ft. As the total dead and live load on the soil amounts to only 4,600 pounds per sq.ft., we have a factor of safety of 3 against a shear failure, thus making it inconsistant to attach any serious importance to the shear failure under the loading test.

From consolidation tests made on undisturbed samples, the footings were designed to keep the average settlement below 1 inch. Due to special conditions, however, it was anticipated that this footing would settle 2.5 inches. About on month after the start of construction when the total dead load had reached 2,500 lbs. per sq.ft., the footing had settled only 0.10 inch.

Having reached the conclusion that the field loading test may be very misleading, you may ask how we can estimate the settlement of a structure. First we must have complete knowledge of the soil conditions for a depth at least equal to $1\frac{1}{2}$ to 2 times the width of the structure. This is essential even though you do not expect to make any tests because a soft stratum at considerable depth below the footing may cause detrimental settlement. In the case of deep beds of uniform cohesionless materials (sand and gravel), a field loading test will give reasonable indications of the settlement of the structure since in these materials the rate of settlement is almost independent of the loaded area.

If the soil is a cohesive material, it is necessary that undisturbed samples be taken of each stratum for laboratory tests and analyses. From the consolidation tests, we can calculate the amount and rate of settlement likely to occur from the compression of the soil. It is essential that consolidation tests be made on each different type of soil and that enough tests be made to get a fair average of each type. The reliability of these predictions will depend on the uniformity of the soil, the care used in selecting the samples so that the test specimens are representative of the existing soil, and whether or not enough specimens are tested to obtain a reliable average. Errors of 10 to 20 per cent are considered exceptionally good estimates and we cannot expect to check as closely as we did in Case "C" even though the soil is uniform in character over the entire area. The difficulty of applying and interpreting laboratory tests becomes very great when the site is underlain by irregular lenses of varying and diverse soils, particularly when these lenses are largely composed of mixtures of sand and clay. However if the estimates were off 100%, the predictions would still be much closer than you would probably obtain from a field loading test.

If there is danger of lateral flowing of the soil, its ultimate bearing capacity should be calculated and the unit load kept below this value. In order to calculate the ultimate bearing capacity, it is necessary to know the shearing strength of the soil. Shearing strengths on undisturbed samples may be determined by the direct shear test, unconfined compression test, and tri-axial compression test. Although investigators agree that the best results are obtained from the tri-axial compression test, they are not in accord as to which is the more practical test to use. On cohesive materials it takes a long time to complete the tri-axial compression test; therefore, it is not generally used except on large structures such as the Denison Dam or in investigational work. It is believed that the average of a number of tests which require a minimum of time to make, will give a clearer picture of the soil conditions than will a few of the more accurate tests. Dr. Terzaghi prefers a large number of the unconfined compression tests and he does not consider 25 or 30 tests a large number. In general, only shallow foundations with relative small bearing areas are liable to shear failure except in the case of extremely soft soils. For large bearing areas, particularly at some depth and in plastic soils, the safe loading will be limited by permissible settlement rather than by soil strength.

It requires at least two weeks to make a consolidation test and although a number of tests may be run simultaneously, several weeks are necessary to complete a series of tests and the calculations. The amount of time involved in determining the shearing strength of the soil depends upon the test used, but in any case, it will require an additional several weeks time. Since borings and undisturbed sampling are also slow procedures, it is essential that you start the foundation investigations at the earliest date possible - even before you start your architectural and structural designs, and preferably before the site is purchased. Often a foundation investigation will disclose that certain sites are not desirable or economical for the proposed type of structure, and a substantial saving may be made if this is known before the building site is purchased.

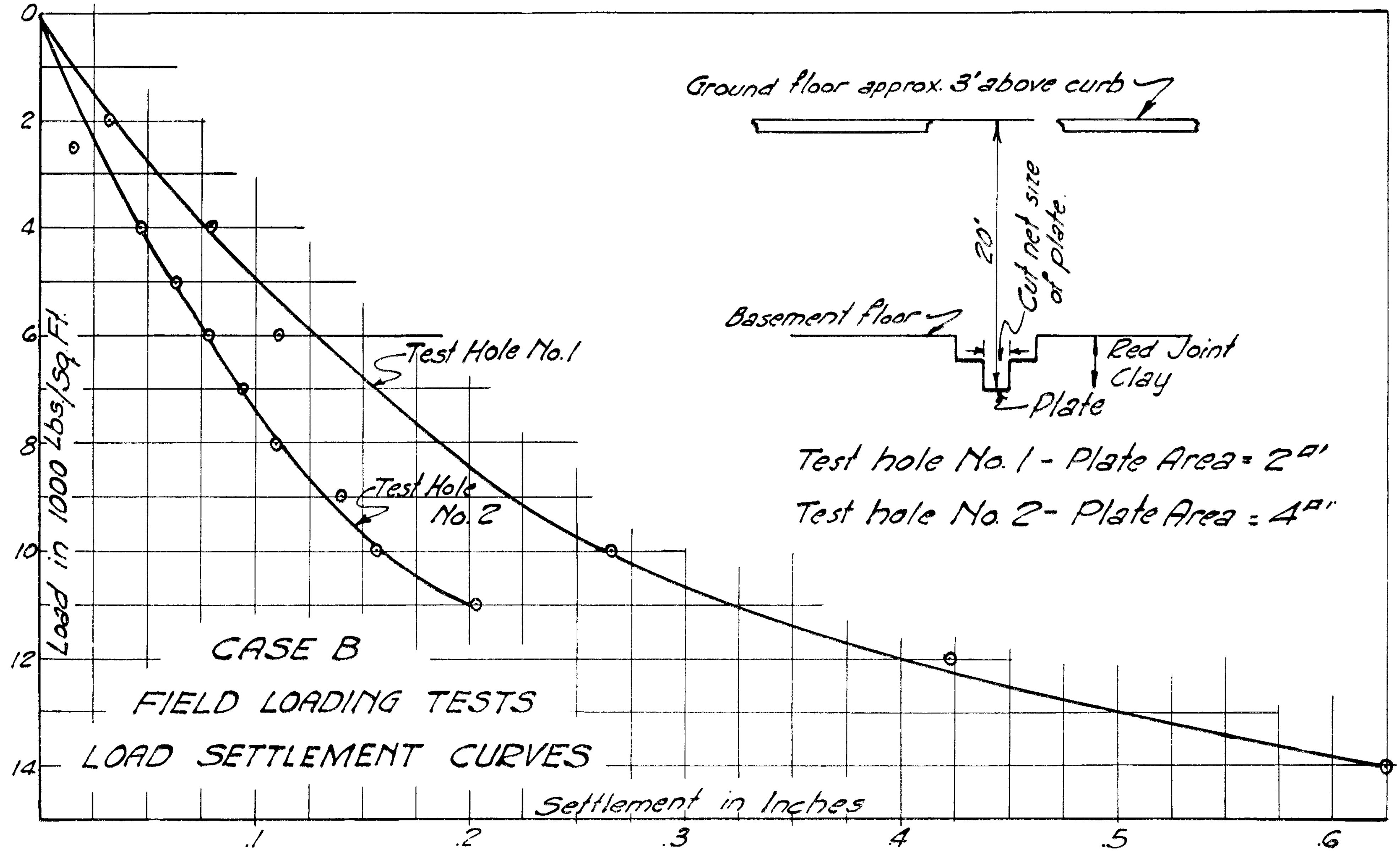
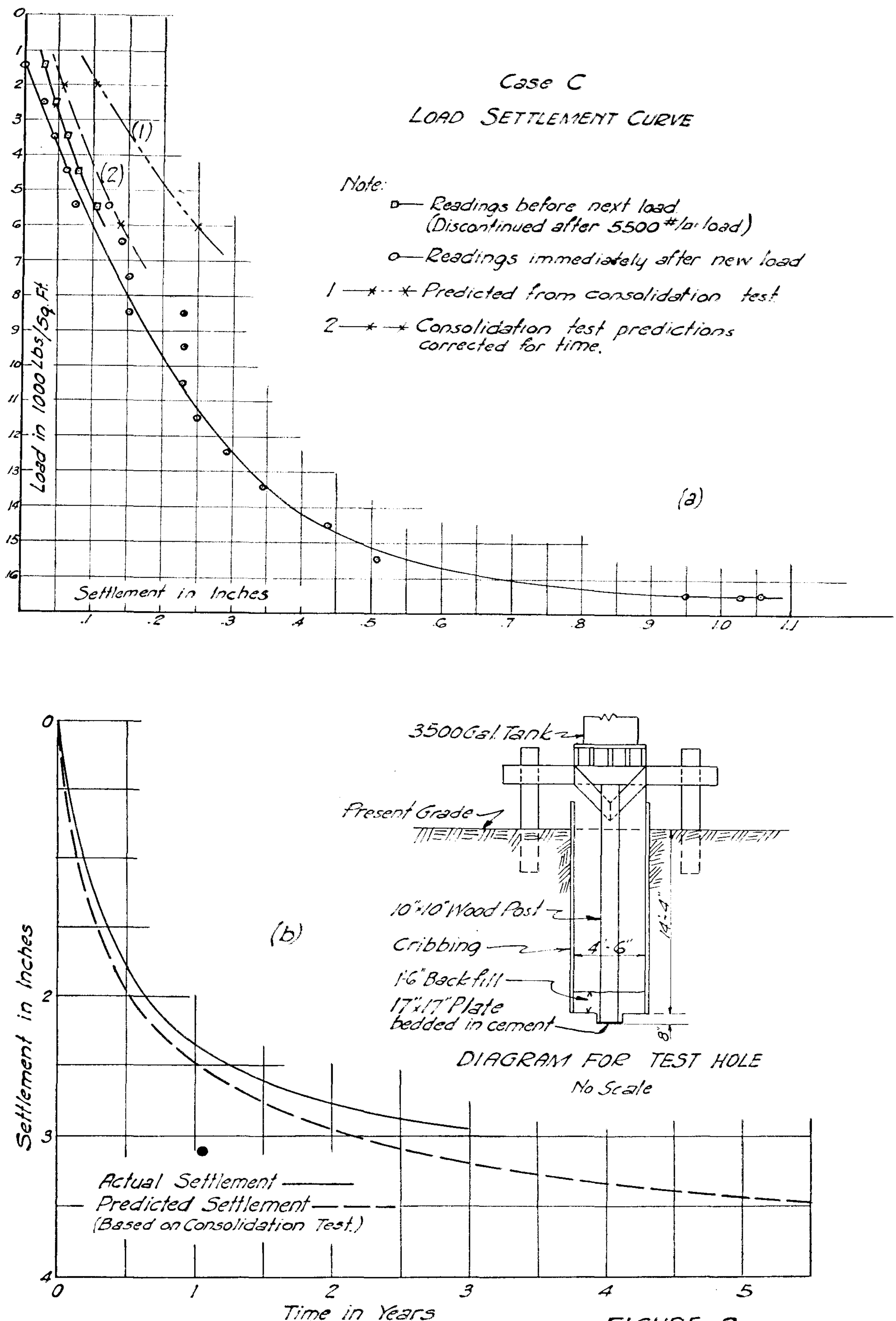

Ground floor approx. 3' above curb
20'
Cut net size of plate.
Basement floor
Red Joint Clay
Plate
Test hole No. 1 - Plate Area = 2□'
Test hole No. 2 - Plate Area = 4□'
Test Hole No. 1
Test Hole No. 2
CASE B
FIELD LOADING TESTS
LOAD SETTLEMENT CURVES
Load in 1000 Lbs/Sq. Ft.
Settlement in Inches
0
2
4
6
8
10
12
14
.1
.2
.3
.4
.5
.6

FIGURE NO. 1



Case C
LOAD SETTLEMENT CURVE
Note:
Readings before next load (Discontinued after 5500 #/□' load)
Readings immediately after new load
1 Predicted from consolidation test
2 Consolidation test predictions corrected for time.
(1)
(2)
(a)
Load in 1000 Lbs/Sq. Ft.
Settlement in Inches
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
.1 .2 .3 .4 .5 .6 .7 .8 .9 1.0 1.1
3500 Gal. Tank
Present Grade
10"x10" Wood Post
Cribbing
4'-6"
14'-4"
1'-6" Backfill
17"x17" Plate bedded in cement
8"
DIAGRAM FOR TEST HOLE
No Scale
(b)
Settlement in Inches
0 1 2 3 4
Actual Settlement
Predicted Settlement (Based on Consolidation Test.)
0 1 2 3 4 5
Time in Years

FIGURE 2

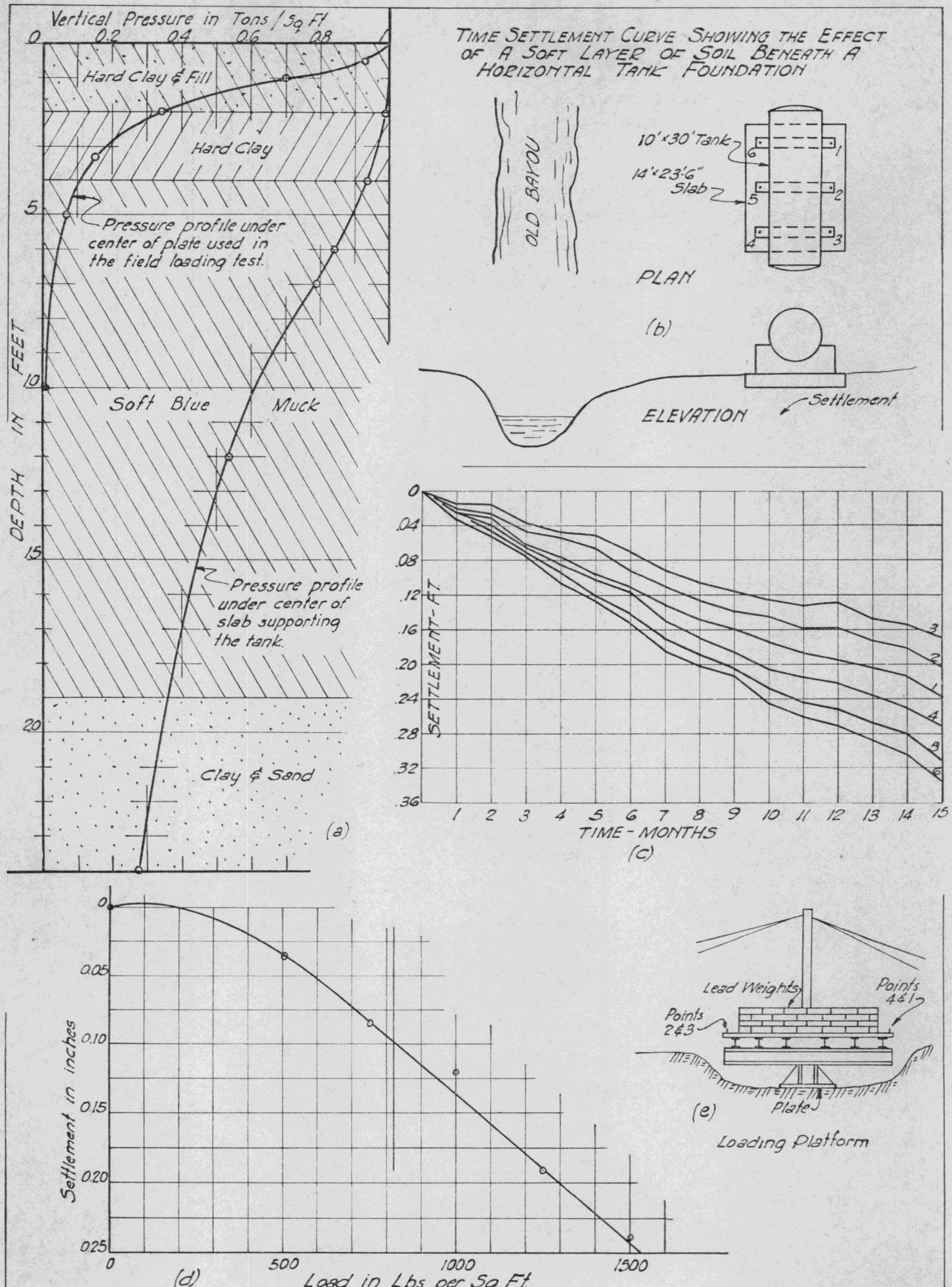


FIGURE 3

CASE D

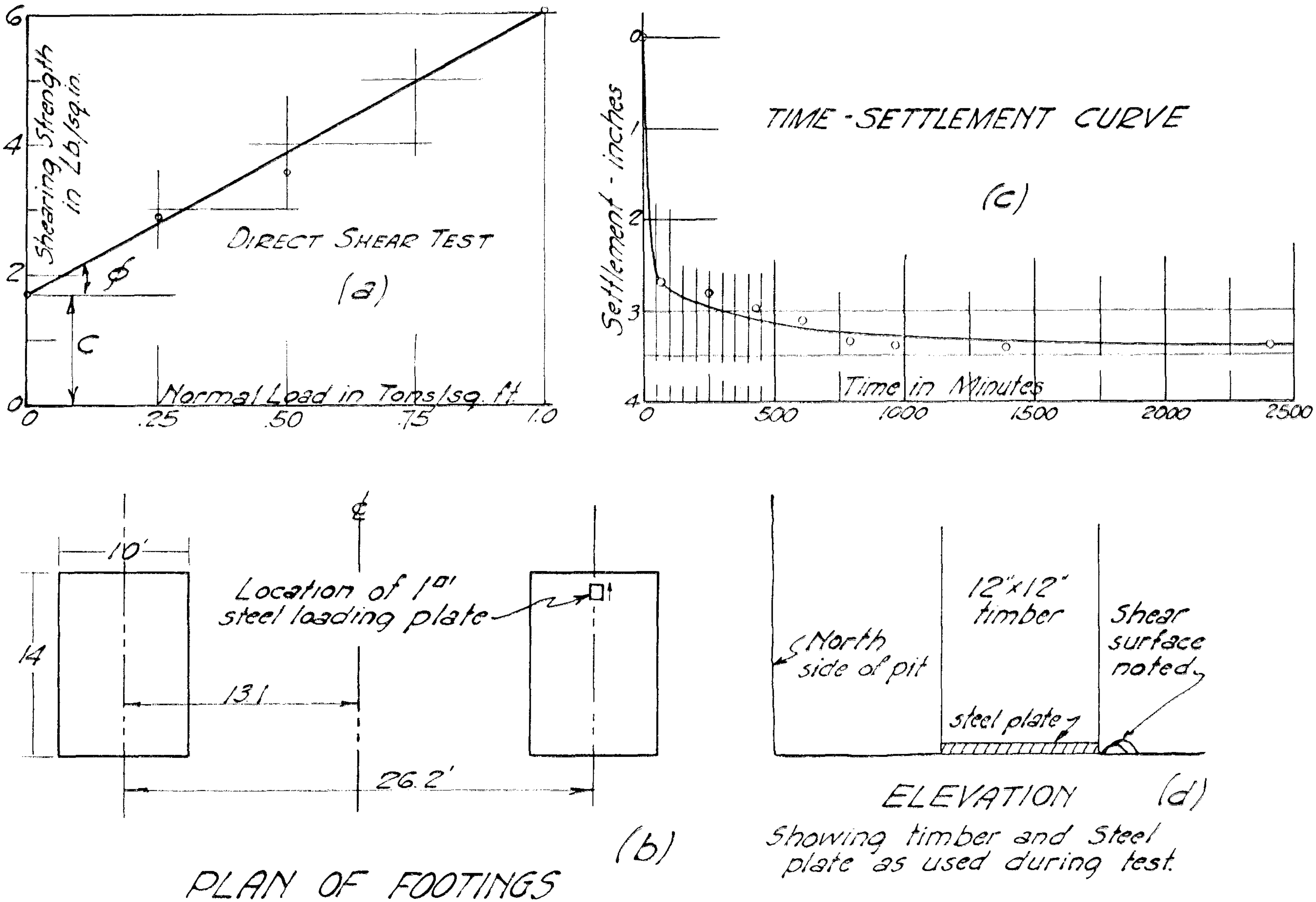

Shearing Strength in Lb./sq.in.
Normal Load in Tons/sq. ft.
DIRECT SHEAR TEST
(a)
C
φ
0
2
4
6
.25
.50
.75
1.0
TIME-SETTLEMENT CURVE
(c)
Settlement - inches
Time in Minutes
1
2
3
4
500
1000
1500
2000
2500
10'
14
13.1
26.2'
℄
Location of 1□' steel loading plate
(b)
PLAN OF FOOTINGS
North side of pit
12"×12" timber
Shear surface noted
steel plate
ELEVATION
(d)
Showing timber and Steel plate as used during test.

FIG. No. 4