Research in foundations and soil mechanics

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Research in Foundations and Soil Mechanics.

Untersuchungen über Gründungen und Bodenmechanik.

L'étude des fondations et la mécanique du sol.

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A. Modern development of loading tests.

Ever since engineers and builders have taken more than a casual interest in the ability of the ground to support building loads, load tests have been the most obvious and direct method of measuring bearing capacity. In the past the failure to recognize the refinements in conducting load tests and interpreting results of the tests which are necessary to obtain reliable information has caused them to fall into some disrepute. Even today those who are unfamiliar with latest developments may criticize and reject load tests for no apparent reason other than the failures which have resulted from their improper use.

During the past 10 or 15 years a few investigators in Europe and America have made substantial progress in the use of load tests. Consequently there are today a number of well recognized principles which are available to guide present practice and which demonstrate the fallacy in the earlier attempts to evaluate bearing capacity. For instance, it is well recognized that bearing capacity varies with the size and shape of the bearing area. It should then be obvious that no adequate measure of the soil resistance can be obtained by loading a single size of bearing area of some incidental shape. Yet the mere admission of this unquestionable truth immediately invalidates the great majority of load tests that have been made in the past and a considerable number of those that are being made today in general building practice. There are comparatively few examples where a comprehensive series of load tests has been made in full recognition of this first principle of sound practice.

There are also a number of other conditions which must be recognized and properly controlled. The process of applying the load to the bearing area and measuring the consequent settlement must be carried out to a much higher degree of accuracy than is commonly obtained. The time element must be carefully controlled to obtain a true relation between load and settlement without dynamic effects. The surface to be loaded must be prepared without disturbing the underlying soil and without changing its properties by wetting or drying. If the tests are to be unconfined, that is without the effect of surrounding overburden, the test pit must be large enough on all sides to remove such confining influence. If the tests are to be confined special precautions must be used to eliminate any opportunity for the soil to squeeze up around the bearing plate. The failure to control any one of the conditions which affect the tests may destroy their value, so careful attention to these details is of primary importance.

However, there is nothing in any of these requirements that cannot be readily accomplished under practical conditions and after some experience they become a part of routine testing procedure. The data that may then be obtained are reliable and their analysis leads to practical conclusions that can be used with confidence.

The greatest advantage of loading tests lies in the fact that the soil is tested in place under the actual conditions to which it will be subjected by the load of the structure to be built upon it. Further than this the bearing capacity is measured directly as load per unit area. Such data do not require translation through the medium of formulae involving complex physical relationships which are incomplete in conception and of controversial nature. A series of loading tests is actually a measure of bearing capacity which integrates directly all of those obscure soil characteristics and properties which affect behavior under load.

It must be recognized that the loaded areas are much smaller than the actual footings which support the structure and, therefore, represent soil resistance at comparatively shallow depths, probably not greatly in excess of the diameter of the bearing areas. In consequence, additional tests may be required if there is a significant variation in the soil strata at greater depths and this fact is accepted in building practice. Nevertheless that portion of the underground which is stressed by the loaded area is a representative sample and one which is much larger than it would be possible to remove and test in the laboratory. It is somewhat difficult to understand the logic that is sometimes used by laboratory technicians who would reject the load test as not being large enough to constitute a representative test and who at the same time express perfect confidence in tests on much smaller samples taken to the laboratory after being entirely removed from the natural conditions of the soil in place.

The investigations in soil mechanics and foundations at the University of Michigan which have been in progress for the last nine years have formed the basis of the preceding general comments on loading tests. The greater portion of the work has been carried out in connection with building projects of some magnitude and the furnishing of information to designing engineers was a primary necessity. Laboratory investigations on pertinent phases of soil mechanics have paralleled the field testing and much valuable and instructive information has been obtained therefrom. But interesting as the laboratory study has been it is only a statement of fact to say that the practical criteria required by the designing engineers have been obtained from the field tests and only supplementary information from the laboratory. In the subsequent discussion it is intended to discuss as briefly as possible the testing procedure and methods of analyzing and interpreting loading tests as developed during the course of these investigations.

I. Test procedure.

Preliminary Investigation.

The first requirement of comprehensive soil testing under field conditions is a preliminary study of the problems presented by any proposed construction and a survey of the foundation conditions of the site. This investigation should determine the general soil conditions to be encountered, the presence of ground water, the necessity of the test pits being sheeted or braced, and the general suitability of the site for making bearing capacity tests. Such a preliminary study may be made by borings which undoubtedly will be later required or by digging a trial test pit which also may be utilized later for conducting a load test. Very often the information necessary for outlining test procedure may be obtained from knowledge of previous construction and general knowledge of soil conditions in the particular locality.

Samples of soil carefully taken may given an experienced engineer a fairly accurate idea of the character of the soil, but the result of visual inspection of

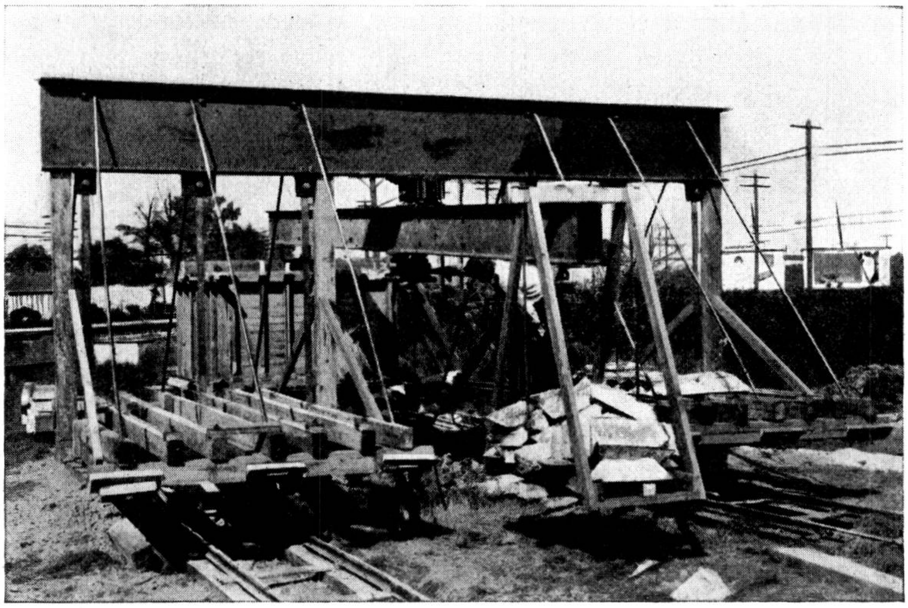

Fig. 1.
60-ton soil testing apparatus.

such samples can in no sense be considered as a measure of the physical properties of the material. While it is true that many structures are designed and important decisions made on information obtained from borings, this practice is to be discouraged if any real progress toward scientific treatment of foundations is to be realized. Information of the type obtained from borings should be considered as only preliminary to a comprehensive determination of soil properties.

A fairly complete knowledge of the proposed construction must be combined with the knowledge of general soil conditions in order to lay out an intelligent program of testing. From this aspect of the problem, an estimate of the probable range of loads, the type of structure, whether rigid or capable of sustaining some differential settlement without injury, the elevation of the proposed substructure, and the use to be made of the structure are all necessary parts of the preliminary survey. Frequently a consideration of the various angles of the problem narrows the choice of type of substructure or elevation of the substructure down to one or more possibilities which can only be properly evaluated by the more definite knowledge derived from actual load tests.

The depth at which the tests are required may be determined by the examination of samples from the various soil strata and, particularly so, when there are significant variations in the material at different depths. On the other hand, the elevation may be definitely determined by the requirements of the proposed structure, the elevation of the basement or sub-basement sometimes being a controlling feature rather than the bearing capacity of the soil at the resulting elevation of the substructure. After selection of the elevation at which the soil is to be tested, the regulation of test conditions must be given full consideration.

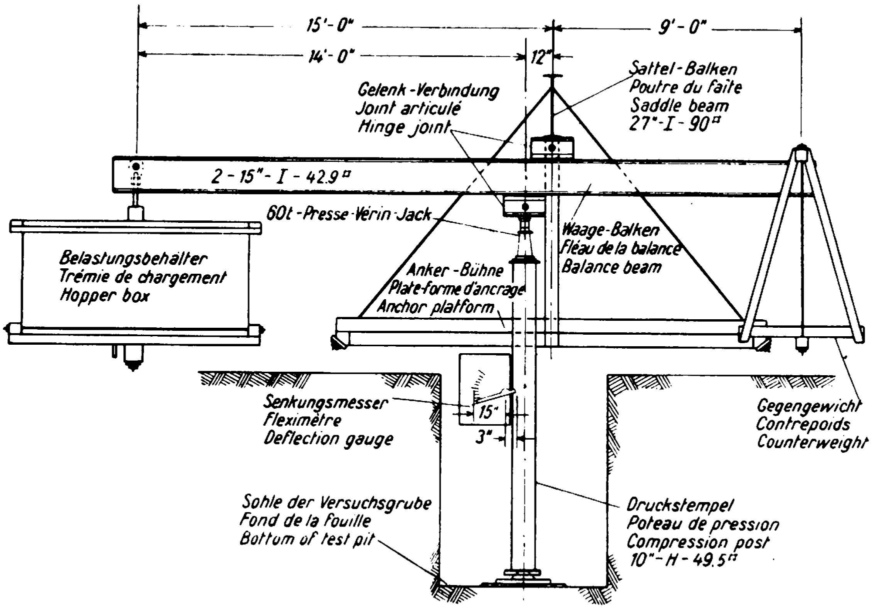

Fig. 2.

Line diagram of soil testing apparatus.

Testing Equipment.

Two types of loading equipment have been used in tests conducted under the writer's supervision and are illustrated in the following figures:

Fig. 1 is a photograph of a 60-ton testing apparatus of the balance-beam type used in most of the series of tests.

Fig. 2 is a line diagram of the assembled apparatus. In conducting a test, the anchor platforms are loaded with pig iron and the bearing plates and compression post are set up in the test pit as shown. In adding the load increments to the bearing plate the balance beam is first leveled by adjusting the counterweight and the jack is brought into contact. The load increments are supplied by siphoning a measured quantity of water into the hopper box and operating the jack to maintain the level of the balance beam. The entire apparatus is set upon industrial trucks and narrow gauge track for moving readily from one test pit to another.

The settlement is measured as indicated in Fig. 2 by a deflection arm which is attached to a drawing board on which the settlement chart is placed. As the compression post settles the movement of the lever arm is marked on the chart providing a continuous record of settlements throughout the progress of the test. This simple arrangement makes it possible to record rapidly the settlement at any instant and supplies an accuracy of settlement measurement to $^1/_{100}$ inch.

The apparatus shown has proved entirely satisfactory and fulfills all of the requirements for supplying adequate soil test data. The load increments may be applied accurately and quickly without jarring impact and the balance beam maintains a constant load on the bearing area as settlement is taking place. The apparatus may be moved from one test pit to another without reassembling, or loading and unloading the anchor platforms. The settlement is measured accurately at any stage of loading, making it possible to distinguish that settlement which takes place immediately upon application of the load from that which accumulates over a longer time interval.

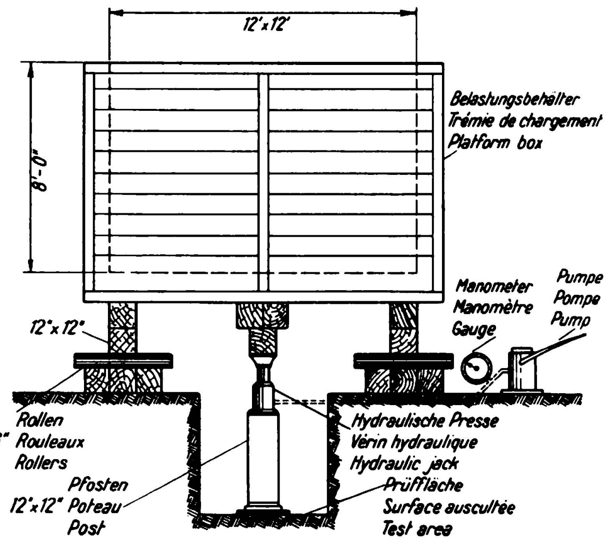

Fig. 3.
Soil loading test with hydraulic jack.

Fig. 3 indicates another method used in loading tests in which the load is applied by a hydraulic jack operating against a loaded bin or platform. The load increments are measured by the gauge pressure and the settlements are recorded by the same device shown in Fig. 2 and previously described. This direct loading of the bearing plates has been found fairly satisfactory but the accuracy with which the load increments are applied is not as great as by the use of a balance beam. The hydraulic gauges commercially available are not as sensitive as is sometimes desirable and there is some fluctuation in the applied load even though the pressure pump supplying the jack may be operated almost continuously. In spite of these disadvantages this method has given results within the limits of practical necessity and may be used when the more desirable balance-beam type of apparatus is not available.

Size and Shape of Test Areas.

The size and shape of bearing areas to be loaded is the first consideration in test procedure, and, in tests thus far conducted, range from 1 to 9 sq. ft. Tests have been made using several different shapes of plates with the result that round bearing plates have been selected for standard testing procedure. The effect of varying the shape of the plate is illustrated in Fig. 4 which shows the load-settlement diagrams for three test areas of 4 sq. ft. in round, square, and rectangular shape, the rectangle having a ratio of length to width of 7 to 1.

It has been observed in conducting load tests that for the lower range of load the concentrations of pressure around the perimeter of the loaded area ordinarily supply a major portion of the resistance. In the upper ranges of load the plates penetrate the soil showing distinct planes of shearing failure at the edges and bringing into play resisting pressure distributed more uniformly over the entire bearing area.

The relative importance of developed resistance at the boundary of the loaded area and that resistance which is independent of the boundary is conveniently expressed by the perimeter-area ratio. In the example given the perimeter-area ratio of the round, square, and rectangular area is 1.77, 2.00 and 3.04 respectively. Correspondingly, the load carried at any given settlement in the lower range is proportionately greater for the areas with the greater perimeter-area ratios. In the upper range of load the order of relative strength is reversed indicating that the plates with the larger perimeter-area ratio carry relatively smaller loads.

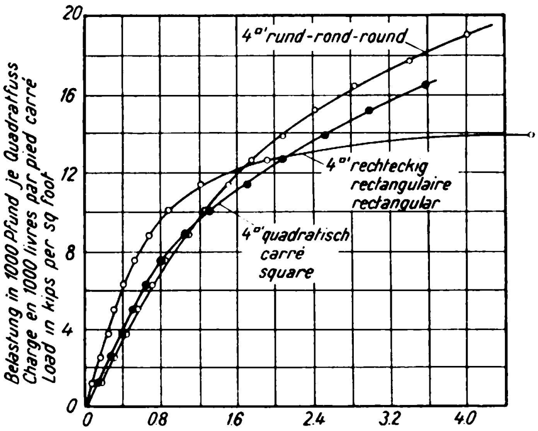

Fig. 4.

Bearing capacity for different shapes.

This reversal of supporting ability dependent on the shape of the plate is interpreted as a failure of the square and rectangular shapes to conform to the natural stress conformation of the body of soil. In a body of indefinite extent, subjected to surface loads, points of equal stress are to be found at equal distance from the concentrated load, that is, the stress pattern in horizontal sections is circular. A round bearing area develops equal resisting pressures at all points on its boundary while square or rectangular areas overstress soil at the corners before other points on the perimeter have reached their maximum resistance. It may be said that in the lower range of load the rectangular shapes tend to develop an area of influence which might be represented by a circle circumscribing the area. In the upper range of load the zone of influence tends toward an inscribed circle inasmuch as the corner concentrations fail to maintain their initial strength.

In attempting to evaluate bearing capacity in terms which properly consider the effect of the size of the bearing area it is important that the same shape of plate be used in any one test series. It is further considered advisable to use round bearing areas as they eliminate dimensional effects, such as corner losses, which are not so much a function of the soil properties as they are an irregularity introduced in test procedure.

Effect of Confinement.

The next test condition to be controlled is that of confining influence due to surrounding overburden. If the tests are intended to evaluate the bearing capacity due to soil cohesion alone, they should be made on a free surface with no superimposed pressure adjacent to the loaded area. To accomplish this, the test pit should be at least three times the diameter of the bearing area.

Confined tests which take full advantage of surrounding overburden should be made in test pits excavated to the exact dimensions of the bearing area. In softer soils it has been found necessary to drive a casing or pipe the same size as the bearing area and conduct the test at the bottom of the casing. In the case of stiffer soils the tendency of the soil to flow around and over the bearing plate is less and the full effect of static head may be obtained without confining the sides of the excavated test pit.

Investigations which include both confined and unconfined tests in order to measure the effect of overburden indicate that the pressure developed by a bearing area is increased an amount at least equal to the superimposed pressure. There is also additional capacity due to changed boundary reactions, but this effect is of a minor order of magnitude. In cases where the unconfined tests have been used as the basis of design for confined footings the general rule of increasing the developed pressure by an amount equal to the superimposed pressure has been considered sound practice and may be justified on a theoretical basis as well as by experimental evidence. Whenever it has been definitely determined that footings of the proposed structure will be fully confined it is, however, desirable to conduct the tests in the same manner as to correspond to the exact conditions to which the footings will be subjected.

Control of the Time Element.

The time element is one of the most important considerations in test procedure. After considerable experimentation it has been found, for the sizes of bearing area used in the load tests, that a time interval of one hour during which the load is held constant is sufficient to measure all but a negligible settlement for loads less than the bearing-capacity-limit or yield value of the soil. For loads in excess of the yield value there is, of course, progressive settlement and the essential consideration is that the time interval be held constant for all load increments and all sizes of bearing area. The constant time interval is by far the most important consideration in order that the variation in supporting capacity for different sizes of bearing area may be properly determined and used to evaluate the stress reactions developed by the body of soil.

Measurement of Load and Settlement.

The load is applied to the bearing in area in progressive increments sufficiently small so that a number of points on the load-settlement diagram may be obtained before progressive settlement occurs. The size of load increments must be estimated from the preliminary examination of the or determined by a preliminary test.

The settlement is marked on the settlement chart in the manner previously described immediately before and after each application of load and at frequent intervals during the time before another load increment is added. The continuous record of settlement which is obtained depicts accurately the behavior of the soil during various stages of loading and enables the operator to judge the gradual approach to the stage of progressive settlement.

II. Analysis of tests.

The analysis of the test data obtained deals with four variable factors which enter into the observations made in connection with the load tests. These are *time, load, settlement,* and *size of bearing area.*

The observations made in the field have been so conducted that the time element may be eliminated in the subsequent analysis inasmuch as all loads and settlements are for a constant time interval. It is considered that settlements measured for loads considerably less than the yield value represent the total settlement independent of the time, while loads in excess of the yield value produce a measure of the rate of settlement for any given intensity of load. The objective of the subsequent analysis dealing with the three remaining variables is to determine the load at which progressive settlement is produced in terms of stress reactions which properly express the dimensional factors controlling bearing capacity.

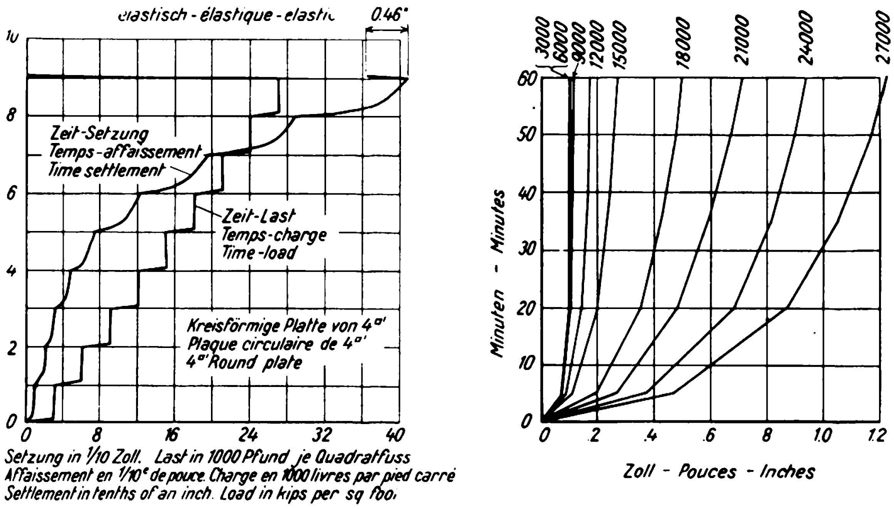

Fig. 5.

Time-load-settlement diagrams.

Time-Load-Settlement Diagrams.

The first step in analysis of the data is to represent the relation between time, load, and settlement for each bearing area in order to obtain a clear picture of the soil behavior. A typical time-load-settlement diagram is shown in Fig. 5.

Load and settlement have been plotted as abscissas against time as the ordinate. The time-load diagram shows the stages by which the applied load was built up and the continuous time-settlement diagram indicates the accumulation of the total settlement. The time-load-settlement diagram is merely a graphical representation of the test data and while not absolutely essential to the analysis it does present a comprehensive picture of the test results which is valuable as a background for the subsequent analysis. The time intervals during which the load is held constant are one hour and the change in settlement for each stage of load is shown at the right of Fig. 5 by the group of curves in which settlement is plotted against time for the various loads. The test used as an illustration is for a 4 sq. ft. round bearing area and is one of a series of three tests used in the subsequent analysis. The settlements for the first three load increments of 3,000, 6,000 and 9,000 lb. per sq. ft. are practically equal, indicating a straight-line relation of load and settlement typical of elastic behavior. A large percentage of the settlement took place as fast as the load was added and all of the measured settlement took place in the first 20 minutes. Definite elastic properties of the soil are also indicated by the elastic rebound of 0.46 in. which was recorded at the termination of the test. After the load had been increased to 12,000 lb. per sq. th. the settlement was progressive throughout the entire period of one hour during which the load was constant. For each succeeding increment there is a corresponding increase in the rate of settlement. From inspection of the time-load-settlement diagram the bearing-capacity-limit for the 4 sq. ft. area would appear to be between 9,000 and 12,000 lb. per sq. ft. The analysis of the test data given later verifies this conclusion and demonstrates that the bearing-capacity-limit of the 4 sq. ft. area is 9,630 lb. per sq. ft.

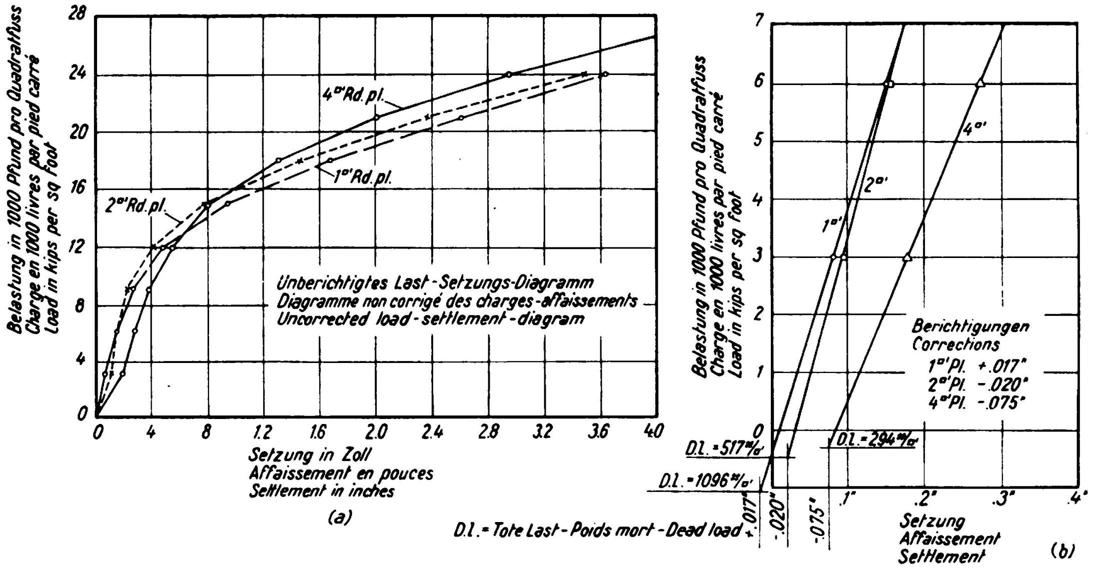

Fig. 6.

Uncorrected load-settlement diagrams and corrections.

Load-Settlement Diagrams.

The next step in the analysis is to plot the total settlement measured for each load increment against the load in pounds per square foot, eliminating the time element. A set of uncorrected load-settlement diagrams is illustrated in Fig. 6a. The relation between load and settlement for the first few load increments has been shown to a large scale in Fig. 6b. There two corrections which must be made before the data are used in further analysis. In the first place, a dead load, consisting of the weight of bearing plates, compression post and jack, has been applied to the soil before any measurement of settlement could be recorded. In the second place, a correction for excessive preliminary settlement must be applied to refer all curves to the same origin. An excessive preliminary settlement arises from the fact that it is impossible to obtain a perfect bearing between the plate and soil before the application of the first load. It is particularly noticeable in the larger plates where the intensity of pressure due to dead load is small. In the smaller plates the total dead load is sometimes enough to bring the plate into firm bearing with the soil before any additional load is added.

The dead load correction is obtained by adding to the measured settlements an increment of settlement proportional to the dead load pressure and determined by the best straight-line relation of load and settlement for the first few live load increments. An excessive preliminary settlement is determined by the intersection of this best straight-line relation with the horizontal axis of settlement and is to be subtracted from measured settlements. The net correction is the summation of the two partial settlement corrections. This net correction may be determined graphically in one operation as shown in Fig. 6b by extending the straight-line relation of load and settlement below the axis of zero load an amount equal to

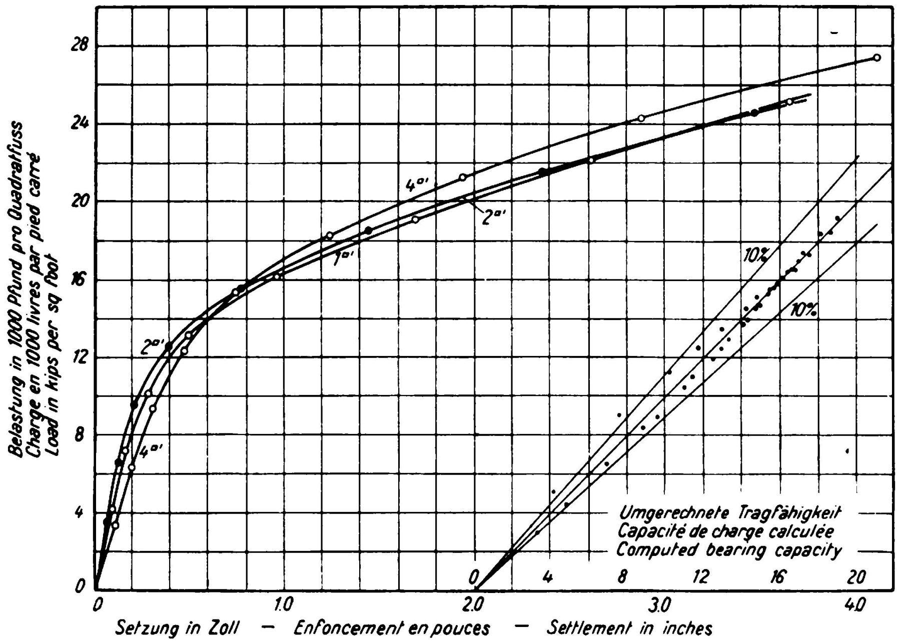

Fig. 7.

Corrected load-settlement diagrams.

the dead load and measuring the net correction as that settlement necessary to be added or subtracted to bring the origin of the line on the axis of zero settlement.

The corrected load-settlement diagrams which are shown in Fig. 7 are obtained by plotting the corrected settlement against the total load, which is the sum of the dead load plus the live load increments. These diagrams present the test data which are now to be used to determine the stress reactions which accompany a variation in the intensity of load carried by the different sizes of bearing areas having different perimeter-area ratio.

Bearing Capacity Relations.

In order to determine the variation in bearing capacity with the size of the loaded area, the loads carried at different amounts of settlement are read from the corrected load-settlement diagrams. An equation expressing this variation has been determined by statistical methods. The analyses of some fifteen series of bearing capacity tests has indicated that a linear relation between load and perimeter-area ratio, is most satisfactory in reproducing such experimental observations.

Such a linear equation has been formulated on the basis that the total load carried by any given bearing area may be expressed as the combined effect of two stress reactions which have been designated as perimeter shear and developed pressure.

This equation is as follows:

$$W = mP + nA \tag{1}$$

in which W = total load in pounds
m = perimeter shear in pounds per lineal foot of perimeter
n = developed pressure in pounds per square foot
P = perimeter in feet
A = area in square feet.

In Fig. 8 is illustrated the conditions under which these stress reactions develop support for the loaded area. The zone which enters into the support of the bearing area has been commonly referred to as the compression cone. This region has been further divided by designating that portion immediately under the bearing area as the central column. As the bearing area is loaded some of the load is distributed by shearing resistance on the perimeter surfaces to that portion of the compression cone surrounding the central column. It is the accumulation of the shearing resistance acting on the boundary surface which gives rise to the boundary stress reaction which has been designated as perimeter shear. Up to the present time the distribution of this shearing resistance with the depth has not been determined.

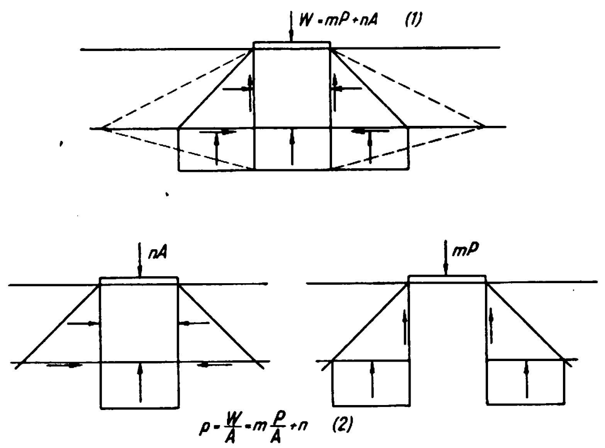

Fig. 8.
Stress reactions in the linear equation for bearing capacity.

Consequently, this source of resistance is included in the equation for bearing capacity as a concentrated force acting along the boundary and expressed in units of pounds per lineal foot.

As the loading progresses the shearing resistance on the perimeter surface becomes inadequate to continue the lateral transmission of vertical force and additional increments are carried by the central column as developed pressure. This concentration of force in the central column may be continued until the ability of the central column to carry vertical load is exceeded. Its supporting capacity arises from two sources. In the first place, vertical concentrations of load may be increased and transmitted downward until the difference between the vertical and lateral pressure exceeds shearing resistance of the soil on inclined planes of maximum shear. In the second place, additional increments may be added without causing failure as long as the lateral pressure furnished by the material surrounding the central column is not exceeded. Summation of these several factors are combined in a single stress reaction which has been defined as developed pressure and which is independent of the size of the bearing area.

Inasmuch as it is customary in building practice to deal with bearing capacity in pounds per square foot, the equation for total load may be conveniently expressed in terms of an average intensity of pressure which is defined as the bearing capacity. This equation for bearing capacity may be obtained by dividing both sides of the equation for total load by the area of load application and is as follows:

$$p = m \frac{P}{A} + n \qquad (2)$$

In the linear equation (2) which is used in load test analysis the stress reactions m and n are unknowns which are to be determined by the solution of a set of equations representing the load in pounds per square foot carried by the several different bearing areas at any given amount of settlement. Any two equations

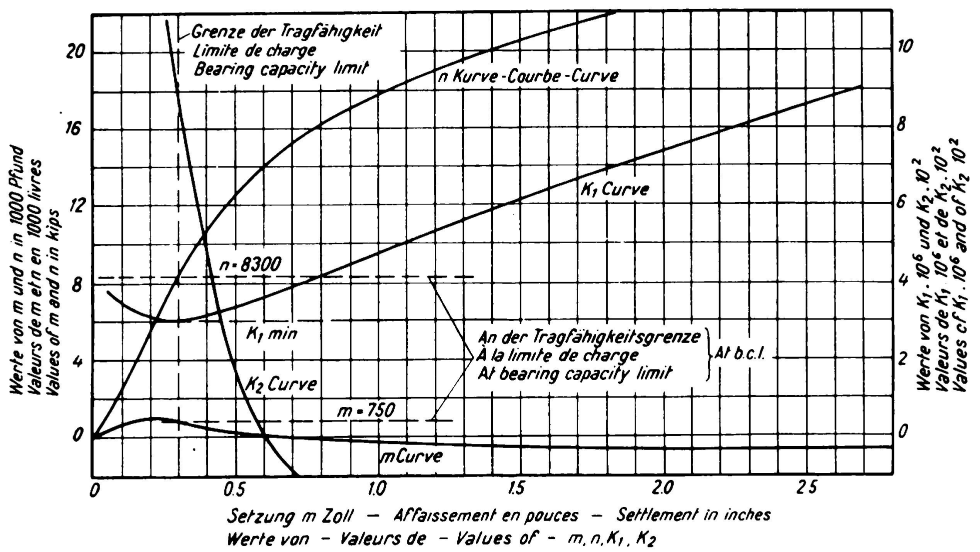

Fig. 9.

Stress reactions and soil resistance coefficients.

are sufficient for one determination of the stress reactions m and n. It has been found desirable, however, to test three or more bearing areas in order to obtain a better average value of these factors. The variation in the stress reactions for the entire range of the tests is determined by the solution of such sets of equations for various amounts of settlement.

In Fig. 9 are shown the values of the stress reactions m and n for the test series which include the three test areas shown in Fig. 7. The accuracy with which equation (2) reproduces the test data may be determined by substitution of the most probable values of m and n and comparing the results with the actual measured load per unit area for each bearing area. Such a comparison is shown in Fig. 7 wherein the computed values of bearing capacity are plotted against the measured load. With the exception of several points in the lower range of load the agreement between the equation and observation is well within an error of 10 per cent, which in addition to checking the validity of equation (2) indicates the experimental accuracy in test procedure.

Soil Resistance Coefficients.

The purpose of any analysis of soil test data is to determine the maximum load which will be sustained without excessive settlement. It is sufficient to consider that there are two stages of soil behavior portrayed by the test data. The first stage is one in which the soil will sustain the applied load without continued or progressive settlement. The second stage is one in which the soil is stressed beyond its yield value resulting in progressive settlement which continues at an essentially uniform rate as long as the load conditions remain unchanged. The point of transition between these two stages may be defined as the *bearing-capacity-limit,* the determination of which is the primary objective of the load tests and toward which the analysis is directed. The m and n curves as shown in Fig. 9 do not in themselves furnish any criterion for the bearing-capacity-limit of the soil, and it is necessary to turn to the soil resistance coefficients K_1 and K_2 which are derived as shown in the Appendix from the measured values of settlement, Δ, m and n, to define the bearing-capacity-limit. K_1, which is the ratio of settlement divided by developed pressure, is defined as the *coefficient of settlement* $\left(K_1 = \frac{\Delta}{n}\right)$. It analogous to the well-known coefficient of compressibility except that it expresses the total settlement as volume change in the body of soil included within the compression cone, rather than being expressed in deformation per unit volume. K_2 is the ratio of perimeter shear divided by developed pressure and is defined as the *stress reaction coefficient* $\left(K_2 = \frac{m}{n}\right)$. It expresses the relative importance of the two stress reactions involved in bearing capacity.

The bearing-capacity-limit of the soil may be determined as the minimum value of K_1 or the maximum value of K_2 depending upon the sequence in which the two types of resistance are developed. For a relatively compressible soil the developed pressure is small for the lower range of loads and the major portion of the applied load is carried by perimeter shear. As the settlement increases and the bearing plate penetrates the surface, developed pressure increases and the values of K_1 decrease as shown in Fig. 9. The decreasing values of K_1 show that the resisting pressure is increasing faster than the settlement and indicate a margin of resistance which is available to bring the loaded area to equilibrium if no more load were to be added. The minimum value of K_1 defines the maximum developed pressure in the case of soils which are relatively compressible. Subsequent increasing values of K_1 in which the settlement increases more rapidly than the developed pressure show that increments of settlement are accumulating without proportional increase in resistance and signify the stage of progressive settlement. Meanwhile the values of K_2 are decreasing and show no evidence of critical changes in behaviour. The supporting capacity due to perimeter shear was available in the initial stage of loading and after having been fully utilized exerts no further influence on the transition to the stage of progressive settlement.

In Fig. 9 the bearing-capacity-limit at a minimum value of K_1 occurs at a settlement of 0.3 in. with a value of perimeter shear m equal to 750 lb. per lin. ft. and developed pressure n equal to 8300 lb. per sq. ft. As an illustration of the use of the data in the linear equation (2) the bearing-capacity-limit of the 4 sq. ft. round plate may be computed as in the following example:

$$p = m \cdot \frac{P}{A} + n \tag{2}$$

$$\frac{P}{A} = 1.77 \qquad m = 750 \qquad n = 8\,300$$

$$p = 750 \times 1.77 + 8300 = 9630 \text{ lb. per sq. ft.}$$

Another series of load tests is shown in Fig. 10 as an example in which the maximum value of K_2 defines the bearing-capacity-limit. In this case the load

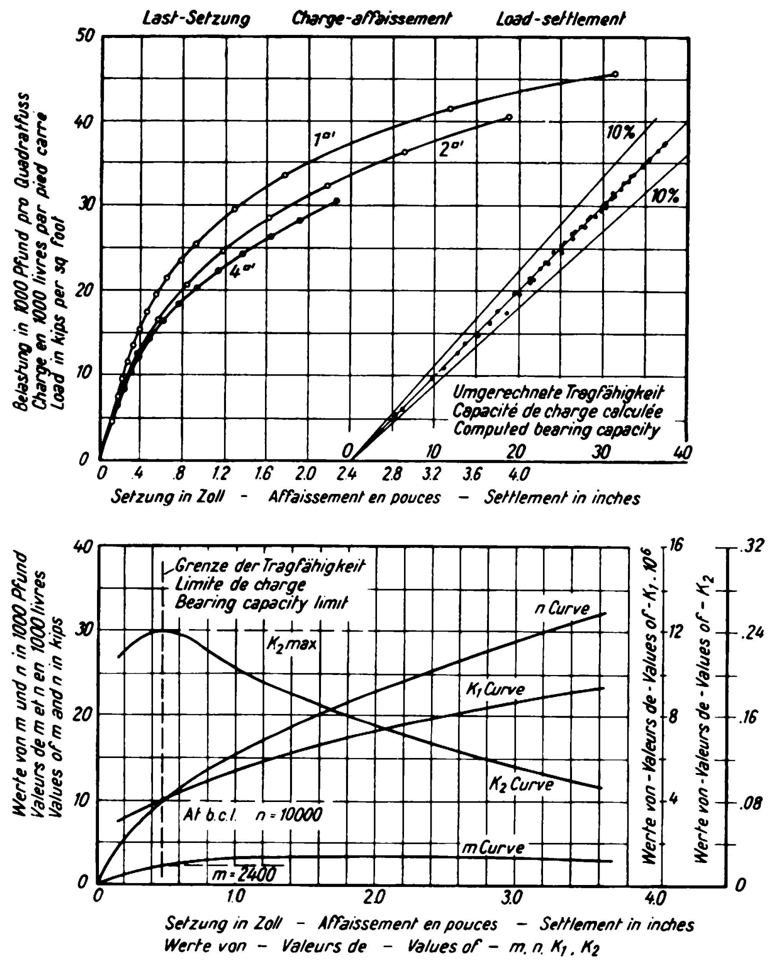

Fig. 10.

Soil resistance coefficients for incompressible soil.

tests were made with round bearing plates on the same soil as the tests shown in the previous example but with the areas confined by a surrounding overburden of approximately 1000 lb. per sq. ft. The developed pressure is increased to 10,000 lb. per sq. ft. the perimeter shear to 2400 lb. per lin. ft., and the bearing-capacity-limit occurs at a somewhat higher settlement of 0.45 in. The agreement between measured loads in pounds per square foot and the bearing capacity computed from the linear equation is exceptionally good.

In the case of a relatively incompressible soil in which resistance to volume change is high the developed pressure is available as the initial resistance and the perimeter shear is developed as the settlement increases. As a result of reversing the sequence with which the stress reactions are developed, the coefficient of settlement K_1 increases throughout the entire range of the test and shows no critical value. The stress reaction coefficient K_2, however, increases during the initial stages and reaches a maximum value which represents the maximum amount of load which may be distributed to the body of soil by shear on the boundary surfaces. The maximum value of K_2 then becomes the criterion for the bearing-capacity-limit.

III. Practical application of load test data.

The data from load tests as exemplified in the preceding examples may be applied directly to the design of footings by use of equation (2) for bearing capacity at any selected settlement.

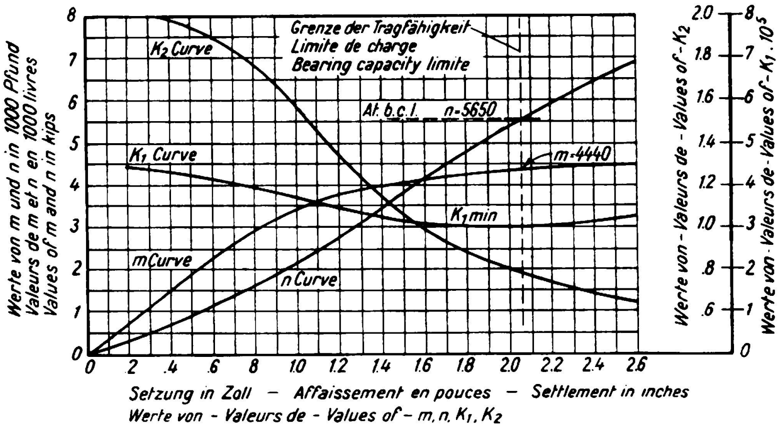

Fig. 11.

Load test data for Fort street grade separation.

Three examples will be given in which load tests have been so used and for which observations of settlement on the completed structures are available.

Fort Street Grade Separation.

Fig. 11 shows the values of m, n, K_1 and K_2 for a series of tests on round plates confined which served as the basis of designing six piers for a grade separation in Detroit, Michigan. The load test values at the bearing-capacity-limit are as follows:

Settlement $\Delta = 2.05$ in.
Perimeter Shear $m = 4440$ lb. per lin. ft.
Developed Pressure $n = 5650$ lb. per sq. ft.

The piers were designed for 1.0 in. settlement for total load including dead weight and assumed live load. Settlement measurements were made at three points on each pier. Approximately three months after the full dead weight of the structure had been imposed on the soil the piers came to equilibrium and showed no further signs of progressive settlement due to these loads. A comparison of actual settlements and predicted settlements is given in Table I. The values of m and n at settlements of 1.0, 0.9, and 0.8 in. have been taken from the test data and the bearing capacity for each pier computed by use of equation (2). The predicted settlement for the applied dead load is found by interpolating between the computed capacities for assumed settlements.

The agreement between the settlement of the piers and the settlement predicted from load test data is sufficiently close to present convincing evidence of the reliability of load tests properly used. The only substantial disagreement in the case of Pier 5 is thought to be largely due to construction difficulties which arose when the excavation for that footing became flooded by heavy rains. The soil immediately beneath the footing was considerably disturbed and a bed of coarse gravel was placed beneath the footing before pouring. The tendency of the clay to squeeze into the voids in the gravel is thought to be the cause of the additional settlement experienced.

Miller Road Grade Separation.

In Fig. 12 are shown the values of m, n, K_1 and K_2 for a series of tests conducted by the Wayne County Road Commission in connection with the design

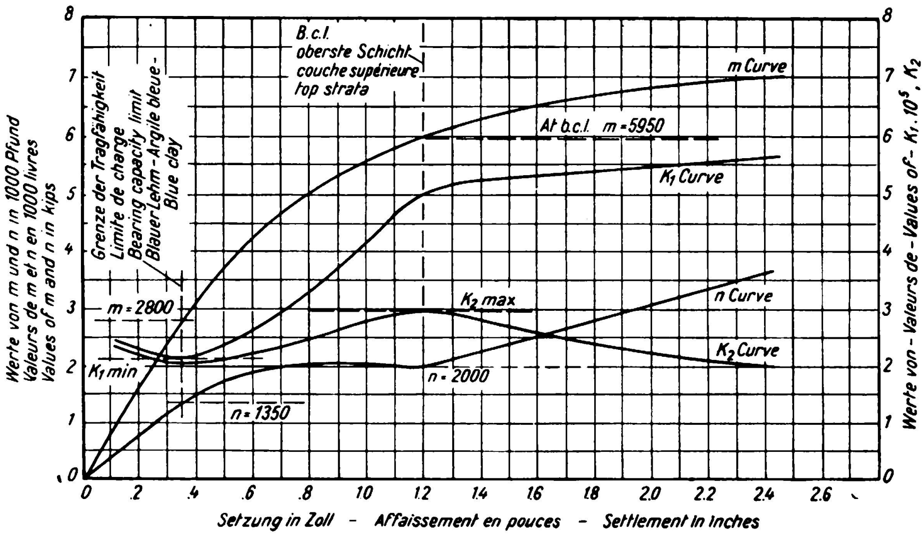

Fig. 12.

Load test data for Miller road grade separation.

of a major grade separation at Miller Road in Detroit. The soil conditions were somewhat out of the ordinary and were very critical for the construction of the large structure proposed. It was found necessary to place the pier footings on a layer of stiff red clay only 18 in. thick, which was underlain by a body of very plastic blue clay extending down to hardpan at a depth in excess of 100 ft.

Tests were conducted at the elevation of the stratum of stiff red clay in the hope that the mat action of this 18 in. layer would be sufficient to carry safely the superimposed loads. The results of the tests were particularly illuminating and showed two distinct bearing-capacity-limits or critical points.

Table I.

Piers		Dimensions		Area	ratio	Vertical Pressure	Bearing Capacities at Various Settlements						Interpolated Settlement		Measured Settlements	Measured Settlements Average
							1.0″		0.9″		0.8″					
							n = 2225 m = 3500		n = 1940 m = 3270		n = 1675 m = 2970					
		Length	Width		$\frac{P}{A}$											
		ft.	ft.	sq. ft.		Pounds per sq. ft.	$m\frac{P}{A}$	$m\frac{P}{A}+n$	$m\frac{P}{A}$	$m\frac{P}{A}+n$	$m\frac{P}{A}$	$m\frac{P}{A}+n$	in.	ft.	ft.	ft.
	E. end														0.061	
1	middle	166	9	1494	0.234	2500	820	3045	765	2705	700	2375	0.84	0.070	0.980	0.073
	W. end														0.060	
	E. end														0.074	
2	middle	166	9	1494	0.234	2380	820	3045	765	2705	695	2370	0.80	0.067	0.084	0.078
	W. end														0.075	
	E. end														0.074	
3	middle	166	8	1328	0.262	2330	920	3142	860	2800	775	2450	0.76	0.064	0.078	0.073
	W. end														0.066	
	E. end														0.083	
4	middle	166	7	1162	0.297	2350	1040	3267	975	2915	885	2560	0.74	0.062	0.072	0.079
	W. end														0.084	
	E. end														0.108	
5	middle	150	9	1350	0.235	2550	825	3050	770	2710	700	2375	0.85	0.071	0.108	0.102
	W. end														0.091	
	E. end														0.073	
6	middle	150	8	1200	0.263	2600	920	3147	860	2800	780	2455	0.84	0.070		0.078
	W. end														0.082	

The first critical bearing capacity is designated by a minimum value of K_1 at a settlement of 0.35 in. The minimum value of K_1 $\left(K_1 = \frac{\Delta}{n}\right)$ represents the maximum pressure that the underlying material is capable of developing without progressive settlement. The values of m and n are 2800 lb. per lin. ft. and 1350 lb. per sq. ft. respectively.

As the loading continues, the developed pressure increases very slowly, and for a considerable range of settlement remains almost constant at 2000 lb. per sq. ft., even showing a slight recession. In the meantime the perimeter shear shows a considerable increase, representing the shearing strength of the top layer of stiff red clay and portraying considerable ability of the top layer to distribute load to the underlying soil. The critical point of this later stage is designated by a maximum value of K_2 $\left(K_2 = \frac{m}{n}\right)$. The increasing values of K_2 are due to the fact that m is increasing faster than n and indicates of margin

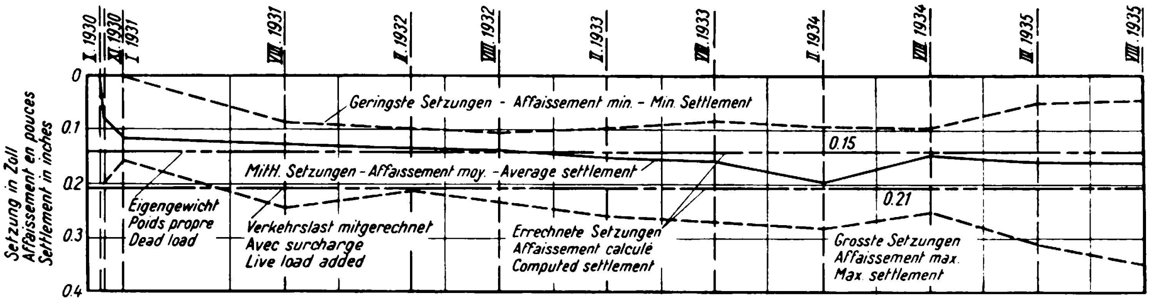

Fig. 13.

Settlement of piers at Miller road grade separation.

of strength due to the perimeter stress reaction. The maximum value of K_2 and subsequent decreasing values indicate that this margin has been exhausted, and the bearing areas are on the point of shearing through the top layer.

On the basis of these test data 48 piers were designed as spread footings resting on the layer of stiff red clay and having a total soil pressure of 1550 to 1700 lb. per sq. ft. Settlement reading were taken of representative piers after the columns seats were in place, and the measured settlement for the remainder of the applied load is shown in Fig. 13. The predicted settlement is shown for dead and live load, and the favorable comparison between measured settlement and settlement predicted for dead load seems to indicate that moving live loads are of minor importance in causing settlement. There is shown the average settlement of all piers as well as the maximum and minimum of individual piers. The average total settlement to date is 0.16 in., slightly greater than the computed settlement for dead load of 0.15 in. and somewhat less than the computed settlement for total load.

The settlement measured for the past four years has been negligible amounting only to several hundredths of an inch, or less than could reasonably be considered within the accuracy of the level measurements. On the other hand, inasmuch as the level measurements are based on an average of a large number of observations they may indicate a slight trend toward an accumulation of settlement due to intermittent applications of live load. The total settlement to be anticipated is 0.21 in. if the live load assumed in the design were realized.

Northwestern Elevated Storage Tank.

The third example of a structure for which the footings were designed from load test data is given in Fig. 14. The footings for this 2,000,000-gal. storage tank were designed on the basis of the series of load tests conducted by the City of Detroit, Department of Water Supply, and illustrated in Fig. 10. The tank was supported by 20 columns equally spaced around the circumference. The substructure consisted of a continuous ring of reinforced concrete 21 ft. wide and 3 ft. thick. The outside diameter of the ring was 101.5 ft., having then a perimeter-area ratio of 0.095. The footing sustains a bearing pressure of 4600 lb. per sq. ft., which, according to the soil-test data, would result in a settlement of approximately 0.17 in.

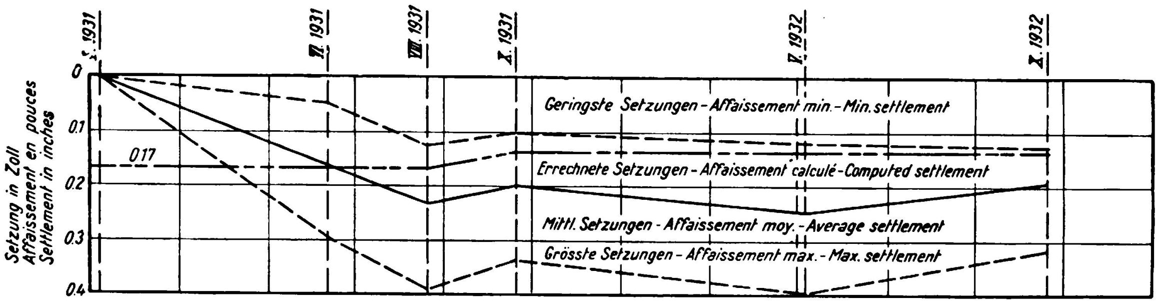

Fig. 14.

Settlement of footing of northwestern tank.

In Fig. 14 are shown the results of settlement measurements on the finished structure compared with the predicted settlement. The settlement is shown as the average of the readings taken at 20 columns as well as the maximum and minimum of individual readings. The tank was under construction until June 12, 1931, and was full from that date to August 19, 1931. On October 20, 1931, the tank was only partly filled, and there was a positive decrease in the total settlement interpreted as elastic rebound. The actual load varies from time to time depending on storage and these changes are reflected in the settlement measurements. The average total settlement to October, 1932, is 0.19 in., only slightly in excess of the design settlement. Data on settlement since 1932 are not available at the present writing. The Department of Water Supply takes settlement measurements on all storage tanks over a period of years and have not reported any significant changes in the footing elevations for this particular structure.

IV. Conclusion.

In the course of the present discussion the attempt has been made to demonstrate that load tests if properly conducted and analyzed on a sound fundamental basis will produce not only reliable information but data that bears directly on the practical problems which the designing engineer must solve. The writer is in hearty sympathy with demand that the science of soil mechanics should produce working tools of real practical value rather than avoiding the issue and mystifying the practicing engineer with the complexities of soil physics. When this demand can be met without sacrificing thorough treatment and sound principles of mechanics it should be done. When the demand is for rule of thumb methods, the usefulness of which is measured by the lack of mental effort required in their application, it does not deserve the same attention.

Foundation problems can never be handled in terms of blind generalities. The large number of factors and the variety of conditions encountered make each problem peculiar to itself. Even with the most satisfactory methods of measurement and soundest theories for using the results of measurement, good judgment will always be required in their application.

As far as load tests are concerned, their successful use in the examples cited and in other cases on record is proof of their reliability which stands unshaken by criticism that may be directed at methods and conceptions used in their application. A sincere effort has been made to develop methods and conceptions which are simplified to the greatest possible extent without sacrificing sound fundamentals. For the present this seems to be the logical basis on which the specialist in soil mechanics and the practicing engineer should meet.

B. General equation for settlement of a loaded area.

A general equation which expresses the relation between settlement, size of bearing area, and load has been derived by integrating soil deformation within the compression cone. In Fig. 15 the relations included in this general equation are illustrated. The problem involves a finite bearing area which has been taken as a square of width b. The increase in the lateral dimension of the loaded area is given by r the tangent of the angle of spread. In a depth h according to the linear approximation of pressure distribution the total load W will be carried over an area with a lateral dimension of $(b + 2\,rh)$. The combination of rectangular and triangular distribution has been selected as the most satisfactory representation of pressure distribution in that it properly portrays lateral distribution of the vertical load as a boundary phenomenon and provides a convenient basis for segregating the two different types of stress reaction involved. Accepting this pressure distribution the total load may then be represented as an equivalent uniform pressure over an area with a total lateral dimension of $(b + rh)$, the area being $(b + rh)^2$.

At any depth y below the surface an element of thickness (dy) will be subjected to an equi-

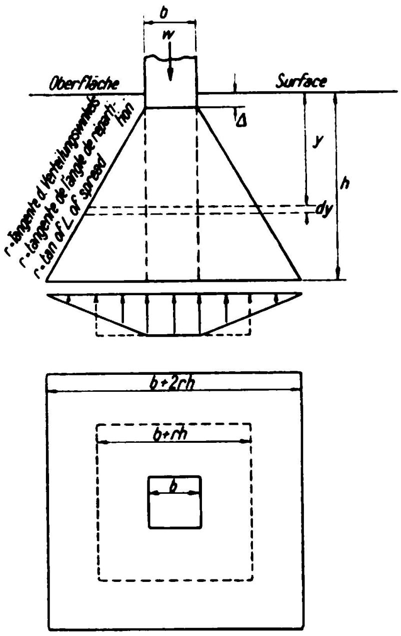

Fig. 15.
Pressure distribution and settlemen beneath a loaded area.

valent uniform pressure p_y. The modulus of incompressibility I is defined as the load per unit area divided by the settlement or deformation per unit depth.

$$I = \frac{p_y}{d\Delta / dy} = \frac{W / (b + ry)^2}{d\Delta / dy}$$

$\Delta =$ Total Settlement in feet.

$$d\Delta = \frac{W\,dy}{I\,(b + ry)^2}$$

$$\Delta = \int_0^h \frac{W\,dy}{I\,(b + ry)^2} = \frac{W}{I}\left[-\frac{1}{r\,(b + ry)}\right]_0^h$$

$$\Delta = \frac{Wh}{I\,(b^2 + brh)} \tag{3 a}$$

Reducing to terms of bearing capacity p and perimeter-area ratio

$$\Delta = \frac{pAh}{I\left(A + P\frac{rh}{4}\right)} \qquad \begin{aligned} W &= pA \\ P &= 4\,b \\ A &= b^2 \end{aligned}$$

Let

$$\frac{h}{I} = K_1 \qquad \frac{rh}{4} = K_2$$

$$\Delta = \frac{K_1 p}{1 + K_2 \frac{P}{A}} \tag{3}$$

Equation (3a) is a general solution first obtained by C. C. Williams.[1] This form is subject to some limitation inasmuch as it contains quantities r, h, and I which are not subject to direct measurement under practical conditions. It is necessary further to express the general relation for settlement in terms of quantities that may be measured by test. This may be done by introducing two soil resistance coefficients K_1 and K_2 and establishing their relation to the straight-line equation (2). Equation (3) is a general expression which shows the relation between settlement, perimeter-area ratio, and bearing capacity. In order to show the relation between the straight-line equation (2) and equation (3) it is merely necessary to consider the settlement Δ as constant and express bearing capacity in terms of perimeter-area ratio.

$$K_1 p = \Delta + \Delta K_2 \frac{P}{A} \tag{2}$$

$$p = \frac{\Delta K_2}{K_1}\frac{P}{A} + \frac{\Delta}{K_1}$$

[1] The Science of Foundations — Its Present and Future — Discussion by *C. C. Williams*, Trans. Am. Soc. C. E. Vol. 93, 1929, p. 309.

$$p = m \frac{P}{A} + n \tag{2}$$

$$m = \frac{\Delta K_2}{K_1} \qquad n = \frac{\Delta}{K_1}$$

$K_1 = \frac{\Delta}{n}$ $K_2 = \frac{m}{n}$ (Soil Resistance Coefficients).

From equation (3) it is seen that the only quantities involved in addition to the variables of load, settlement, and perimeter-area ratio are the two coefficients K_1 and K_2. From the direct relation to equation (2) it is apparent that K_1 and K_2 may be evaluated in terms of Δ, m, and n which have been measured by bearing capacity tests. In the analysis m and n were determined for any given settlement and for the same conditions it follows that K_1 and K_2 are constants. The values of m and n, the stress reactions, will vary for different types of soil and for different ranges of load. The coefficients K_1 and K_2 will also vary with m and n and thus they become the coefficients which express the essential properties of the particular soil and on the basis of which the varying behavior of the material for different ranges of load may be definitely evaluated.