This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

[https://www.issmge.org/publications/online-library](https://www.issmge.org/publications/online-library)

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.
Yesterday in my opening address I called your attention to the importance of adequate evidence. No rule and no procedure should be accepted for practical usage unless its validity is sustained by adequate evidence obtained from observations on full-sized structures. The topic of this discussion gives me an opportunity to illustrate the meaning of my comments by means of specific examples.

Instantaneous and Gradual Settlement. If we construct a building and observe the movement of its base during and after construction with adequately sensitive measuring devices we always find that the building settles to some extent, regardless of what the material beneath its foundations may be. If the building is not perfectly rigid, the settlement is almost always unequal, though the load may be perfectly uniformly distributed. The observed settlements can be divided into two parts. One part occurs simultaneously with the application of the load. The second one represents the gradual increase of the settlement after the load has assumed its maximum value. These two fractions will briefly be called the instantaneous and the gradual settlement. For a given system of loads both parts of the settlement depend on the compressibility, the permeability, the thickness and the succession of the soil strata located beneath the base of the foundation. Thus we have assembled a group of empirical facts to start with.

Methods for Investigation. In order to bring some system into the confusing variety of observed facts and in order to learn something about the relation between the settlement and the factors cited above, we are compelled to investigate the influence of each one of those factors individually. A considerable part of this investigation has already been carried out. However, since the investigations were by necessity based on a radical simplification of the real properties of the soils, no final conclusion can be reached without checking the theoretical results against the results of observations on full-sized structures. The major part of this second task still remains to be performed although the Proceedings of our Conference contain already a great number of valuable and promising contributions.

State of Stress Beneath Loaded Areas. In every case the foundation represents a load which acts on or at some depth below the surface of a more or less compressible stratum or system of strata. The load produces in the loaded material some state of stress which in turn causes the instantaneous and the gradual deformations which are responsible for the settlement.

The state of stress produced by the load in the loaded material has been investigated many times, for very different materials, both by theory and by experiment. All these investigations, without exception, led to the following conclusions. If we make a horizontal section through the loaded material at some arbitrary depth below the base of the load, the normal stresses which act along this section have a maximum beneath the central part of the loaded area and gradually decrease in either direction, as shown in Fig. 1a. Furthermore, if we determine the distribution of the normal stresses for a great number of horizontal sections and plot the maximum vertical normal stress for each section as an abscissa against the depth of the section below the base of the load we obtain a curve 1b whose abscissae decrease with depth.

The Bulb of Pressure. If we determine the position of all the points beneath a loaded area, in which the normal stress on a horizontal section is equal to a definite fraction, 1/n, of the unit load p, we obtain a bulb-shaped surface which passes through the rim of the loaded area. Fig. 2a shows a
section through such a surface, commonly called the "bulb of pressure". This conception played quite an important part in the early American literature concerning the bearing capacity of the soils. For any given load we can get an infinite variety of pressure bulbs, since a different bulb corresponds to every value of \( n \) between zero and one. Therefore the term "pressure bulb" has no definite meaning, unless it refers to a certain, though arbitrary, value of \( 1/n \), for instance to \( 1/5 \) of the unit load \( p \). If the loaded material is perfectly homogeneous to an infinite depth, the depth \( t_n \) of the bulb, corresponding to a definite value of \( 1/n \) increases in simple proportion to the width, \( b \), of the loaded area, provided all the loaded area have a similar shape. Hence if we have two similar loaded areas with a width \( b \) and \( B \), located above bulbs of pressure with a depth of \( t_n \) and \( T_n \) respectively, the value \( T_n \) is determined by the simple relation

\[
T_n = t_n \cdot B/b
\]

(1)

For practical purposes it seems most appropriate to select a value of \( n = 5 \). This recommendation is based on the fact that the greater part of the settlement takes place within a depth smaller than \( T_n = 5 \), unless there is an unusually soft layer of clay or peat located at a greater depth beneath the loaded area.

If the loaded area is square or round with a side or diameter \( B \), the value of the ratio

\[
\frac{T_n}{B} = \frac{t_n}{b} = \text{const} = f(n)
\]

(2)

and the settlement due to distortion and compression of the loaded solid within the bulb with the depth \( T_n = 5 \) represents approximately 80% of the total settlement. For a perfectly uniform and very thick stratum of sand the value of the ratio \( T_n/B \) is somewhat smaller than \( 1.5 \).

If the loaded material is stratified the value of the ratio \( T_n/B \) depends not only on \( n \) but also to a certain degree on the soil profile. If the strata which occupy the lower part of the pressure bulb are considerably stiffer than the upper ones the value of the ratio is greater and if they are softer, the value of the ratio is smaller than the one corresponding to a homogeneous material. Yet in any event, the depth \( T_n \) of the bulb increases with increasing values of \( B \). In practice, the compressibility of the soil always changes with depth in a more or less erratic fashion, although according to the results of ordinary test-borings the ground may appear to be homogeneous. The result of a small scale loading test depends merely on the properties of the soil located within the shallow bulb of pressure produced by the bearing block, while the settlement of the full-sized structure also depends
Combined Bulbs of Pressure. If a structure is supported by individual footings, the bulbs of pressure for the individual footings combine as shown in Fig. 3, unless they are spaced very far apart. The depth of the resulting bulb of pressure can be many times greater than that of the individual ones. In a similar manner the bulbs of pressure produced by loading the piles of a pile foundation also combine. This process increases the depth of the body of soil subject to intense compression by the superimposed load to many times the depth of the soil which is affected by a loading test on an individual pile.

Estimate of Settlement from the Results of Loading Tests, from Previous Settlement Records and from the Results of Soil Tests. The ultimate purpose of preliminary investigations for the design of foundations is to estimate the settlement. To achieve this purpose one of the following three methods may be used:

A. By extrapolation from the results of small-scale loading tests,
B. By inference from existing settlement records, and
C. By computation from the results of soil tests on undisturbed samples.

Method A. As indicated in the foregoing discussion, extrapolation from the results of loading tests is permissible only in those cases in which the subsoil is homogenous throughout the depth \( T_n = 5 \) of the bulb of pressure of the proposed foundation. From this statement we can draw at once the conclusion that no settlement estimate of the type A should be accepted unless it is accompanied by conclusive proof that the necessary degree of the homogeneity is present. The second requirement for a satisfactory estimate of the type A consists in a knowledge of the relation between the settlement of a small and of a large area. For a perfectly elastic, homogenous solid the settlement of loaded areas on the physical properties of the deeper strata.

In order to decide whether or not the soil can be considered practically homogenous and in order to obtain information on all those strata whose properties are likely to influence the settlement of the full-sized structure it is necessary, first of all, to drill to a depth which is at least equal to the value of \( T_n = 5 \) for the proposed building. The second requirement consists in securing undisturbed samples. If this cannot be done, the nature of the strata located between the base of the future foundation and the depth \( T_n = 5 \) must be investigated by tests performed either in the drillhole or in a shaft. Test-boring records based on a visual inspection of the drill samples can be utterly misleading, because experience has shown that every disturbance of the original structure of the soil has a very marked effect on its essential physical properties. For this reason an important part of the Proceedings of this Conference is devoted to methods of securing undisturbed samples.

The facts presented in the preceding paragraph also lead to the conclusion that the settlement of an individual footing or of a single pile is always smaller than the settlement of an entire group of such units under equal load per unit. The value of the ratio between the settlement of the group and that of the unit depends on the number of units, on their spacing and on the compressibility of the soil located between the base of the bulb of the unit and that of the entire group. These two types of "bulbs" are shown in Fig. 3.
condition and in addition the knowledge of the relation between the size of the loaded area and the corresponding settlement was utterly inadequate. Therefore the results were practically worthless and the method as such is in dire need of revision.

In many cases the settlement of a building gradually increases to many times that observed after all the loads had been applied. There is not a single case on record which demonstrates the possibility of a reliable prediction of the time rate of settlement of such buildings from the results of loading tests. Hence the application of the method is further limited to those cases in which the major part of the settlement occurs while the load is applied. The method also fails to inform us on the distribution of the settlement over the base of a flexible structure. In many cases it is far more important to know the distribution of the settlement than to get information on the average or the maximum value. Hence it seems that the application of the method A will always be limited to very few cases, regardless of the future increase of our capacity for reliable extrapolation.

Method B. Another method for predicting the settlement of a proposed foundation consists in consulting settlement records of existing buildings. In order to use this method to full advantage, two requirements must be satisfied. First of all the settlement record should be complete, that is, it should cover the entire downward movement of the building since the time when the foundation was laid and it should include not only a time-settlement but also a time-load record. If the structure is not perfectly rigid, the record should include the movement of at least one dozen reference points located at different parts of the periphery and of the interior of the building. The second requirement consists in a complete record of the nature and the property of the soil located between the base of the foundation and a depth $T_1 = 5$ beneath this base for both the existing and the proposed building. The data required for reliable identification depend on the nature of the soil. The compressibility of strata of sand or of silt can only be compared on the basis of the results of penetration tests in drill-holes or of loading tests in shafts, at various levels down to a depth $h = 5$. Numerous testing methods of this kind have been invented during the last ten years. Another procedure for estimating the relative compressibility of a bed of sand or silt was worked out by Hertwig in Berlin. It is based on a measurement of the characteristic frequency and the velocity of the transmission of elastic waves through the material. Finally, in the Paper No. B-2, Vol I of the Proceedings of this Conference, J. T. van Bruggen describes a method of securing undisturbed samples of clean sand, which permits the determination of the compressibility of the same in the laboratory. It is one of the tasks of future investigations to find out which one among the different methods is likely to furnish the required results at a minimum amount of time and expenditure.

On the other hand, an adequate description of the properties of beds of clay can only be secured by means of laboratory tests on undisturbed samples, including compression and consolidation tests.

The method B, based on existing records, has the advantage of involving no theoretical conceptions other than those on which the selection of the criteria for identification are based. A consistent application of the method is bound to increase our knowledge of the subject independent of the progress along theoretical lines and in some cases such as those involving the settlement of buildings founded on beds of sand or silt it is the only one which is at present at our disposal. Yet in order to use this method, the practice of adequately investigating the subsoil and of observing the settlement must become far more universal than at present.

By far the most promising field for a successful application of the method B, based on analogy, is to be found in big cities. Although the soil located beneath our cities may vary to a certain extent in type and consistency, these variations are seldom important enough to prevent the existence of relatively simple relations between the load, the size and shape of the loaded area, the type of foundation, the soil profile and the settlement. If adequate settlement records for a large number

with a similar shape increases strictly in direct proportion to the width of the loaded areas. As a crude approximation, this rule also applies to rock and to very cohesive, homogenous soils. The relationship for an elastic homogenous solid is shown by the straight line $C_1$ in Fig. 4. In this figure the abscissae represent the width $B$ of the loaded area and the ordinates the settlement of a rigid slab which covers this area. However, the more the character of the loaded material approaches that of a clean sand, the more important becomes the deviation from the straight line law. For a clean sand we obtain a curve similar to $C_2$ in Fig. 4. Up to the present, our knowledge of the relation between the size of the loaded area and the corresponding settlement has been obtained almost exclusively from tests on a relatively small scale. Since the line which represents this relation is likely to be curved, attempts to extrapolate should be accepted with caution. The gaps in our knowledge of this relation can only be closed by further observations including the measurement of the settlement of full-sized structures, supported by fairly homogenous strata of great depth.

In the past no attention has been paid to the homogeneity of the subsoil and the necessity of a complete record of the nature and the property of the soil located between the base of the foundation and a depth $T_1 = 5$ beneath this base for both the existing and the proposed building. The data required for reliable identification depend on the nature of the soil. The compressibility of strata of sand or of silt can only be compared on the basis of the results of penetration tests in drill-holes or of loading tests in shafts, at various levels down to a depth $h = 5$. Numerous testing methods of this kind have been invented during the last ten years. Another procedure for estimating the relative compressibility of a bed of sand or silt was worked out by Hertwig in Berlin. It is based on a measurement of the characteristic frequency and the velocity of the transmission of elastic waves through the material. Finally, in the Paper No. B-2, Vol I of the Proceedings of this Conference, J. T. van Bruggen describes a method of securing undisturbed samples of clean sand, which permits the determination of the compressibility of the same in the laboratory. It is one of the tasks of future investigations to find out which one among the different methods is likely to furnish the required results at a minimum amount of time and expenditure.

On the other hand, an adequate description of the properties of beds of clay can only be secured by means of laboratory tests on undisturbed samples, including compression and consolidation tests.

The method B, based on existing records, has the advantage of involving no theoretical conceptions other than those on which the selection of the criteria for identification are based. A consistent application of the method is bound to increase our knowledge of the subject independent of the progress along theoretical lines and in some cases such as those involving the settlement of buildings founded on beds of sand or silt it is the only one which is at present at our disposal. Yet in order to use this method, the practice of adequately investigating the subsoil and of observing the settlement must become far more universal than at present.

By far the most promising field for a successful application of the method B, based on analogy, is to be found in big cities. Although the soil located beneath our cities may vary to a certain extent in type and consistency, these variations are seldom important enough to prevent the existence of relatively simple relations between the load, the size and shape of the loaded area, the type of foundation, the soil profile and the settlement. If adequate settlement records for a large number
of buildings are available, simple identification tests combined with a knowledge of the facts illustrated by Fig. 1 to 5 may suffice for a prediction of the settlement of new structures with a reasonable degree of accuracy. For this reason the careful and continuous supervision of the settlement of newly constructed buildings in our cities represents the simplest and most economical method for eliminating the hazards resulting from our inadequate knowledge of the subject.

Method C. The principal drawback of the preceding method consists in the fact that it cannot be applied to foundations without a known precedent both for the soil conditions and for the principal dimensions of the foundation. Therefore it was considered necessary to work out a third method which should make it possible to compute the settlement from the results of soil tests. In order to accomplish this purpose the theories of settlement developed. In pure science the term theory is used to indicate a conception of the interrelations of a group of natural phenomena, whose validity appears to be well-established and more or less generally accepted. If this definition of a theory is also accepted for the field of foundation engineering it should be emphasized that none of the existing theories deserves this name, because the properties of real soils are far more complicated than the properties of the imaginary materials to which our theories rigorously apply. Hence the term theory is merely retained for the sake of convenience. In reality our theoretical methods only refer to ideal substances whose properties were obtained by disregarding all but one or two of the properties of the real soils. For this reason the theories furnish a radially idealized conception of what should be expected to happen in nature and none of the theories can be accepted for practical usage until we have determined the importance of the error by comparing the computed data with the results of observations on full-sized structures in a great number of cases. Prior to this supplementary investigation any theory of settlement can only be expected to disclose an interesting possibility, regardless of whether or not the theory has been confirmed by small-scale tests.

Deformation and Compression. In order to correlate the existing theories with engineering practice, we divide the settlement S of the structures into two components, S_0 and S_v respectively. The first part, S_0, is solely due to the lateral yield of the loaded soil without any change in the volume. Owing to this lateral yield, the originally vertical lines ab and cd in Fig. 5 assume the shape a'_1 b'_2 and c'_1 d'_2, whereupon the load descends through a distance S_0. The second part, S_v, is due to an increase of the density of the loaded soil. If the density of the soil located within the space a_1 b_2 c_1 d increases, the load further descends through a distance S_v, Fig. 5.

In applied mechanics, the relation between S_0 and S_v is determined by the modulus of lateral expansion, 1/m, of the loaded material. If the material is perfectly incompressible, the value 1/m is equal to 1/2 and the increment S_v of the total settlement is equal to zero. With decreasing values of 1/m, the importance of S_v increases. In applied mechanics it is also assumed that the deformation of the loaded material, including its change in density occurs instantaneously. As a consequence both parts, S_0 and S_v of the settlement are assumed to take place at once, as soon as the load is applied. If such is the case we would have no means for dissolving S with its two components.

On the other hand, if the loaded material is porous, its voids completely filled with water, and its permeability very low, the traditional concept of applied mechanics loses its validity, because it takes a long time for the excess water to drain out of the space a_1 b_2 c_1 d, Fig. 5. In this case the material subjected to a superimposed load would appear to be incompressible and the settlement immediately after the load application would only be equal to S_0. Then, as time goes on, the settlement would gradually increase from S_0 to S = S_0 + S_v. The lower the permeability of the compressible strata the smaller is the rate at which the increment S_v approaches its ultimate value. This statement is also based on adequate evidence.

The Principal Theories of Settlement. The theories which deal with the relation between the physical properties of the loaded material and the settlement are the following:

a. The theory of elasticity which assumes that the loaded material is perfectly elastic, isotropic and homogenous.

b. The generalized theory of elasticity which assumes that the loaded material is perfectly elastic and homogenous, but that the modulus of elasticity is different in different directions.

c. The theory of consolidation.
a. The application of the theory of elasticity to the computation of the settlement of a load was accomplished by Boussinesq in 1885. The equations devised by this investigator also permit the computation of the equation of the curves shown in Fig. 1a and b and the equation for the "bulb of pressure" for any value of $1/m$. A summary of subsequent supplementary theoretical investigations which are also based on the assumption of an elastic, isotropic and homogenous character of the loaded material, can be found in the Paper No. E-10, Vol II of the Proceedings of this Conference. All the formulas for settlement which are based on Boussinesq's theory contain the coefficient of lateral expansion $1/m$.

The value of this coefficient determines the volume change associated with the deformation of the stressed material. A value of $1/m = 1/2$ indicates a volume change equal to zero. If the volume change occurs very slowly, the settlement which takes place while the load is being applied is equal to that corresponding to $1/m = 1/2$ regardless of the value of $1/m$ of the loaded material. This instantaneous settlement corresponds to the fraction $S_0$ in Fig. 5. As time goes on the settlement gradually increases by the fraction $S_v$ in Fig. 5. According to Boussinesq's theory, the fraction $S_v$ cannot exceed an amount of about 0.35$S_0$, provided the loaded area is circular, the modulus of elasticity remains unaffected by the volume change and the value $1/m$ which determines the final value of the settlement is equal to $1/5$, or equal to the smallest value which we are likely to encounter. This information is very valuable because in certain cases it enables us to learn from the results of settlement observations to what extent the loaded material is elastically isotropic.

b. The application of the generalized theory of elasticity to settlement problems was worked out at my suggestion by K. Wolf (Ztschr. für angew. Mathem. und Mech., 1935, H. 5.) It should however be emphasized that the strict validity of this theory is also limited to ideal materials with very simple, imaginary properties. The degree of anisotropy of these ideal materials is expressed by the value of the ratio $E_h/E_v$ between the modulus of elasticity in a horizontal and in a vertical direction respectively. For $E_h/E_v = 1$, Wolf's theory becomes identical with Boussinesq's theory for isotropic, elastic solids. For the case $E_h/E_v = \infty$ the settlement is exclusively due to a compression of the loaded material under complete lateral confinement. On this condition, the fraction $S_0$ of the settlement in Fig. 5 becomes equal to zero.

c. The theory of consolidation is based on the following assumptions: the voids of the loaded material are completely filled with water. Both the water and the solid particles are considered incompressible and the flow of the water through the material is determined by the law of Darcy. The water escapes from the loaded material only in a vertical direction and the increase of the settlement with time is exclusively due to delayed drainage. (K. Terzaghi and O. Fröhlich, Theorie der Setzung von Tonschlöchern, Wien 1936). In addition, several successful attempts were made to solve the equations for an escape of the water in a horizontal direction (G. Gilboy) or in horizontal, radial directions (L. Rendulic).

Other Auxiliary Theories. The theories a to c deal with the relation between the load and the settlement for certain well-defined, ideal materials. There are, in addition, several theories with a more limited scope, dealing only with the distribution of the stresses in the loaded material or with the ultimate bearing capacity. Their validity is also limited to imaginary materials. These theories are:

d. Theory of stress distribution in perfectly elastic materials whose isotropy is limited to horizontal sections. In a vertical direction the modulus of elasticity increases according to some specified law (O. Fröhlich, Druckverteilung im Baugrund. Wien 1934).

e. Theory of the Bingham bodies. This theory is based on the assumption that the material is perfectly isotropic and homogenous and that it starts to flow as soon as the shearing stress $t$ exceeds a certain critical value $t_1$. The rate of flow, $v$, increases according to the straight line law represented by the line AB in Fig. 6. The elastic phenomena are entirely disregarded.

f. Theory of plasticity. This theory differs from the preceding one in that the angle $\alpha$ in Fig. 6 is assumed to be equal to $90^\circ$. The value $t_1$ is supposed to represent some function of the normal pressure on the plane of shear. No solutions are available for cases other than those in which the flow occurs parallel to a plane ($A$, Nadai, Plasticity. New York, 1931).

Sources of Error in Settlement Estimates. All these theories, a to f, apply only to very simple imaginary materials. In real soils the properties ascribed to these ideal materials combine with many others which are ignored by the theories. Foremost among these properties is the capacity to undergo important unelastic deformations which increase not only at a higher rate than the corresponding stresses but which also increase to some extent with time, under constant stress. A further increase of settlement at a constant load can be produced by vibrations which are also disregarded by the theories cited above.
The errors due to simplified assumptions combine with the errors due to erratic stratification of the natural soil strata and to a partial destruction of the original structure of the soil during the operations of drilling, sampling and transporting the samples into the laboratory. Other sources of uncertainty are the impossibility of estimating the average ratio $E_r/E_s$ for natural soil strata, and the absence of reasonably convenient methods for determining the critical shear values $t_1^*$ and $t_2^*$ for clays in Fig. 6.

Considering these facts it would appear utterly out of place to interpret the word "theory" used in connection with settlement in the same fashion as in pure science or in the sections of applied mechanics dealing with steel constructions. Our equipment for solving the problem of settlements consists merely of a few rather crude theoretical methods for estimating the settlement in those special cases in which the properties of the soil are tolerably similar to those of the imaginary materials and the accuracy of the forecast is limited by the degree of precision with which we can determine the numerical values of the vital soil properties. None of the theories has any practical value until we have learned by repeated and varied experience to what extent the theoretical results coincide with the behavior of full-sized structures. Hence no new method can be accepted for practical usage until it is supplemented by a set of empirical data, illustrating the range of possible error.

Present Status of Settlement Computation. At the present state of our knowledge, the prospects of the different theories for practical application are about as follows:

The theories of stress distribution are accurate enough to inform us as to the order of magnitude and the distribution of the normal stresses over horizontal sections at different depths below the base of foundations and for an approximate computation of the contours of the bulb of pressure for any given value of $1/n$ such as shown in Fig. 2. The methods for stress computation assist us in choosing the depth to which the soil for a proposed structure needs to be investigated and in determining the approximate shape of the curves of equal settlement for flexible structures supported by shallow foundations on cohesive soil or by friction piles.

The distribution of the soil reactions over the base of rigid foundations on cohesive soils has at least the general character which one should expect from the theory of elasticity (a). It involves a maximum along the rim and a minimum near the center. The same conclusion seems to hold true for the base of deep foundation piers resting on sand. For rigid slabs resting on the surface of a bed of sand, the distribution of the pressure is approximately parabolic, with a maximum at the center and zero at the rim. Nothing is known about the distribution of the soil reactions over the base of rafts located at a medium depth, ranging between 5 and 20 ft. below the surface of sand beds. Extrapolation from small-scale tests to full-sized structures can be misleading.

The prospects of a successful forecast of the magnitude of the settlement by semi-theoretical methods are different for different types of soil. For sands the fraction $S_1^*$ of the settlement $S$ (Fig. 5) is much more important than the fraction $S_0^*$. The value of $S_0^*$ is determined essentially by the non-elastic deformation of the sand which is beyond the scope of existing theories. Therefore the only feasible method of dealing with the problem consists in utilizing existing settlement records such as those described above.

If the settlement is essentially due to the compression of well-defined layers of clay located between beds of sand or other very permeable materials, the theory of elasticity combined with the theory of consolidation was found to give promising results. The agreement between the theoretical and the real curves of equal settlement is always satisfactory provided the loaded strata are fairly uniform in a horizontal direction. The Paper No. E-1, Vol I in the Proceedings contains instructive examples. The agreement between the theoretical and the real time rate of consolidation is also very good. According to the theory of consolidation the application of the load produces in the water content of the clay beds a hydrostatic excess pressure equal to the total pressure created by the super-imposed load. As time goes on the excess water escapes from the clay into the adjoining permeable strata, while the hydrostatic excess pressure gradually decreases and approaches the value zero. This conception is fully corroborated by the results of observations in the field such as those communicated in the Paper No. F-9, Vol I. However the absolute values may be very much smaller than those computed from the results of consolidation tests on undisturbed samples. According to Paper No. D-1, Vol I, the ratio between the observed and the computed settlements of buildings located above strata of silt and clay in Cairo is as follows:

- Silty soils 1/2 to 1/3
- Stiff brown clays 1/2
- Black clays 1
- For certain glacial "hardpan" in Germany and for the stiff blue Tertiary clay underlying the city of Vienna, the value of the ratio is even smaller than 1/3.

On the other hand, for the Zanesville dams described in a paper read by Dr. Gilboy, or in some of the cases investigated by myself, the observed settlement was almost identical with the computed value. From what little we know at present about this subject it appears that the important differences between theory and reality are limited to those cases in which the loaded material was intensely pre-compressed at some stage of its geological history. Hence a knowledge of the load history, expressed by the shape of the compression curve of undisturbed samples may assist in establishing empirical rules for correcting the error.

If the clay contains numerous streaks of silt or thin layers of sand the settlement of a super-imposed structure is essentially due to a compression of the clay under almost complete lateral con-
finement. At the same time the escape of the excess water occurs very rapidly. The Paper K-2, Vol I, contains an instructive example.

Very thick beds of silt or clay should be expected to exhibit characteristics intermediate between those of semi-infinite, elastic isotropic solids and those of sands. For the elastic solids the fraction \( S/S_0 \), Fig. 5, of the settlement does not exceed about 30% of the instantaneous settlement \( S_0 \) and for sands the fraction \( S_r \) is even smaller. In contrast to these findings, the gradual settlement \( S_g \) of structures supported by raft foundation on thick beds of silt or clay is almost always many times greater than the instantaneous settlement \( S_0 \) which develops while the load is being applied. On the other hand, the settlement of small individual or narrow continuous footings, spaced far apart do not exhibit this characteristic property. These facts seem to indicate the existence of a great resistance against the lateral yield of the strata located at a depth of more than 8 or 10 feet below the surface, which practically eliminates in some cases the fraction \( S_g \) of the settlement \( S \) in Fig. 5 for buildings supported by a raft foundation. The foundation of complete confidence in obtaining a complete settlement record is still too small to permit a final judgment on the physical causes of this wide-spread phenomenon.

The settlement due to the compression of thick beds of silt or clay often continues for a long period at a higher rate than one would expect on the basis of the theory of consolidation. Similar phenomena are also found to occur in connection with consolidation tests on completely confined specimens of undisturbed clay (see A. S. K. Buisman in F-7, Vol I, and N. Gray in D-11, Vol II). Another possible cause for these progressive movements consists in a very slow, plastic flow under constant stress. Whatever the cause of the causes may be, the difference between the theoretical and the real time-rate of settlement is seldom great enough to be of any practical importance.

Conclusions. The above discussion reveals the following situation:

1. Every structure is bound to settle during and after construction. For flexible structures, including buildings of any type, the settlement is always unequal although the load may be uniformly distributed over the surface of a uniform stratum. In some cases the settlement increases for years or decades at a practically constant load and approaches values which are many times greater than the settlement produced by the process of applying the load.

2. All the theoretical and experimental investigations into the state of stress which exists beneath a loaded area have disclosed the fact that the settlement depends on the physical properties of all the strata located between the loaded area and a depth \( T \) beneath this area, which is at least 1.5 times the width of the loaded area. This conclusion is sustained by adequate evidence. It precludes the possibility of predicting the settlement from the results of small-scale loading tests. The only exceptions are those rare cases in which the soil is homogenous to the depth \( T \).

3. The physical properties of the soils are by far too complicated to permit an estimate of the settlement on a purely theoretical basis. Hence the foremost requirement for future progress consists in collecting complete settlement records, including the curves of equal settlement, the time-load and the time-settlement diagrams, and test-boring records which disclose the nature of the soil to a depth which is at least equal to the value \( T \) mentioned above. No procedure derived from theoretical or experimental investigations should be accepted for practical use, unless the degree of accuracy of the results has been determined previously by checking the results against the actual settlement of an adequate number of full-sized structures.

4. The investigations of the physical properties of the soils disclosed the universal fact that the results of a visual inspection of drill samples or of an experimental investigation of disturbed drill samples does not suffice for a reliable identification of soil strata. As a consequence, test-boring records which are based solely on such data leave an intolerably wide margin for interpretation. Reliable identification can only be achieved by means of numerical data obtained from tests on the soil in an undisturbed state.

5. The prospects for a successful theoretical computation of settlement are for the time being limited to those cases in which the settlement is due essentially to the compression of well-defined, fairly homogenous beds of clay whose excess water escapes in a nearly vertical direction. By far the majority of harmful, progressive settlements is due to this cause. Hence it is a fortunate coinci
dence that the prospects for successful theoretical treatment exist precisely in those cases in which they are most urgently needed.

6. According to the above statements, the most outstanding achievements of scientific approach to the settlement problem are the following: analysis disclosed the utter futility of the traditional attempts to forecast the settlement from the results of small-scale loading tests by means of simple, standardized rules valid for a wide range of conditions. It informed us on the depth to which the properties of softer strata are likely to influence the settlement. It furnished basic information concerning the requirements for a reliable soil investigation, which opens the way to a semi-empirical treatment of the subject. It provided us with rules for preparing settlement records in such fashion that the records can be used as a basis for an estimate of the settlement of future structures. And finally it accomplished a promising start towards developing a method for computing the settlement in certain special cases of outstanding practical importance.

Considered in their totality the accomplishments cited are most encouraging. The painstaking analysis of the settlement phenomena accomplished the replacement of vague and misleading conceptions by positive though fragmentary knowledge. It has provided us with a clean-cut program for further research in this field and disclosed the limits for the validity of theoretical conclusions. In this connection it demonstrated beyond any doubt that no progress can be expected without a systematic accumulation of adequate settlement records.
Whoever expects from soil mechanics a set of simple, hard-and-fast rules for settlement computation will be deeply disappointed. He might as well expect a simple rule for constructing a geological profile from a single test-boring record. The nature of the problem precludes the possibility of establishing such rules. If a supervising or a construction engineer wants to enjoy the benefits of recent developments in this field he should first of all study the rules for securing reliable settlement records and then start to observe the buildings of his district. After he has done this for a certain period he will discover for himself the value of the information which he can obtain from soil mechanics.

No. F-17

DISCUSSION

INTERPRETATION OF LOADING TESTS FOR FOOTINGS

Donald M. Burmister, Instructor in Civil Engineering, Columbia University, New York City

In a great many cases there seems to be a large discrepancy between the results of a small-scale loading test and the experience with the full-size structure. Although loading tests are often required by building codes for design purposes, the load-area of footing-settlement relations have not yet been so completely defined that the results of such tests can be used to make a reliable estimate of the settlement of the structure. Very often little is known about the underground, yet in order to obtain any reliable information, the underground should be a deep, fairly uniform deposit of granular or somewhat cohesive soil, which does not contain saturated layers of fine silt or clay capable of consolidating under the building load.

Typical loading test curves are illustrated in Fig. 1a and 1b on various sizes of bearing areas.

The loading test itself serves as an Integrating Factor and reflects the combined influence of all the soil properties and conditions at the site, but of course only to a rather limited depth, as illustrated in Fig. 2.

Most of the settlement results from two causes, first from a compression of the soil with in, for example, the 0.1p pressure isobar. Professor Terzaghi has pointed out that the depth of this isobar of pressure bears a constant ratio to the width of the loaded area. For circular areas r/z is approximately
equal to \( \mu \). Second, from a lateral bulging and finally displacement of material from under the edges of the footing where the shear stress is greatest with a tendency for upheaval of the soil around the footing. The first becomes more important as the size of the footing increases. The second is more pronounced for small-size footings. The resistance of the underground is considered by many to increase with depth. Therefore, the supporting capacity of the soil varies with the size of the loaded area in some way as determined by these factors.

It seems to be of greater importance to obtain an integrated soil constant from a loading test, expressing the load-area-settlement relations, than to define it in terms of specific characteristics and behavior of the soil. These relations are obtained by the use of a modified elastic equation for the center deflection of a loaded circular bearing area. The elastic equation, which is given in Timoshenko's Theory of Elasticity, page 335, Eq. 204, is equal to

\[
S = \frac{2\left(1 - \frac{1}{m}\right)^2}{E} \cdot p \cdot r
\]

where \( \frac{1}{m} \) is Poisson's ratio

- \( E \) is the Modulus of Elasticity
- \( p \) is the load per unit area
- \( r \) is the radius of the circular area

This equation is modified by replacing the factor containing the modulus of elasticity and Poisson's ratio by an empirical expression varying with depth.

The settlement equation then becomes equal to

\[
S = \frac{2\pi r}{C(1 + az)}
\]

Where \( C \) and \( a \) are empirical coefficients which depend on the soil properties and conditions at the site and \( z \) is taken, for example, as the mid-depth of the 0.1p pressure isobar, that is \( z \) is equal to the width or the diameter of the footing.

To define the relations requires that loading tests be made on at least two different size bearing areas, so that the two constants may be evaluated. One test should be made on an area as large as practicable in order that it may serve as an integrating factor to greater depths. The smallest test should be at least 2 feet by 2 feet to eliminate excessive lateral soil displacement.

The coefficients are determined for the curves in Fig. 1a and 1b. A limiting maximum unit load is chosen within which the load settlement relations do not deviate greatly from a straight line. For example, in Fig. 1a computation at a unit load of 1.4 tons per sq ft yields values of \( C = 3.6 \) and \( a = +0.13 \), while in Fig. 1b at a unit load of 2.5 tons per sq ft \( C = 5.2 \) and \( a = -0.029 \). The character of the soil and the conditions at the site determine the sign of the coefficient "a". It is probable that the break in the load-settlement curves for large areas will occur at lower loads than for the test curves. This important point cannot be determined by small-scale tests. But Professor Terzaghi has pointed out that footings should be designed to limit the settlement to some safe total amount or to limit the differential settlement to, for example, 1/8 inch between adjacent footings in order not to induce excessive secondary stresses in the structure.

It seems logical to conclude, therefore, that, when the coefficient "a" is positive, a fairly reliable estimate of the settlement of a structure is possible from a loading test. The settlement for a constant unit loading may approach a value, which independent of the size of the loaded area. On the other hand when the coefficient "a" is negative, no loading test on small areas can give any reliable information about the behavior of the structure itself.

The settlement-area relations for a constant unit loading of 2 tons per square foot are given in Fig. 3. It is evident that two quite different soil conditions exist.

---

**Fig. 3 Settlement-Area Relations**

For a Constant Unit Loading of 2 Tons per sq. ft.
ADDITIONAL EXPERIENCE ON SITU-CAST CONCRETE PILES

Dr. Rudolf Tillmann, Ge.I.A.V., Bldg. Dept. of the City of Vienna, Austria

Situ-cast piles, formed in ground by a special machine fitted with heavy drop hammer and concrete tampers, were invented and originally used in Austria on a larger scale especially in municipal tenement house building of Vienna. In this connection may be reported on a failure occurring about 10 years ago in the foundation of such a very large house, in which the aforementioned type of piles was used. This negative experience shall be explained here in order to warn foundation engineers that when adopting this type of piling special care on soil investigation has to be taken.

The underground was composed of yellow silt and loam for about 3 meters below the surface, then of pure sand and gravel. Between these two strata there was inserted a relatively thin layer of dark and soft mud (0 to 1 m thick) on a considerable part of the area of the large building site in question. The foundation was designed so that concrete rafts had to be born by situ-cast concrete piles resting on resistant gravel. The conical pile forms were driven down, then lifted, and subsequently the pile holes were filled up with plastic concrete worked by ramming. Unfortunately the consequence of equilibrium disturbance, caused by piercing the aforementioned soft mud layer had not been taken into account. This mud bulged out towards the inside of the hollow space of the piles, so locally diminishing the pile cross-section. This not having been observed, concrete, after having been poured in, had not the statically claimed form and piles with a very restricted middle section cracked under load and caused enormous settlements of a large portion of the 5 storied tenement house during building of the superstructure. Settlements reached in some spots 300 mm and were of course unequal for the middle and outer walls, so that the superstructure got wide cracks not only in transverse but also in a longitudinal direction, especially in the cantilever ends of all reinforced concrete ceilings and in partition walls. Besides may be mentioned, that the deformation of ceilings due to unequal settling of the walls was greatest in the lowest floor and decreased in an upward direction.

Reconstruction of this failure was very expensive and was performed upon expert advice by underpinning the supporting walls by reinforced concrete footings each of them resting on a pair of foundation walls, based on the gravel underground. After this thorough reconstruction of the foundation of the damaged building parts, also the superstructure was repaired, so that no further damage might occur.

The conclusion which may be drawn from this experience shortly presents itself as follows: In the case of adoption of conical situ-cast piles, when soft layers are interferring, never pull out conical metal shells from the hollow forms of piles. Earth forms alone may satisfy only, when stiff material stands along pile length. In this case pouring of rather stiff concrete and thorough ramming of it—as also stated by W. S. Hanna and G. Tchebotareff (Paper No. F-1, Vol I) has proved to be advantageous.

DISCUSSION OF PAPER NO. F-2

Dr. Rudolf Tillmann, Ge.I.A.V., Bldg. Dept. of the City of Vienna, Austria

Similar considerations as those on earthquakes may be adopted in the study of vibrations of buildings caused by technical effects, as e.g., periodical impulses of driven machines or vehicles of common traffic. The Municipal Building Department of Vienna is fairly experienced in this line. Here many seismographical observations were made on vibrations caused either by industry or by traffic. In a majority of cases technical vibrations of buildings do not damage the superstructure itself, even when machines are installed in an upper floor, because of the high factor of safety allowed in these structures (5 to 5 and even more). A contrary statement must be made with respect to the foundation, which generally has a factor of safety of not more than 2 in many cases even less than 1.5. Therefore failures in the superstructure due to technical vibration effects—as well as to earthquakes—are initiated by exceeding the soil resistance. For this reason investigation of a mechanical vibration effect of a building is for most cases in first line a foundation or soil problem.

After seismographical observation seismograms are thoroughly investigated. The parts of these time-deviation curves showing the straightest vibration periods and the highest amplitudes, are selected for mathematical analysis. Generally the most important figure in this computations is the maximum value of acceleration in vibrating movement, taken for each of the 3 cardinal directions of space.

This acceleration for a direction x is to be expressed by $g = l_n^2 \omega_x^2 A_x n_x^2 = A_x \omega_x^2$ $A_x$ being the amplitude, $n_x$ the frequency in Hertz and $\omega_x = 2 \pi n_x$ the "circular frequency" of the vibration component in the aforementioned direction. Then the reach of simultaneous effect on a structure by a certain vibration has to be confined by means of computing the propagation of the elastic wave in all directions within the structure during half a vibration period. The mass of the structure within this reach may be termed by $M_x$ so that the maximum power exerted by the vibration in question is determined by $P = M_x g_0 \delta$. This force is supposed to act in the centre of gravity of the mass $M_x$. In the foregoing term $\delta$ is a coefficient greater than 1 by which the additional effect of the individual frequency of the structure is to be taken into account. With respect to the circumstance, that technical vibrations of buildings are usually of long duration, the fatigue of building material and
especially of soil must be taken into account. According to experience one may do this by introducing the double value of ordinarily computed stresses.

For most cases magnitude of technical vibrations has been found to be characterized by the following figures:

\[
A = 5 \text{ to } 30 \mu \\
\nu = 5 \text{ to } 100 \text{ Hz} \\
3 = 0.1 \text{ to } 5 \text{ m/s}^2
\]

(in most cases not more than 1 \text{ m/s}^2)

By vibration effects, especially when caused by horizontal movements, edge pressures of soil may be increased from 20 to 100 and sometimes even for 150% relative to soil stresses, caused by static load (dead load + live load). From the foregoing remarks there may be recognized, that technical vibrations may be of considerable magnitude for buildings, not less than slight earthquakes. Vibration may be counteracted by the springing effect of insulating materials such as cork or rubber and above all by steel springs. By such insulation original power effect of vibration is reduced to a certain percentage being determined generally by

\[
\varepsilon = 100 \sqrt{\frac{1 + \theta}{\omega^2 - \omega_0^2 + \theta}}
\]

wherein \( f \) = coefficient of springing effect (cm/kg-1),
\( \theta \) = coefficient of internal friction.

Adopted for the judgment of earthquake effects, the upper formula shows that power effect of vibration is reduced the more, the greater springing effect of insulating layer may grow. Each elastically working sublayer, as e.g. also foundation soil, especially when it is of clay nature, has such a springing effect with the coefficient

\[
f = \frac{1}{F_{soil}} (F = \text{foundation area in cm}^2, C = \text{coefficient of compressibility in kg/cm}^3)
\]

This springing effect of the soil enables the engineer to reduce the amount of acting forces to be introduced into the statical computation of the structure and so may lead to considerable savings.

In this connection there may be reported an interesting case which Vienna's Municipal Building Department had to deal with. A new municipal tenement house had been in course of construction. Since the soil consisted of a fairly thick layer of fill the foundation was designed with reinforced concrete rafts, resting on precast reinforced concrete piles. During construction of the superstructure walls settled not much, but to an unequal extent. While in the northwest-corner of the building settlements were only 1 to 5 mm, at the same time walls on the opposite side showed settlements of about 8 to 12 mm. One guessed at once that the cause of this differential settlement was to be found in the vibrations exerted by an electrically driven concrete and mortar mixing plant, installed before the front wall, just beside those parts of it, which had showed a considerable tendency to settle.

After having observed this settling tendency seismographic investigation was done in order to ascertain the degree of vibration effect. It was found thereby, that by both vertical and horizontal vibrations the mixing plant mentioned caused an additional soil pressure at the raft edge of the front wall foundation to an extent of 6 to 10 per cent of the statical stress. In spite of this relatively moderate increase of soil pressure the troubling effect here in question had been exerted, because the vibration had a fairly high frequency (20 Hz) and long duration and since the loose fill materials of the subsoil are more susceptible against dynamic actions than natural earth deposits. After the aforementioned mixing plant had been stopped, no further settling of the building has yet been observed.

In Fig. 1 you see an example of model loading tests made in our laboratory by Professor Hertwig. In sand failure occurs when the stress exceeds the friction along the curved surface called "Gleitfläche". We usually sketch a cross-section of such tests which are made with a rectangular footing. When, however, the loaded area is a square, the ultimate load before failure will be much larger, because sliding will take place in all four directions. With sand of the same pore space and compaction the load at failure on a square bearing plate will be approximately double the load which a long rectangular strip of equal area would carry.

Referring to the remarks of Professor Cuevas I hope that some examples will interest you because they refer to our practice. If I had known that there was to be a kind of competition (or a Derby race with heavy horses) on "dirty" soils, then I would have brought more pictures with me and perhaps won the prize. The soils in Java and Sumatra are very bad, since the rainfall there is about six to ten meters a year and much soil is carried by erosion into the streams. In these regions harbors are constructed on soil where ten years ago there were only mangrove trees growing in salt water. These soft soils consolidate slowly, of course, and even light tanks with 0.5 kg/cm² load had a settlement of fifty centimeters during two and a half years. I am sorry that I did not get more settlement records from there. And I hope that our Dutch friends will continue the observations so that we may
get the settlements in years to come.

A part of the harbor of Belawan (east coast of Sumatra is built on mud like that described above. In Fig. 2 you can see the coast line and mangroves, in the foreground a quay wall made of steel sheet piling, the backside of which is later to be filled up with sand. Cracks in the top beam are shown in Fig. 3 and the deflection of this beam is seen in Fig. 4. The screw bolts are the ends of anchor rods. The anchor plates were not far enough from the foreside of the retaining wall, so when the fill consolidated, the steel sheets were loaded by the so-called "negative skin friction", and put into the hard layer with some inches more. The bearing soil was gravel with sand and corang (coral-limestone). It is porous, but altogether it is a very good combination of subsoil. You can see it on a cross-

section Fig. 5. Below this hard layer, though, is another layer of clay about 200 meters deep.

At this particular site piles were not used for the several tanks and a pumping station, because the piles for one tank would have cost more than all of the steel work. The height of these tanks was only about five meters and the load was 0.5 kg/cm². They are filled with palm oil which is produced there. It is stored in these tanks and then transferred to the ships.

On the cross-section (Fig. 5) you will see underneath the
building a small offerdam and an excavation filled up with sand. The sand was necessary because in that mud we could only pour a concrete slab after a layer of sand had first been put down.

Fig. 6 shows the settling of four points on Tank IV together with the load of palm oil which the tank carried. When this tank was test loaded with water it settled only 6 cm. Then the owner discontinued settlement observations until the first cracks were noticed in the building. Then the level observations were resumed. The curves show a continuous settlement through two and one-half years. Later the tank rises when empty. The rise of points c and d is small because other tanks adjoining this side (I and V) are still loaded. Notice on Fig. 5 that the amount of settlement in the building (points a, b, c, and h) has been reduced by the excavation.

No. F-21

DISCUSSION OF PAPERS NO. F-12 AND F-13 (By Letter)
ON SETTLEMENT OF STRUCTURES IN SHANGHAI, CHINA
Dr. Karl v. Terzaghi, Professor at the Technische Hochschule, Vienna, Austria

These papers contain very valuable contributions to our scanty knowledge of the settlement of "floating pile foundations". Paper No. F-12, Vol II, also contains, in Appendix 2, a set of proposed rules for the computation of beams in buildings supported by foundations of this type. The value of F-13, Vol II, could be greatly increased by a supplement containing a section through the old Lokawai power station, a plan showing the relative position of the power units and a set of time-settlement curves. The following discussion is an attempt to correlate the results of the observations with our general knowledge of the subject and to find out what supplementary information would be required to
make an adequate adaptation of the Shanghai building code to the characteristics of floating pile foundations.

Soil conditions and soil pressures. According to Paper No. I-I, Vol II, the subsoil of Shanghai consists to a depth of about 900 ft of silt with a variable admixture of fine sand and clay. Owing to the lenticular structure of the deposit the compressibility is likely to change from point to point in a rather erratic fashion. This lack of uniformity is reflected in the settlement characteristics of the observed buildings. Fig. la shows a typical, simplified cross-section through the buildings described in the papers. The silt within the prismatic space a b c d located beneath the base of the building is surrounded and reinforced with round, tapered Oregon pine piles. The distribution of the vertical normal pressure over a horizontal plane AB, located at the points of the piles which are at a depth T below the surface is shown by the ordinates of the dotted curve eB, Fig. la. The total area enclosed between AB and eB is equal to the difference between the normal pressure which acted on AB before and after the building was constructed. In order to simplify the pressure distribution we replace the wedge-shaped area AeB by a rectangle Aefg which has the same height, Ae, and the same area. The height Ae = \( w_0 \) of the rectangle is called the effective normal pressure on the horizontal section AB. The width Ag of the rectangle Aefg in Fig. 1a is determined by the condition

\[
\text{area AeB} = \text{area Aefg}
\]

It is equal to half the width, b, of the prism plus a certain fraction, nT, of the depth T

\[
Ag = b + nT
\]

For the buildings described in the Paper No. F-12, the depth T is approximately 16 ft (more or less). If the piles were non-existent and if the loaded material were perfectly elastic and isotropic, the width Ag, Fig. 1a, could be computed by means of Boussinesq's theory. It would be approximately equal to

\[
b + 1.1T \quad \text{or} \quad n = 1.1.
\]

For a pile foundation in a natural, more or less stratified soil the value of n is not known. The stratification tends to decrease the width Ag over which the load spreads. The extreme allowance which one could make for this effect would be to replace n = 1.1 by n = 0.25, a value less than one-quarter of that for an elastic, isotropic medium. An upper limit for the relieving effect of side friction is given by the condition that the shearing stresses along ac and bd cannot exceed the shearing resistance of the soil. For the Shanghai soils the skin friction on piles is about 150 lbs/sq ft. Hence the shearing resistance of the soil is at least as high as that.

If we assume a value of n = 0.25, the total area F over which the weight of the building is distributed at a depth T below the surface is equal to the sum of the area A covered by the building and the area \( A_1 \) of a belt with a width of 0.25 T as shown in Fig. 1b.

The difference between the load carried by the soil before and after construction is equal to the difference between the effective weight of the building and the weight of the excavated soil. According to the Paper No. F-12 the term "effective weight" of the building means the sum of the dead load and that part of the Code live load which can be expected to act under normal service conditions. The weight of the excavated soil is in Shanghai approximately equal to 100 lbs per cu ft. Let

\[
w_o \quad \text{be the effective weight of the building per unit of the area A covered by the building,}
\]

\[
w_o' = w_o - 100 T_j \quad \text{the effective unit load reduced by the weight of the excavated soil,}
\]

\[
w_o'' = \text{the increase of the normal pressure on the section AB, Fig. la, per unit of the area F, produced by the construction of the building,}
\]

\[
L = \text{the perimeter of the area A in Fig. 1b and}
\]

\[
t = \text{the average shearing stress along the vertical sides ac and bd in Fig. 1a.}
\]

Using these symbols we obtain by a simple calculation

\[
w_o'' = w_o' - \frac{A}{F} = \frac{A}{F} (w_o - 100 T_j)
\]

and

\[
t = (w_o' - w_o'') \frac{A}{LT}
\]

Table I contains these values for the nine buildings which are described in the Paper No. F-12. It also contains the increase S of the settlement over a period of 2 years starting about 1 year after construction was finished, and the difference \( \Delta S \) between the greatest and the smallest value of S. The value t in the last column gives the shearing stress per unit of area of the surface of the piles on the assumption that the whole effective load is carried by the piles. The greatest value the skin friction can assume is about 150 lbs/sq ft.

Table II contains the data regarding these two units. The value S represents the total settlement after a period of 4 years. The table shows plainly that the settlement depends essentially on the value of \( w_o'' \). Since both units are located within the same building it seems very likely that the difference between their settlement is essentially due to the difference in the length of the piles.
TABLE I

<table>
<thead>
<tr>
<th>Bldg</th>
<th>A sq ft</th>
<th>L ft</th>
<th>T ft</th>
<th>100 T lbs/sq ft</th>
<th>W_e lbs/sq ft</th>
<th>W_e' lbs/sq ft</th>
<th>w lbs/sq ft</th>
<th>t lbs/sq ft</th>
<th>S in</th>
<th>ΔS in</th>
<th>t_1 lbs/sq ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Victoria nurses</td>
<td>13285</td>
<td>712</td>
<td>60</td>
<td>--</td>
<td>2200</td>
<td>1220</td>
<td>300</td>
<td>1.7</td>
<td>2.9</td>
<td>1.2</td>
<td>272</td>
</tr>
<tr>
<td>2 C.R. Police quarters</td>
<td>15780</td>
<td>316</td>
<td>12</td>
<td>2200</td>
<td>1550</td>
<td>900</td>
<td>225</td>
<td>0.6</td>
<td>1.5</td>
<td>0.9</td>
<td>287</td>
</tr>
<tr>
<td>3 Indian Warders</td>
<td>12628</td>
<td>50</td>
<td>1000</td>
<td>1670</td>
<td>725</td>
<td>250</td>
<td>990</td>
<td>1.0</td>
<td>2.4</td>
<td>1.0</td>
<td>292</td>
</tr>
<tr>
<td>4 Juvenile Block</td>
<td>7550</td>
<td>429</td>
<td>50</td>
<td>--</td>
<td>1166</td>
<td>100</td>
<td>100</td>
<td>2.1</td>
<td>2.8</td>
<td>0.7</td>
<td>725(1)</td>
</tr>
<tr>
<td>5 Cell Block R.S.</td>
<td>11500</td>
<td>60</td>
<td>1000</td>
<td>1520</td>
<td>1152</td>
<td>106</td>
<td>13-1.8</td>
<td>0.5</td>
<td>486(2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Police hospital</td>
<td>7340</td>
<td>505</td>
<td>50</td>
<td>1920</td>
<td>975</td>
<td>350</td>
<td>975</td>
<td>0.8</td>
<td>1.9</td>
<td>1.1</td>
<td>468</td>
</tr>
<tr>
<td>7 Gaol</td>
<td>6155</td>
<td>465</td>
<td>50</td>
<td>--</td>
<td>2312</td>
<td>280</td>
<td>105</td>
<td>1.9</td>
<td>3.0</td>
<td>1.1</td>
<td>287</td>
</tr>
<tr>
<td>8 Admin. block</td>
<td>7070</td>
<td>484</td>
<td>70</td>
<td>1020</td>
<td>1400</td>
<td>150</td>
<td>150</td>
<td>1.0</td>
<td>2.2</td>
<td>0.8</td>
<td>223</td>
</tr>
<tr>
<td>9 West S.R. Firo st.</td>
<td>10680</td>
<td>526</td>
<td>80</td>
<td>--</td>
<td>1584</td>
<td>830</td>
<td>170</td>
<td>0.7</td>
<td>1.0</td>
<td>0.3</td>
<td>308</td>
</tr>
</tbody>
</table>

(1) Difference 725-450 taken up by footings, 867 lbs/sq. ft.
(2) Difference 761-450 " " " " 956 " " " "

The buildings 3 to 8 are located within short distances from each other.

TABLE II

<table>
<thead>
<tr>
<th>Power Units</th>
<th>A sq ft</th>
<th>L ft</th>
<th>T ft</th>
<th>W_e lbs/sq ft</th>
<th>W_e' lbs/sq ft</th>
<th>W_e&quot; lbs/sq ft</th>
<th>t lbs/sq ft</th>
<th>S in</th>
<th>ΔS in</th>
<th>t_1 lbs/sq ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 2750</td>
<td>ca220</td>
<td>28</td>
<td>2900</td>
<td>0</td>
<td>1650</td>
<td>130</td>
<td>10</td>
<td>125</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 2750</td>
<td>ca220</td>
<td>75</td>
<td>2900</td>
<td>0</td>
<td>870</td>
<td>80</td>
<td>5</td>
<td>133</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Soil pressure and settlement. Fig. 2 shows the relation between the value W_e (abscissae) and the two-year settlement S (ordinates), for the buildings 1-3 and 6-9 in Table II. For all the buildings, with the exception of No. 3, the value S increases in simple proportion to W_e and it ranges between 0.8" to 2.9" per 1000 lbs/sq ft and for a period of two years. The scattering from the average can be due to local variations either of the compressibility or of the permeability of the soil. Hence a slow time rate of settlement involves by no means a low ultimate value.

Since the settlement increases over a long period, the ultimate settlement is likely to exceed 12" for each 1000 lbs/sq ft. According to Appendix 1 of the Paper No. F-12 the admissible load for shallow foundations in Shanghai is equal to 1000 lbs/sq ft for dead load alone and to 1500 lbs/sq ft for dead plus total code live load. It is hardly conceivable that the total settlement of shallow raft foundations, exerting a pressure of 1000 or 1200 lbs/sq ft on the soil should on an average be greater than the 12" in produced by such pressures when acting at a depth of 50 or 60 feet below the surface. Otherwise the soil pressures of 1000 or 1500 lbs/sq ft would not be considered admissible. Hence it seems that the settlement in Shanghai depends essentially on the value of W_e regardless of the depth at which this pressure acts. Yet, for the same final settlement the time rate of settlement of a shallow foundation is by no means identical with that of a pile foundation. In general one should expect that the time rate of settlement for a shallow foundation should be greater than for a pile foundation, although the final result may be the same. Since the papers do not contain any settlement records for simple raft foundation, this conjecture cannot yet be verified.

One of the remarkable phenomena revealed by the settlement observations consists in the conspicuous absence of a bowl-shaped depression in the central part of the loaded area. The absence of this feature is particularly noticeable in Figs. 21 and 22 of the Paper No. F-12 which show the settlement of the central and of the peripheral parts respectively, of the Abattoir. A simple raft foundation
always assumes the shape of a shallow bowl, although the load may be perfectly uniformly distributed. The absence of the central depression seems to indicate that the prismatic body of reinforced soil located beneath the building is stiff enough to counteract, at least during the first years after construction, the tendency of the central part to settle more than the peripheral parts. The differential settlement seems to be almost exclusively due to local variations in the compressibility of the soil. Yet here again the above statements should be accepted with reserve, because the settlement observations do not include the settlement during construction. They do not inform us as to the subsidence in the vicinity of the buildings and they contain no data regarding the settlement of points located in the interior of the buildings. It is by no means impossible that the loaded surface assumed the shape of a bowl during the period of construction, which is not covered by the observations.

Another, rather conspicuous feature of the settlement records in Paper No. F-12 consists in the irregular shape of some of the time-settlement curves. The diagrams shown in Figs. 21 and 22 in Paper No. F-12 may serve as an example. The curve for point 12 in Fig. 21 is perfectly smooth, while that for point 8 in Fig. 18 consists of almost horizontal and of steep sections. In Fig. 20 all the settlement curves consist of a series of steps. Particularly disconcerting is the striking contrast between the settlement diagrams of Fig. 15 for building No. 4, and those of Fig. 18 for building No. 5, because the foundations of these two structures are practically identical and the distance between the two structures is less than 300 ft.

The causes of a very irregular time-settlement curve may reside in errors of observation, in an intermittent movement of the bench mark or in an intermittent movement of the observation point. In order to assign the irregular shape of curves in the Paper No. F-12 to one of these three causes, supplementary investigations would be needed.

Seat of the settlement. The weight of buildings Nos. 1-3 and 6-9 in Table I is exclusively carried by the skin friction of the piles. Owing to this fact the gradual increase of the settlement is almost exclusively due to progressive consolidation which begins at the level AB, Fig. 1a, and proceeds from this level both in an upward and in a downward direction. In this connection it would be very important to know whether the major part of the settlement has its seat above or below the plane AB. If it occurs above this plane, the final value of the settlement could be reduced by increasing the number of the piles, provided the beneficial effect of increasing the number of piles is not partially or wholly eliminated by the disturbing effect of the supplementary piles on the structure of the soil. Finally a knowledge of the seat of the settlement would permit a decision as to whether the footings or a raft can be expected to carry any load in excess of the maximum load which can be carried by the skin friction of the piles (in Shanghai soils about 500 lbs per sq ft of the surface of the piles). The knowledge of the seat of settlement can only be obtained by means of underground observation points, to be established at the bottom of drill holes. In order to get the desired information the points should be established at various depths below and above the level AB, as indicated by small circles in Fig. 1a. Reference points of this type were used for somewhat different purposes during the construction of the storage dam "Swir 3" (Paper No. N-5, Vol I) and in Mexico City (Paper No. N-5, Vol I). See also the discussion of the Paper N-5 and the writer's paper "Tragfähigkeit der Pfahlgründungen", Die Bautechnik, 1930, Heft 31 and 32.

Effect of the settlement on adjacent buildings. The practical importance of the settlement consists essentially in its effects on the adjacent buildings and on the structure which is supported by the foundation. The loads which were assigned to the piles beneath building No. 1 (Victoria Nurses' Home in Paper No. F-12) is very conservative. Yet there is little doubt that the settlement will gradually increase to a value of more than 12 in. Hence the strip of ground which surrounds the building will gradually sink to a depth of more than 12 in below its original position. If this strip is occupied by existing structures, the influence of the settlement on these structures may be far more detrimental than its influence on the structure whose weight causes the subsidence, because within the loaded area, the differential settlement will hardly exceed 2 in. (See Paper No. F-12). Yet in the proposed revision of the S.M.C. Building rules in Appendix 2 of the Paper No. F-12, this important and very disagreeable by-product of the settlement phenomena is not considered. In order to get some information to start with it would be desirable to extend the settlement observations over the vicinity of the subsiding structures. For the time being very little seems to be known regarding the width of the area which is affected by a settling pile foundation and it is by no means impossible that some of the bench marks are located within this area. As time goes on, the width of the affected area may increase.

Effect of the settlement on the subsiding structure. If the terms "allowable soil pressure" or "allowable load" have any tangible meaning at all it is the following. If the pressure exerted by the structure is equal to or smaller than the allowable value the settlement produced by the pressure has no harmful effect on the building. Yet even this statement is rather vague, involving an intolerably wide margin for interpretation, unless we specify what constitutes a harmful effect. From general experience regarding the effect of settlement we draw the following conclusions. Plain brick buildings can stand for more important differential settlements than those described in the Paper No. F-12 without showing any signs of being overstressed. Continuous reinforced concrete beams with a height of about 2 ft will crack above the supports. Yet the cracks will be too narrow to be readily visible and the effect of the uneven settlement on the stresses in the beams can be taken care of by observing the rules contained in Appendix 2 of the Paper No. F-12. Hence the structure will not be faultless,
but it will serve its purpose for an indefinite period. On the other hand, if the reinforced concrete beams have a height in excess of 4 ft, or if the side-walls of a cellar are made out of reinforced concrete the cracks will be plainly visible and the reinforcement may be exposed to corrosion. Furthermore, cracks of this type in structural members located below the ground water level may suffice to injure the isolation and to permit important infiltrations.

According to these statements the detrimental effect of a given settlement may range anywhere between none at all, as in the case of a plain brick building and very objectionable breakage liable to affect the safety of the structure. In order to adapt the rules for designing different types of buildings to the local soil conditions two requirements must be satisfied. The first one consists in making a systematic collection of complete settlement records. A record is not complete unless the number of reference points is sufficient to serve as a basis for an accurate contour map of the warped surface produced by the settlement. These reference points should be established along the outside walls, in the interior of the buildings and on the strip of ground surrounding the building. The figures 30 to 33 in the Paper No. E-1, Vol I, are good examples. By means of the micrometer hose level which I introduced some five years ago in Vienna, the readings can be made far more rapidly and conveniently than with a levelling instrument and the error of observation can be kept well below 1/100 in. (See H. Lischner, Genauigkeitsuntersuchung zur Messung von Setzungen nach dem Verfahren von Prof. Terszachi. Ztschr. fflr Instrumentenkunde, 56. Jg. 1936, H. 4.)

The second requirement consists in collecting data regarding the maximum deformation which the construction members of reinforced concrete structures can stand without objectionable breakage. Since theory does not inform us on the amount of plastic deformation of the concrete nor on the nature and the width of the cracks, reliable data can only be secured by means of careful measurements and observations on buildings whose base has been subject to a known degree of warping on account of unequal settlement. Investigations of this type can be compared to the "condition survey" on concrete roads.

Soil exploration. The design of pile foundations in Shanghai seems to be governed exclusively by the rule of keeping the load per pile within specified limits. When following this rule, one does not get more than a very vague idea of the settlement of the supported structure. Although the Paper No. F-12 contains not more than 10 records, it includes two very similar buildings which illustrate this statement. The design of the foundation for the building No. 1 in Table 1 is much more conservative than that for No. 6. Yet the value S for No. 1 ranges between 1.7 and 2.9 in. while that for No. 6 ranges only between 0.8 and 1.9 in. The real margin for the value of S for buildings with fairly identical foundations is probably far in excess of that disclosed by the buildings No. 1 and 6. Since the subsoil of Shanghai consists, to a great depth, of more or less uniform silty materials, the reason for the margin can only reside in local variations in the consistency and permeability of the soils. Since the permeability merely influences the time-rate of settlement, the vital property is the consistency. Variations in the consistency could easily be detected by means of some cone penetration device such as I used in 1929 in New York (Die Bautechnik, 1930, H. 31 and 32), or, if the soil is sufficiently soft, by means of the tool described in the Paper No. E-3, Vol I of the Proceedings. The results of such observations may also assist in selecting the most economical length for the piles.

Conclusion. The information contained in the Papers No. F-12 and F-13 represents a very promising start towards accumulating the raw material required to establish an adequate building code for the city of Shanghai. However, in order to secure all the vital information which is needed for this purpose it would be necessary to intensify the observations and to broaden their scope along the lines suggested in this discussion. Considering the benefit to be derived from a reliable building code the costs involved in securing the information are negligible. These costs consist in the cost of establishing an adequate number of reference points, and in the salaries of two or three young, trained men.

No. F-22 DISCUSSION ON THE MOVEMENTS OF THE PIERs OF THE MISSISSIPPI RIVER BRIDGE, PAPER No. F-11

Participants: Dr. Karl von Terszachi, Austria, and Prof. William F. Kimball, U.S.A.

Karl von Terszachi: When reading Mr. Kimball's Paper No. F-11, Vol I, dealing with the settlement record for a bridge across the Mississippi in New Orleans, I was rather surprised by the good agreement between the computed and the observed values of the settlement of the river piers. For all these piers the seat of settlement is located within a thick bed of fine sand, subject to very rapid consolidation. By driving the sampler into a material of this type some compacting effect seems to be inevitable. Hence the samples cannot be considered strictly identical with the undisturbed material. Furthermore, the computation was based on the assumption of complete lateral confinement of the loaded material. On this assumption, one should obtain for the relation between the unit load, p, and the settlement, S, a curve whose slope decreases with increasing values of p, similar to the curve C1 in Fig. 1. Eight years ago I made a loading test on a reinforced concrete slab covering an area of about ten sq ft, located on the bottom of a shaft with a depth of about 50 ft, in Houston Street, New York. The seat of the settlement was entirely located within a stratum of medium dense, dry sand. The sides
of the shaft were firmly timbered in order to imitate the conditions which exist above the base of a pier. By gradually loading the slab we obtained a settlement curve similar to curve C2 in Fig. 1. The downward curvature indicates a rather important lateral yield of the loaded sand. By neglecting this yield in a settlement computation one underestimates the amount of subsidence. The satisfactory agreement in Mr. Kimball's case seems to indicate a marked anisotropy of the sand located beneath the river piers, involving an unusually high resistance against compression in a horizontal direction. The test boring records did not contain any indication of the existence of such an anisotropy, and as a matter of fact, it is difficult to see how the records could disclose its existence at all. Hence the satisfactory agreement between the computed and the real settlements was due to a chance factor which is not present in many cases. For piers supported by an isotropic bed of sand we have no means of estimating the settlement except from previous settlement records.

In his report Mr. Kimball also describes the downward and upward movement of the piers which was produced by the rise and fall of the level of the river. I have two similar cases on record. In one of them the piers of a railroad viaduct were found to undergo a periodic upward and downward movement involving two maxima and two minima per year for the elevation of the piers. The underground of the piers consisted of stiff clay and of disintegrated mica schist which covered the slopes of the valley. The second case involved a periodic movement of the foundation of the railway terminal of the Compagnie Transatlantique in Le Havre, France. This movement is produced by the alternation between low and high tide in the adjoining bay. The average tidal variation of the water level is approximately equal to 16 feet. The structure is located along a driveway bordering the crest of a quay wall. The rise and the fall of the sea level produces a simultaneous rise and fall of the foundations located close to the driveway, with reference to the foundations located at a greater distance. The amplitude of the movement is approximately equal to 4 mm or 1/6 inch.

In contrast to what was observed in Le Havre, a rise of the water level of the Mississippi river produced not a rise but a descent of the river piers and vice versa. My conception of the causes of this phenomenon are illustrated by the sketch shown in Fig. 2. According to Mr. Kimball's report, the bridge is located above a bed of sand with a depth of about 2000 feet. The sand contains horizontal, continuous layers of stiff and very feebly permeable clays, such as AB in Fig. 2. Since there is no free communication between the river and the ground water located below AB an increase of the cross-section of the river as indicated by the shaded area in Fig. 2, increases the load on AB over a belt with a width of about 2600 feet. This, in turn, increases the pressure on the sand located beneath AB over a belt with the same width. This local increase of the pressure produces by necessity a subsidence. Since the process of application and subsequent removal of the load represented by the shaded area in Fig. 2 is repeated year after year, the subsidence produced by the load has become almost perfectly elastic and reversible and accounts for the periodic upward and downward movement of the river piers.

William P. Kimball: I should like to comment on Dr. Terzaghi's discussion regarding the cause of settlement and rebound of piers on the Mississippi River bridge with the rise and fall of the river stage.

In my paper I suggested the possibility of elastic compression of the soil caused by the weight of fifteen feet of water, which is the approximate rise in the river level during flood stage. This was the cause suggested by Dr. Terzaghi also, in his discussion. Since then I have had discussions with Mr. Cummings regarding computations which he had made of what this elastic compression might amount to. He has computed the elastic compression of the deposit, based on the following assumptions: That there is a strip loading, 2600 feet wide, approxi-
mately the width of the river, and infinite in length, the approximate length of the Mississippi River. The weight of the deposit is 120 pounds per cubic foot. That is probably a fair assumption. The load is 1000 pounds per square foot, which is about right for fifteen feet of water. The equations of Mohr for stress under strip loading and of Froehlich for settlement, using \( n = \frac{1}{2} \), are correct. Also, the depth of the deposit below the river is 2000 feet. That is not definitely known, but probably the depth to bed rock is of that order.

It was necessary in this calculation to assume a modulus. Mr. Cummings used Froehlich's modulus, \( w \), and he used it for value 0.006. This value of \( w \) corresponds to a value of \( k \), derived by Dr. Terzaghi and shown in his Erdebaumechanik, page 92, Table 21. He shows variations of \( k \) between 83 and 176. As I understand it, that is the value, \( X \), to be used in the equation, \( E = X p \). It is the elastic coefficient, where \( E \) is the modulus of elasticity of the soil. \( w \) is the real proportion of \( X \), and 0.006 corresponds to a value of 167 for this term, \( X \), which is fairly high. In other words we have assumed a fairly stiff material, the values of \( X \) shown ranging between 80 and 176. We have chosen a value pretty close to the top.

Mr. Cummings obtained a value for the elastic compression of one and one-quarter inches. This corresponds almost exactly with the observed dip, shown in Paper No. F-11, Vol I, varying between three-quarters of an inch and one and one-quarter inches. That is a remarkably close agreement. However, this value corresponds to a modulus of elasticity of the soil of 1/0,000,000 pounds per square inch, which seems to me to be extremely low for a modulus of elasticity of a confined soil mass such as this.

We are considering the possibility of true elastic compression. As I understand it, this consideration must eliminate the possibility of flow of either soil or water in a confined mass.

If we consider another possibility, which is the one I like to consider myself, that the only elastic compression possible is that of the water, and if we repeat our calculations using as the modulus of elasticity of the water the value of 300,000 pounds per square inch instead of 1/0,000,000 pounds per square inch, which I believe is the modulus given at temperatures between forty and fifty degrees Fahrenheit, then obviously we obtain a value for the elastic compression amounts to about one-half inch. This is only about half of that actually observed.

It seems, therefore, that elastic compression cannot account for all the rise and fall of those bridge piers. The thought that we should consider modulus of elasticity of the water is that, if we can't have any escape of water and if the water in the soil takes the load when the load is first applied, as we assume in the consolidation theory, then the water must be carrying the load, and if the water is carrying the load it is the water which is going to compress elastically.

There is just one other point that I should like to make, and that is in regard to the use of consolidation tests to estimate the settlement caused in fine sands. I should agree with Dr. Terzaghi's skepticism, but I would suggest in this particular case that the sand strata lie more than one hundred feet below ground surface, and may for this reason act as a confined mass, as simulated in the consolidation test.

I should also like to second Dr. Baas' suggestion of yesterday, that it is the only method we have and therefore it is the best.

**Karl von Terzaghi:** Although the subject of Mr. Kimball's remarks has no outstanding practical importance I wish to answer them for the mere sake of argument. The term \( X \) which appears in his formula refers to a laterally confined material. If the depth of a layer of sand is greater than about one-half of the width of the loaded area, the greater part of the settlement is due to the lateral yield of the sand. This fact was conclusively demonstrated by numerous, independent investigations. In the case of the Mississippi River the width of the loaded area is about 2500 feet and the depth of the bed of sand approximately 2000 feet.

**William P. Kimball:** If that lateral yield occurs then there must be a heaving.

**Karl von Terzaghi:** The heave is always imperceptible, unless the material is loaded close to its ultimate bearing capacity. This can be convincingly demonstrated by placing a weight on the horizontal surface of a thick layer of gelatine. The settlement of the weight is almost exclusively due to the lateral yield of the gelatine. Yet the heave is so imperceptible that it cannot be noticed with the naked eye. For Boussinesq's semi-infinite elastic solid the heave is always equal to zero although the material may be perfectly incompressible. Hence the explanation which I suggested for the movement of the bridge piers may not be so improbable after all.

**No. F-23**

**DISCUSSION OF PAPER NO. F-14 (By Letter)**

**SETTLEMENT RECORDS OF THE MISSISSIPPI RIVER BRIDGE AT NEW ORLEANS**

William P. Kimball, Assis. Prof. of Civ. Eng., Thayer School of Civil Engineering, Hanover, N.H.

Since the Conference additional settlement records have become available showing the recent behavior of the piers of this bridge. It is the purpose of this discussion to present these records as a supplement to the original paper.

Table II, page 91 of the Proceedings, Vol. I may be brought up-to-date by adding the settlement observations of June 18, 1936 as follows:
Settlement readings made during the 1936 flood period indicated the same peculiar behavior of the piers as in 1935; that is, additional settlement was induced by the rise in the river stage, and a rebound or upward movement was recorded after the flood had subsided. The 1936 flood period was not as long as the 1935 period, and the river stage rose only 10 feet instead of 14 feet as in 1935. The rebound of the five main river piers following the 1936 flood varied from 5/8 to 1 1/8 inches, averaging about 3/8 inch. The rebound of the five main river piers following this year's flood varied from 3/8 to 3/4 inch, averaging about 3/16 inch. It is interesting to note that the ratio of 3/4 to 3/8 is about equal to the ratio of the corresponding rise in river stage for 1935 and 1936, 11 feet to 10 feet. In general, it appears that the progressive settlement of the piers is unaffected by the periodic down-and-up movement occasioned by the floods. From a purely utilitarian point of view this behavior is of course not alarming, but as an unexplained and hitherto practically unobserved phenomenon it seems to the author that it should arouse the curiosity of soil scientists.

No. F-21

DISCUSSION

William S. Housel, Assist. Prof. of Civ. Eng., University of Michigan, Ann Arbor, Michigan

Ten minutes is far too short a time to present completely one's position on one or several controversial points which have been the subject of discussion during the meetings of this Conference. Consequently I have attempted to formulate as briefly as possible in written form some of my own reactions which have been accumulating during the discussion of the past several days and which are clamoring for expression too insistently to be ignored for my own peace of mind.

I have been impressed with the tremendous opportunity for everyone attending the Conference to learn from the experience and observations of members from so many different countries who have presented their problems to the Conference. I feel that we all owe Harvard University and the organizers...
of the Conference a debt of gratitude for bringing together so many different points of view on these matters of common interest. I have also been a little bit dismayed by the apparent complexity of the problems which have been discussed and the apparent failure of various investigators to find a common basis on which to solve some of the so-called mysteries of soil behavior.

Some of the results of soil investigation with which I have had the privilege of being connected have been briefly presented in a "Report from the Soil Mechanics Laboratory" at the University of Michigan which has been abstracted for Vol II of the Proceedings and which I understand will be presented in complete form in the final copy of the Proceedings. Conclusions drawn from these investigations do not agree in all respects with some of the discussions which have been presented, and in other cases may serve to corroborate deductions which others have made from their studies.

First, I would mention a few points in connection with the interesting work of Mr. Cuevas in Mexico City regarding the soil movement when an excavation is made and when a building is subsequently erected. In Detroit, if it is necessary to qualify the statement, plastic clay, according to my observations, shows exactly the same behavior as building clay noted by Mr. Cuevas but to a reduced scale which is of course consistent with a much older and more stable deposit. Mr. Cuevas discussed two cases, which are also borne out by settlement observations in Egypt presented by Messrs. Hanna and Zashchobovareff, of adjacent buildings, in one case very close together, in the other case further apart. Similarly in Detroit such observations have been made and show substantial agreement. A basis of analyzing and describing such observations is available and has been some years before soil mechanics achieved recognition as a separate branch of engineering science. If one can accept the equations for stress distribution developed from elastic theory by Boussinesq as qualitatively correct, it appears that with some later refinements the broader aspects of the problem are solved even though it must be recognized that soil is far from being the ideal material of elastic theory.

It is possible from these equations to work out the lateral and vertical components of stress in the region between the two buildings and depending upon the proximity of the buildings account for all of the observations which have been made. The excavation may be treated as removal of weight or negative pressure and the elevation of a building as a load application or positive pressure. In regions very close to a weight removal the ground may rise due to the removal of vertical components which are in excess of lateral components. In regions further removed which depend upon the supporting influence of lateral components, the ground or a building may settle. To a certain extent, depending upon the soil resistance, the reverse may be true upon the erection of a building adjacent to another structure. As an illustration, I have investigated two cases in Detroit where buildings have settled enough to be damaged somewhat due to the removal of weight, occasioned by the wrecking of one heavy structural adjacent to another.

Second, I would like to make a few comments on load tests and their application to building design. It is well recognized that load tests are affected by the size and shape of the bearing area, by the rate of application of the load, and by the degree of confinement due to surrounding overburden all of which must be properly considered in making the tests and in analysis of the results of such tests. It is, however, entirely feasible to conduct such tests so that the data obtained is truly representative of the conditions under which actual footings will be expected to carry the design load for which they will be proportioned. It is recognized that tests on bearing areas from 1 sq ft to 9 sq ft will be controlled by soil resistance in a depth which probably does not exceed greatly the diameter of the bearing area, and it is obvious that if there are significant variations in soil strata, tests must be made at several elevations or wherever these changes are found during preliminary investigations. The bearing test is actually an integrated measure of the behavior of a representative sample of the underground in its natural undisturbed state and under conditions whioh cannot possibly be reproduced.

These statements represent conclusions drawn from investigations made under a situation which may be considered fortunate or unfortunate depending upon one's point of view. Practically all of the investigations which I have had an opportunity to make have been offered by engineers who had a structure to build and no basis for designing their substructures. While perfectly willing to undertake a comprehensive investigation, it was definitely understood that within a reasonable period of time someone would have to decide how many pounds per square foot the footings would be designed for and in some cases how much settlement would result. When all is said and done, the fact remains that load tests on comparatively small areas have been made, the results analyzed and used in the design of structures which have since behaved in substantial agreement with anticipated results.

Third, I wish to comment on the subject of the settlement of structures and certain aspects of continued settlement. According to my observations of time-settlement relations, there appear to be two basic phenomena which may be represented by time-settlement curves.

In the first place there is consolidation of the soil due to volume changes which represent the compression of void spaces in the soil structure. In porous soils this part of the settlement may be relatively large while in well-consolidated materials it may be relatively small. This consolidation will take place over a period of time which may be four or five years in a large mat foundation, two or three months in the case of a pier footing, or considerably less than an hour for a smaller test area. In clay soils with water filled voids and a relatively high degree of impermeability I have yet to encounter conclusive evidence that the migration of water through the soil due to applied pressures
within yield value of the soil under plastic flow is of more than negligible importance.

After the period of consolidation one of two situations may arise. For a certain intensity of pressure one may say that the consolidation has been complete, the pressure being less than the yield value there is no continued or progressive settlement and the settlement curve approaches a horizontal asymptote. For a higher intensity of pressure the consolidation is also complete but the load is greater than the yield value of the soil and settlement continues. It appears, as mentioned by Dr. Terzaghi earlier in the discussion, that such settlement continues at a uniform rate and the settlement curve approaches a sloping asymptote.

I cannot see, however, anything about this situation new or awaiting explanation by investigators of soil mechanics. This is entirely in accord with the conceptions of plastic substance, outlined, I believe, by James Clerk Maxwell approximately in the middle of the last century. It is not at all surprising that plastic clays follow the laws of plastic flow which are quite well known, in fact it would be surprising if they didn't.

According to these principles, Bingham, Nadai, and others, define a plastic material as a substance which will sustain a certain shearing stress without movement but at a higher stress will be deformed gradually without rupture, the rate of deformation being directly proportional to the stress in excess of the yield value.

The determination of yield value in my opinion is the most important factor which practical foundation engineering has to consider. Incidentally this point bears on a question put to the Conference which, so far as I am aware, has not been definitely answered. The yield value according to definition as applied to cohesive soils is the shearing resistance at zero normal pressure assuming, of course, that no dynamic effects are introduced due to rapid load application.

I have encountered one example of load tests and building settlement which may be of interest to the discussion. Load tests were conducted on a clay soil on several strata of different degrees of resistance at the site of a power plant in Detroit. See Fig. 1a. According to tests a soft strata of clay some 25 feet below the foundation mat appeared to be overloaded. An examination of the 15 year settlement record of the structure corroborated the deduction from load tests showing a definite progressive settlement which had been essentially uniform after the first five years. A correlation of pressure intensities on the soft layer with rates of settlement within zones of equal pressure verified the linear relation between pressure in excess of the yield value and rate of settlement of that region. See Fig. 1b and 1e. This linear relation extended back to zero rate of settlement showed a very close agreement between the results obtained from load tests.

These examples are not all but many investigators have uncovered similar evidence. Thus in addition to consolidation we have with us plastic flow of plastic soils if that be strange. There are also load tests which must be considered, and in my opinion, one of the most promising avenues of attack on foundation problems. Data and experimental evidence is available and more will be collected. If we may personify such things as data, we may say that they are insistent little devils. We may forget them, we may ignore them, or we may at times even laugh at them, but they will return to plague us in our idle moments when we sit back to deliberate and review that which has been accomplished.

No. F-25 DISCUSSION ON PAPER NO. F-8, (By Letter)
Maurice Buisson, Ancien Eleve de l'Ecole Polytechnique
Chef du Service de Contrôle des Constructions Immobilières, Bureau Veritas, Paris

Comparison between the computed settlements and laboratory tests on undisturbed soil samples. Very likely the reasons why experimental results are less than computation on small loadings and more than computation on large loadings may be stated as follows: first, the sample was unloaded about 100 gr/cm² previous to test; second, a "disturbance" produced through the effect of shearing tensions existing in the ground in the vicinity of the borehole. As the author does not state either the characteristics for the studied soils, or the method used for "C" determination, it seems difficult to build an explanation on a strong basis.

Concerning this observation that "in spite of the small permeability of the peat, water pressure decreases practically immediately" it would be necessary to know the method of determining the permeability coefficient. This observation supports my belief as to the effect of shearing tensions stated above. When shearing tensions are developed in the ground, permeability increases because of the consequence "disturbance". On the other hand, permeability increases in a general way, when pressure increases, so that, if the permeability was measured by the direct method under a usual pressure amounting to 30/40 gr/cm², no wonder there is a large difference.

Finally the intact sample may also have been "disturbed" so that even under low loadings the "voids ratio" may be well under the ratio existing at the position in ground.
Before construction of the bridge was started there were sunk two boreholes for a depth of about 21 m each, to ascertain the character of the underground, viz. filling, gravel, clay and silt. From most of these layers samples were taken to have them tested in the Soil Mechanics Laboratory of the Technical University in Vienna. Furthermore a series of load tests were performed in the boreholes in different levels (see numbered spots in Fig. 1) by means of the "Soil load tester". For loading the soil a circular sheet-iron plate of an area of 177 cm² was used in this case. The testing device is seen in Fig. 2. Resulting load-settlement diagrams as shown in Fig. 4 yielded the values of the coefficients of compressibility to be determined in connection with the different levels in the underground from Fig. 3. This shows clearly a maximum of soil resistance in the level 4, and an intense decrease of bearing capacity in the layers down from level 5. Therefore foundation was designed as a shallow one with bottom level corresponding nearly to number 4.

The next table gives a summary of soil data from the aforementioned layers, determined in Prof. Terzaghi's Laboratory in Vienna.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Material</th>
<th>Mises</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Schotter a Sand</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>Schläffer Ton</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>Ton</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>Schläffer Ton</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Konkretionen</td>
<td>6</td>
</tr>
</tbody>
</table>

Table I

Verteilung der Bodenpressungen.

Fig. 1-4

Verteilung der Bodenpressungen.

Fig. 5-9

Verteilung der Bodenpressungen.

Fig. 10-12

A further problem to solve was the resistance of the 2.75 m thick clay layer against piercing through by the abutment, the underlying material being an almost liquid silt. Distribution of pressures on top of the clay layer was computed following the formulae of Boussinesq and represented by the bell-shaped diagram of Fig. 11. Then the clay layer was statically computed as a flat slab loaded by these pressures and resting on a continuous support with a coefficient of compressibility of $c = 10 \, \text{kg/cm}^2$. The resulting bending moments and transversal powers are shown. Maximum stresses are the following: 0.2 kg/cm² in bending, 0.015 kg/cm² in shear. Cohesion had been determined to be 0.5 kg/cm² by Laboratory test, so that the factor of safety of the clay layer against rupture was 2.5.
In the old abutment cracks had occurred, beginning next the
points of support of the superstructure and going obliquely out-
ward. They might be taken as an argument, that soil resisted
more than masonry work in this case. The type of ground pegs,
set in the new abutment foundation is shown in Fig. 12.

This last figure represents a diagram of load and corres­
ponding soil stress increase of the new abutment during construc­
tion. Most of the load has been applied by the concrete mass
of the abutment itself. After a 3 months winter period under
this load the work had settled only for 3 to 4 mm. Total and
final settlement is expected to be not more than 20 mm. For
the eventual possibility of lifting the superstructure has been
provided.

No. F-27

DISCUSSION OF PAPER NO. F-6 (By Letter)
Thomas A. Middlebrooks, U.S. Engineer Office, Fort Peck, Montana

It is the writer's opinion that the excessive settlement
was not due to either theory A or B since both are based on
movement of the fine sand from underneath the pier. This
opinion is based on the following reasons:
a. Fine sand is a firm material and capable of support­
ing extremely high loads when well drained, as it must have
been in this case, with coarse material above and below.
b. If failure was going to occur due to plastic flow it
would have occurred in the first few years, since the fine
sand would increase in shearing strength due to consolidation
over the long period of years (13 yrs). Based on the theory
of consolidation, this fine sand should have been completely
consolidated in that time.
c. Overburden and vertical load from the pier should
have been ample to develop the necessary shearing resistance
in the fine sand.
d. There is sufficient overburden to prevent leaching
out of the fine sand.
a. Horizontal movement of the piers can be satisfactorily
accounted for due to unequal settlement, since a small dif­
ferential settlement would have caused a considerable hori­
zontal deflection. Two possible reasons for the pier failure
which seem more logical to the writer are: a. That the fine
sand as shown by the wash borings was in reality a clay
stratum (see Fig. 1 for typical example); and that it failed
due to plastic flow. This follows the reasoning offered by
the author on the fine sand but with fewer possible objections,
although it still leaves unexplained the fact that failure
should have occurred shortly after construction rather than
waiting 13 years. b. That the failure was in the lower por­
tion of the pier itself and was not caused entirely by the
soil conditions. A structure usually decreases in strength with time, whereas a loaded soil always
increases in strength.
This paper is an abstract of a report which will be presented to the International Association for Bridge and Structural Engineering, Second Congress, Berlin, October, 1936. (The reports of this Congress will be published by Wilhelm Ernst & Sohn, Berlin).

Realizing the importance of soil investigations at an early date the General Inspector for the German highways has ordered, soon after taking office, that prior to construction of any sections of the new superhighways all subsoils should be investigated in accordance with the most modern viewpoint of soil mechanics. During the year 1934, every one of the 15 chief construction field offices of the "Reichsautobahnen" was equipped with a soils laboratory for the purpose of not only investigating the subgrade beneath the future pavement, but also the foundation conditions of any structures, and thus to influence the selection of the type of bridges and of the foundations. Hence, these soils laboratories are also equipped with apparatus for investigation of the consolidation and permeability characteristics of undisturbed soil samples, thus making settlement analyses possible. Such settlement analyses have the following main purposes:

1. Survey of the upper soil layers in the whole of Germany regarding their suitability for carrying structures.

2. Comparison of the theory for settlement analysis with actual settlement observations.

3. The possibility of providing greater economy in the design of bridge foundations.

Points 2 and 3 are closely related since greater economy during construction can only be obtained on the basis of a settlement analyses.

Very few of the engineers and geologists working in these soils laboratories have had the opportunity to study soil mechanics at school, since only a few of the universities have as yet introduced this subject into their curriculum. This fact, together with the lack of laboratory help and the speed of construction, explains why the settlement analyses of those structures, which have been finished so far, are incomplete. Nevertheless, the wealth of available data now permit interesting conclusions, which are summarized in the following tables.

The relatively few complete settlement analyses show a coincidence between the computed and the observed settlements, which is satisfactory from a practical standpoint. It is noteworthy that the predictions of settlements based on theory are usually higher than the observed settlements. In a few cases this difference reached a maximum of 100 per cent, meaning that instead of a computed value of 100 millimeters, an actual settlement of only 50 millimeters was observed.

For the purpose of this study the soils may be classified approximately into the following four main groups: (1) clay and loess, (2) silt, (3) glacial till, ( = sandy, gravelly clays, or sand and gravel containing clay), (4) sand and gravel (in some cases containing clay). If we compare the settlements which correspond to these four soil classes (Table II), referring for the sake of simplicity the individual values with the help of the theoretical settlement curves to the elapsed time in which the major portion of the subsidence has occurred, then one can recognize a certain regularity in spite of the different influence of a number of factors, like thickness of soil stratum and soil pressures. To prevent misunderstandings, it is emphasized that the comparison contained in Table II and the conclusions which can be derived therefrom refer only to the soil conditions of Germany.

Of 72 observed structures about one-half are founded on glacial till and about one-third on sand-gravel strata of considerable thickness. From Table I one can see that for these two soil classes the settlements were between 0 to 20 millimeters and 0 to 10 millimeters, respectively. Stability analyses show that bridge piers with maximum soil pressures of 1.0 and 3.0 kilograms per square centimeter respectively have a factor of safety against rupture of the soil of between 7 and 9. The small settlements together with these large factors of safety would permit, therefore, larger soil pressures on glacial till and sand-gravel subsoils. For soil pressures of 6 and 5 kilograms per square centimeter respectively the factor of safety would still be 2 and 3 respectively, with only slightly larger settlements. Uniform settlements of even much larger magnitude are not harmful as indicated by the experiences of structures founded on clays which have settled in a relatively short time up to 200 millimeters. Damage to structures, e.g. tilting of the abutments, are confined exclusively to such cases where the embankments were built after the structure was finished or when the wedge of material, left open for backfill between the abutment and the embankment, was very large. The same is true also for structures founded on soft silt soils as long as they were not affected by slides and subsidences within the adjacent embankments. The majority of the structures are resting on raft foundations, while piles foundations are confined to such cases in which the irregular character of the underground indicated the possibility of differential settlements, or where the foundation had to be constructed in very soft silt soils.

Among the conclusions which are stated in the full report, the following are emphasized:

1. Glacial till and sand-gravel strata of considerable thickness permit larger soil pressures than are customary.

2. Embankments should be constructed as soon as possible to prevent irregular settlements of adjacent structures. Wedges of backfill behind the abutments, which must be completed after the construction is finished, should be kept as small as possible, particularly whenever the structure is founded on soft and impervious soil strata.
<table>
<thead>
<tr>
<th>Nr.</th>
<th>Type of Foundation</th>
<th>Ave. Soil Pressure in kg/cm²</th>
<th>Soil Stratum of Principal Imp.</th>
<th>Construction Finished</th>
<th>Predicted Settlement in mm.</th>
<th>Observed Settlement in mm.</th>
<th>Progress of Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Raft Found.</td>
<td>1.0</td>
<td>clay, stiff</td>
<td>June, 1935</td>
<td>—</td>
<td>abutmt. 51, Pier 56 Apr. 1936</td>
<td>continues</td>
</tr>
<tr>
<td>2</td>
<td>Raft Found.</td>
<td>2.5</td>
<td>clay, stiff</td>
<td>Dec., 1935</td>
<td>—</td>
<td>Pier 52, Mar., 1936</td>
<td>continues</td>
</tr>
<tr>
<td>3</td>
<td>Raft Found.</td>
<td>abutmt., 1.5 pier 1.0</td>
<td>silty clay, soft</td>
<td>Aug., 1935</td>
<td>abutmt., 150 pier 60</td>
<td>abutmt., 14, pier 25 Aug. 1935</td>
<td>completed</td>
</tr>
<tr>
<td>4</td>
<td>Raft Found.</td>
<td>2.75</td>
<td>clayey silt, soft</td>
<td>Feb., 1936</td>
<td>95</td>
<td>av. 50, Apr., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>5</td>
<td>Raft Found.</td>
<td>1.1</td>
<td>sandy clay</td>
<td>Nov., 1935</td>
<td>220</td>
<td>av. 110, Apr., 1936</td>
<td>continues</td>
</tr>
<tr>
<td>6</td>
<td>Raft Found.</td>
<td>1.5</td>
<td>clayey silt, soft</td>
<td>Dec., 1935</td>
<td>—</td>
<td>av. 280, Apr., 1936</td>
<td>continues</td>
</tr>
<tr>
<td>7</td>
<td>Raft Found.</td>
<td>1.4</td>
<td>clay, clay and silty clay</td>
<td>July, 1935</td>
<td>—</td>
<td>abutmt. 35, pier 70 Aug. 1935</td>
<td>continues</td>
</tr>
<tr>
<td>8</td>
<td>Raft Found.</td>
<td>2.6</td>
<td>loess</td>
<td>June, 1935</td>
<td>—</td>
<td>av. 40, Apr., 1936</td>
<td>almost completed</td>
</tr>
<tr>
<td>9</td>
<td>Raft Found.</td>
<td>2.6</td>
<td>loess</td>
<td>Aug., 1935</td>
<td>—</td>
<td>23 Apr., 36</td>
<td>almost completed</td>
</tr>
<tr>
<td>10</td>
<td>Pile Found.</td>
<td>abutmt., 1.1 pier 1.1</td>
<td>silty, silty clay</td>
<td>Dec., 1935</td>
<td>120</td>
<td>abutmt., 130 Pier 60 Apr., 1936</td>
<td>continues</td>
</tr>
<tr>
<td>11</td>
<td>Floating Pile Found.</td>
<td>1.1</td>
<td>soft silt</td>
<td>May, 1936</td>
<td>170</td>
<td>May, 1936</td>
<td>av. 500, May, 1936</td>
</tr>
<tr>
<td>12</td>
<td>Floating Pile Found.</td>
<td>1.5</td>
<td>silt, silty clay</td>
<td>Dec., 1935</td>
<td>200</td>
<td>May, 1936</td>
<td>av. 150, May, 1936</td>
</tr>
<tr>
<td>13</td>
<td>Floating Pile Found.</td>
<td>1.5</td>
<td>silt, soft</td>
<td>Aug., 1935</td>
<td>250</td>
<td>Mar., 1936</td>
<td>av. 60, Mar., 1936</td>
</tr>
<tr>
<td>14</td>
<td>Floating Pile Found.</td>
<td>1.3</td>
<td>silt, soft</td>
<td>Oct., 1935</td>
<td>350</td>
<td></td>
<td>av. 230, May, 1935</td>
</tr>
<tr>
<td>15</td>
<td>Floating Pile Found.</td>
<td>1.0</td>
<td>silt, soft</td>
<td>Nov., 1935</td>
<td>350</td>
<td></td>
<td>av. 200, May, 1935</td>
</tr>
<tr>
<td>16</td>
<td>Floating Pile Found.</td>
<td>2.0</td>
<td>silt, soft</td>
<td>July, 1936</td>
<td>350</td>
<td>westerly abutmt. 500</td>
<td>av. 300, May, 1936</td>
</tr>
<tr>
<td>17</td>
<td>Raft Found.</td>
<td>1.8</td>
<td>silt</td>
<td>July, 1935</td>
<td>350</td>
<td>May, 1936</td>
<td>av. 280, May, 1936</td>
</tr>
<tr>
<td>18</td>
<td>Raft Found.</td>
<td>2.5</td>
<td>glacial till, sandy clay</td>
<td>Dec., 1935</td>
<td>80--100</td>
<td>12 Dec., 1935</td>
<td>almost completed</td>
</tr>
<tr>
<td>19</td>
<td>Raft Found.</td>
<td>3.0</td>
<td>glacial till, sandy clay</td>
<td>Mar., 1936</td>
<td>10</td>
<td>av. 11, Mar., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>20</td>
<td>Raft Found.</td>
<td>3.0</td>
<td>glacial till, sandy clay</td>
<td>Nov., 1935</td>
<td>40--60</td>
<td>av. 7, Jan., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>21</td>
<td>Raft Found.</td>
<td>3.0</td>
<td>glacial till, sandy clay</td>
<td>Nov., 1935</td>
<td>20</td>
<td>av. 5, Jan., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>22</td>
<td>Raft Found.</td>
<td>3.0</td>
<td>glacial till, sandy clay</td>
<td>Nov., 1935</td>
<td>20</td>
<td>av. 15, Jan., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>23</td>
<td>Raft Found.</td>
<td>2.5</td>
<td>glacial till, sandy clay</td>
<td>July, 1935</td>
<td>60--80</td>
<td>av. 16, Jan., 1936</td>
<td>completed</td>
</tr>
<tr>
<td>24</td>
<td>Raft Found.</td>
<td>abutmt., 2.5 pier 3.0</td>
<td>glacial till, sandy clay</td>
<td>Jan., 1935</td>
<td>—</td>
<td>av. 5, Aug., 1935</td>
<td>completed</td>
</tr>
<tr>
<td>25</td>
<td>Raft Found.</td>
<td>2.5</td>
<td>glacial till, sandy clay</td>
<td>Apr., 1935</td>
<td>—</td>
<td>av. 15, Aug., 1935</td>
<td>completed</td>
</tr>
<tr>
<td>26</td>
<td>Raft Found.</td>
<td>2.0</td>
<td>glacial till, sandy clay</td>
<td>Aug., 1934</td>
<td>—</td>
<td>av. 120, Aug., 1935</td>
<td>completed</td>
</tr>
<tr>
<td>27</td>
<td>Raft Found.</td>
<td>abutmt., 2.6 pier 3.0</td>
<td>glacial till, sandy clay</td>
<td>Nov., 1934</td>
<td>—</td>
<td>av. 12, Aug., 1935</td>
<td>completed</td>
</tr>
<tr>
<td>28</td>
<td>Raft Found.</td>
<td>abutmt., 2.6 pier 3.0</td>
<td>clay, sand, glacial till</td>
<td>Nov., 1934</td>
<td>—</td>
<td>av. 10, Aug., 1935</td>
<td>completed</td>
</tr>
<tr>
<td>29</td>
<td>Raft Found.</td>
<td>3.0</td>
<td>glacial till, sandy clay</td>
<td>—</td>
<td>—</td>
<td>no settlement</td>
<td>—</td>
</tr>
<tr>
<td>30</td>
<td>Raft Found.</td>
<td>2.5--4.0</td>
<td>glacial till, sandy clay</td>
<td>—</td>
<td>—</td>
<td>no settlement</td>
<td>—</td>
</tr>
<tr>
<td>31</td>
<td>Raft Found.</td>
<td>2.65</td>
<td>glacial till, sandy clay</td>
<td>—</td>
<td>—</td>
<td>no settlement</td>
<td>—</td>
</tr>
</tbody>
</table>
TABLE I (continued)

<table>
<thead>
<tr>
<th>Nr.</th>
<th>Type of Foundation</th>
<th>Ave. Soil Pressure in kg/cm²</th>
<th>Soil Stratum of Principal Imp.</th>
<th>Construction Finished</th>
<th>Predicted Settlement in mm.</th>
<th>Observed Settlement in mm.</th>
<th>Progress of Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>Raft Found.</td>
<td>2.2—2.5 loamy sand</td>
<td>--</td>
<td>--</td>
<td>0—5</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>Raft Found.</td>
<td>3.0 silty sand with little clay</td>
<td>June, 1935</td>
<td>--</td>
<td>--</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>Raft Found.</td>
<td>2.5 clayey sand</td>
<td>Aug., 1935</td>
<td>--</td>
<td>--</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Raft Found.</td>
<td>abutmt. 2.0 sand with slight clay content</td>
<td>Sept. 1935</td>
<td>--</td>
<td>--</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>Raft Found.</td>
<td>abutmt. 1.5 sand-gravel, clay, peat</td>
<td>Mar., 1936</td>
<td>--</td>
<td>abutmt. 16, pier 15, 19</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>Raft Found.</td>
<td>abutmt. 1.5 sand-gravel, some clay</td>
<td>Aug., 1935</td>
<td>--</td>
<td>abutmt. 20—100, pier 15, Apr. 1936</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>38</td>
<td>Raft Found.</td>
<td>2.5 sand</td>
<td>Feb., 1936</td>
<td>10</td>
<td>Feb., 1936</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>39</td>
<td>Raft Found.</td>
<td>2.0 sand-gravel, loamy</td>
<td>Oct., 1934</td>
<td>--</td>
<td>horiz. 15, Mar., 1935</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>Raft Found.</td>
<td>2.5 sand</td>
<td>--</td>
<td>--</td>
<td>no settlement</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>41</td>
<td>Raft Found.</td>
<td>2.7 sand, locally with some clay</td>
<td>Apr., 1935</td>
<td>--</td>
<td>westerly abutmt. 15, easterly abutmt. 15</td>
<td>completed</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>Pile Found.</td>
<td>2.1 sand-gravel, clay</td>
<td>--</td>
<td>--</td>
<td>no settlement</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>Raft Found.</td>
<td>2.2 sand-gravel, clay</td>
<td>--</td>
<td>--</td>
<td>no settlement</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>44</td>
<td>Raft Found.</td>
<td>2.2 sand-gravel, clay</td>
<td>--</td>
<td>--</td>
<td>no settlement</td>
<td>--</td>
<td></td>
</tr>
</tbody>
</table>

TABLE II

<table>
<thead>
<tr>
<th>No. of Structures</th>
<th>Soil Class</th>
<th>Ave. Soil Pressure in mm.</th>
<th>Settlements in mm.</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>silt</td>
<td>1.1—2.0</td>
<td>200—1000</td>
</tr>
<tr>
<td>8 (11)</td>
<td>loam, etc.</td>
<td>1.1—2.6</td>
<td>50—200</td>
</tr>
<tr>
<td>31</td>
<td>glacial till, sandy and gravelly clay</td>
<td>2.5—4.0</td>
<td>0—20</td>
</tr>
<tr>
<td>24 (21)</td>
<td>sand, gravel</td>
<td>1.5—3.0</td>
<td>0—10</td>
</tr>
</tbody>
</table>

Note: Examples of settlement analysis, as carried out on structures of the "Reichsautobahnen", are contained in the following paper, No. F-29, by E. v. Gottstein.
Introduction. In several contributions to Volumes I and II of the Proceedings some examples are given, showing that the observed settlement reached in general only 20—30% of the calculated values. It may be of interest to show some examples in which calculated and observed settlements are practically in complete agreement. The following investigations and observations have been carried out by W. Knörlein and K. Vogl. Many additional examples are described in a report to the International Association for Bridge and Structural Engineers, 2d Congress, Berlin, October, 1936, by Dr. Ing. Leo Casagrande.

General remarks. In the following cases we have to do with overhead crossings with adjoining embankments. The subsoil consisted in all cases of soft glacial deposits of silt and clay. The settlement analyses were based on Terzaghi's theory of settlement of clay layers due to progressive consolidation. The compressibility and permeability of the compressible layers of the subsoil was determined by tests on undisturbed samples. The stress distribution under the structures was computed with Boussinesq's theory. The presence of piles has been neglected. It may be of interest to emphasize that in all cases the theoretical settlement curves have been calculated before construction.

Building I. Overhead crossing with 3 openings and adjoining embankments 3 m high. (Fig. 1)

Foundation. The abutments and the pillars are erected upon two mats of concrete, which were put upon Franki-piles 10 m in length. The soil pressure under the concrete plates was about 1.07 kg/m² and under the adjoining fill about 1.65 kg/m².

Subsoil. From 1.5 m to a depth greater than 28 m very soft glacial silt deposits of a lake, probably in an unconsolidated state. The physical data is given in Table I

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth in m</td>
<td>3.60</td>
<td>5.30</td>
<td>8.60</td>
<td>15.8</td>
<td>19.0</td>
</tr>
<tr>
<td>Natural water content</td>
<td>52%</td>
<td>34%</td>
<td>60%</td>
<td>40%</td>
<td>61%</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>72%</td>
<td>65%</td>
<td>56%</td>
<td>35%</td>
<td>15%</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>44%</td>
<td>42%</td>
<td>11%</td>
<td>27%</td>
<td>29%</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>28%</td>
<td>23%</td>
<td>15%</td>
<td>6%</td>
<td>16%</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
</tr>
<tr>
<td>Reduced coeff. of permeability</td>
<td>(6 \times 10^{-5})</td>
<td>(3.5 \times 10^{-5})</td>
<td>(1.8 \times 10^{-5})</td>
<td>(1.8 \times 10^{-5})</td>
<td></td>
</tr>
</tbody>
</table>

Grain size distribution:
- Sand, greater than 0.1 mm: 12%, 34%, 20%, 8%, 12%
- Mo, 0.1 mm - 0.02 mm: 51%, 42%, 71%, 42%, 26%
- Silt 0.02 mm - 0.002 mm: 30%, 24%, 7%, 50%, 16%
- Clay, less than 0.002 mm: 4%, --, 2%, --, 16%

Remark: 80% of the soil is soluble in HCl

Settlement calculation. For the determination of the chronological course of settlement the compressible layer was supposed to be of an infinite thickness and to be homogeneous. The coefficient of permeability of this fictitious homogenous layer was equal to the average value of the permeability of the different layers. The calculated time-settlement curve is given in Fig. 1b.

Observed settlements and remarks. The settlements have been observed from the beginning of the erection of the structure and are shown in Fig. 1b.

The observed settlements are identical with the calculated values. An irregularity, indicated by A, was produced by slides in the adjoining embankment.

Building II. Overpass with one opening and adjoining embankments 3 m high. (Fig. 2).
Foundation. Each abutment built on wooden piles 7 m long. The average soil pressure under the foundation mat was about 1.5 kg/m².

Subsoil. The cross-section of the subsoil is represented by Fig. 2b. The physical data see Table II.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth in m</td>
<td>10.2</td>
<td>11.2</td>
<td>19.5</td>
</tr>
<tr>
<td>Natural water content</td>
<td>36%</td>
<td>33%</td>
<td>32%</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>—</td>
<td>36%</td>
<td>36%</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>—</td>
<td>27%</td>
<td>27%</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>—</td>
<td>9%</td>
<td>11%</td>
</tr>
<tr>
<td>Reduced coeff. of permeability cm/min</td>
<td>$1.8 \times 10^{-5}$</td>
<td>$1.6 \times 10^{-6}$</td>
<td>$1.5 \times 10^{-6}$</td>
</tr>
<tr>
<td>Grain size distribution:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sand</td>
<td>35%</td>
<td>8%</td>
<td>8%</td>
</tr>
<tr>
<td>mo</td>
<td>55%</td>
<td>15%</td>
<td>12%</td>
</tr>
<tr>
<td>silt</td>
<td>10%</td>
<td>15%</td>
<td>58%</td>
</tr>
<tr>
<td>clay</td>
<td>—</td>
<td>7%</td>
<td>22%</td>
</tr>
</tbody>
</table>

Settlement calculation. For the determination of the time-settlement curve the subsoil was supposed to consist of two layers. The upper layer of mo-silt, 7.5 m thick, was supposed to drain to both sides because of a layer of water-bearing sand to a depth of 17.5 m. The under layer of clay of infinite depth was supposed to drain only by the above-mentioned sand layer. The calculated time-settlement curve is given in Fig. 2c.

Observed settlements. The settlement observation began after the driving of the piles. Though observed settlements are approximately of the same magnitude as the calculated values. (Fig. 2c). As to the time-settlement curve it must be remarked, that the observed curves show important deviations from the theoretical curve, indicated by A and B. Until the final soil pressure was nearly attained, no settlements of the points 3 and 4 could be observed. Then, a short time before the construction of the adjoining embankment was finished, suddenly there were settlements. It may be possible that at that instant an internal rupture of the intact structure of the soil took place. The same deviation from the theoretical time-settlement curve show the observed settlements of the substructure V.

Building III. Overhead crossing with one opening and adjoining embankments 1.5 m high. (Fig. 3)

Foundation. Each abutment built on Franki-piles 7 m long. Average soil pressure under foundation mat about 1.3 kg/m².

Subsoil. The cross-section of the subsoil is represented by Fig. 3b. The physical data see Table III.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth in m</td>
<td>7.20</td>
<td>11.2</td>
<td>17.4</td>
<td>20.0</td>
</tr>
<tr>
<td>Natural water content</td>
<td>54%</td>
<td>38%</td>
<td>44%</td>
<td>34%</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>51%</td>
<td>33%</td>
<td>?</td>
<td>32%</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>37%</td>
<td>23%</td>
<td>?</td>
<td>27%</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>13%</td>
<td>10%</td>
<td>?</td>
<td>5%</td>
</tr>
<tr>
<td>Reduced coeff. of permeability cm/min</td>
<td>$1.3 \times 10^{-6}$</td>
<td>$3.3 \times 10^{-6}$</td>
<td>$6.4 \times 10^{-6}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>Grain size distribution:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sand</td>
<td>15%</td>
<td>15%</td>
<td>53%</td>
<td>28%</td>
</tr>
<tr>
<td>mo</td>
<td>25%</td>
<td>15%</td>
<td>38%</td>
<td>18%</td>
</tr>
<tr>
<td>silt</td>
<td>51%</td>
<td>36%</td>
<td>7%</td>
<td>22%</td>
</tr>
<tr>
<td>clay</td>
<td>15%</td>
<td>4%</td>
<td>2%</td>
<td>2%</td>
</tr>
</tbody>
</table>

Settlement calculation. The same assumptions have been made as for Building II. For the calculated time-settlement curve see Fig. 3c.

Observed settlements and remarks. The average settlements, Fig. 3c, are only 10% smaller than the calculated values. The chronological course of the settlements is in good agreement with the calculated course.
Building IV. Overhead crossing with adjoining embankments 3.0 m. Fig. 4.

Foundation. The abutments are erected upon a concrete mat which was built on Franki-piles 10 m long. The average soil pressure under the concrete mat was about 1.0 kg/m².

Subsoil. Plastic silt-clay to a depth greater than 28 m. The physical data see Table IV. A disturbance of the structure of the clay greatly influences its consistency.

<table>
<thead>
<tr>
<th>TABLE IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample No.</td>
</tr>
<tr>
<td>Depth in m</td>
</tr>
<tr>
<td>Natural water content</td>
</tr>
<tr>
<td>Liquid limit</td>
</tr>
<tr>
<td>Plastic limit</td>
</tr>
<tr>
<td>Plasticity index</td>
</tr>
<tr>
<td>Reduced coeff. of permeability in cm/min</td>
</tr>
<tr>
<td>Grain size distribution:</td>
</tr>
<tr>
<td>sand</td>
</tr>
<tr>
<td>mo</td>
</tr>
<tr>
<td>silt</td>
</tr>
<tr>
<td>clay</td>
</tr>
</tbody>
</table>

Settlement calculation. For the determination of the chronological course of settlement the compressible layer was supposed to be of an infinite thickness. The calculated time-settlement curve is given in Fig. 4o.

Observed settlements. The observed settlements (Fig. 4o) are in good agreement with those calculated.

Building V. Bridge with 3 openings and adjoining embankments 2.5 m high. Fig. 5.

Foundation. Floating pile foundation, Franki-piles 10 m long. The average soil pressure under the foundation mat of the abutments disregarding the piles was about 1.03 kg/m², beyond the pillars 1.14 kg/m².

Subsoil. The cross-section of the subsoil is represented by Fig. 5b. The physical data are the same as for Building II.

Settlement calculation. The computation of the time-settlement curve (Fig. 5o) is based on the same assumption as was made for Building II.

Observed settlements and remarks. The actual values of the final settlements of the pillars are in good agreement with those calculated. The settlement of the abutments are 50% greater than those calculated, perhaps because the pressure of the piles had not been observed. The time-settlement curves show a very important deviation from the theoretical course. Until the final soil pressure was nearly attained no settlement at all could be observed. Then a short time before the construction of the adjoining embankment was finished, suddenly there were settlements. For that fact the same explanation may be given as for the same observation made for Building II.

No. F-30

COMMENTS ON VARIOUS PAPERS

(Editorial notes abstracted from oral and written communications.)

Paper F-2: The statement in the next to the last paragraph has been challenged. According to a communication received the shearing resistance does not decrease due to the application of a load, but remains at least equal to the shearing resistance which existed before the load was applied. In the example cited the computed shearing resistance of 0.12 kg per sq cm should be added to the shearing resistance which existed before application of the load.

Paper F-114: In several communications attention is called to the fact that the load tests for the chimney as well as for the large building described in the paper, were carried out on such small areas that they could not possibly give an indication of the settlements which will be produced by the structures.

Although the diameter of the base for the chimney is 10 m, and the underlying soil is stratified, the design was based on load tests on an area of only 0.2 x 0.2 m.

In the case of the large building the foundation is resting on a stratum of gravel of 3 m thickness, which is underlain by gray sand with unknown properties. In spite of these facts the design is based on the results of load tests on an area 0.13 x 0.13 m performed on the stratum of gravel.