GENERAL DISCUSSION

DONALD M. BURMISTER.[1]—There are two aspects of soil dynamics that are considered to be most important and on which the writer wishes to make a few comments. The first aspect has to do with the development of testing apparatus which can satisfactorily apply dynamic loadings. The papers presented in this Symposium well illustrate the problems and difficulties involved in the development and instrumentation of such apparatus. A great deal of ingenuity, patience, and time has been involved in these developments. The authors are to be congratulated on their achievements.

The second aspect has to do with the soil itself. In the final analysis, the significant determination of responses and performances of soil "in place" must be the major objective for developing methods and procedures for dynamic loading conditions. It is believed that dynamic responses and performances of soils cannot be properly determined, understood, and evaluated until the fundamental static responses of soils in laboratory tests and performances of soils "in place" in field tests have been more thoroughly investigated, understood, and significantly established on a fundamental basis by the applications of simulated performance and prototype methods of testing.

Soil is a most unusual construction material. There are no simple model-prototype relations between soil responses in the laboratory tests and full scale performances in the field that can be used invariably and reliably, such as exist for the common materials of construction. Furthermore, soil is always a prestressed material with a prestress at any depth at least equal to the weight of overburden above this depth. This generally is a very favorable condition, which improves performances. The stressing and straining of soil under foundation loads never starts from the state of zero stress and strain. The prestressing of triaxial specimens to increasing stress levels by a confining stress actually represents one aspect of simulated performance and prototype testing to simulate the influences of increase in depth upon the strength properties and stress-strain responses.

The responses of soil in laboratory tests and the performances of soil in small-scale field tests or in full scale construction sequences for structures are strongly influenced and conditioned by the kind, relative dominance, and sequence of conditions imposed. As a consequence, a first fundamental concept of soil responses and performances states that a prestressed and preconditioned soil is in reality a "new construction material," having different and essentially new responses and performances. There is no difficulty for engineers to realize and to treat, for example, prestressed concrete as a different and improved construction material in contrast to ordinary reinforced concrete. As a corollary to this concept, significant responses and performances of soils which have direct and valid applications to particular natural situations can be determined and established only by a thorough visualization

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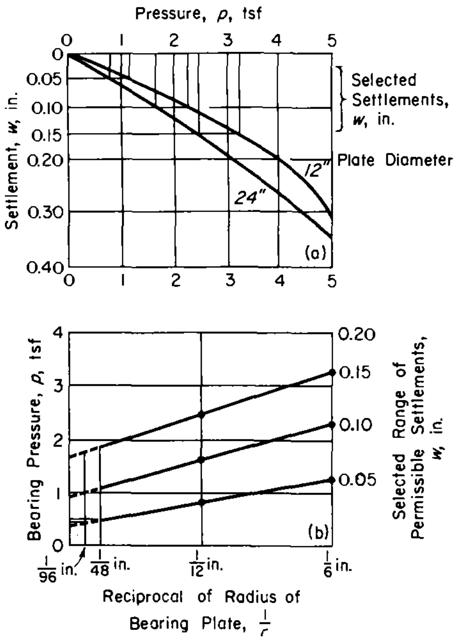


(*a*) Pressure-settlement curves from load-bearing tests.

(*b*) Bearing pressures versus reciprocal of radius of bearing area for series of selected permissible settlements.

Fig. 1.—Prototype Pressure-Settlement Performances. Housel's Concept. Evaluation of Bearing Pressure for Selected Permissible Settlement from Pressure-Settlement Curves for Two Sizes of Bearing Plates.

Housel's Empirical Equation:

$$p = n + m^{P}/_{A} \qquad (1)$$

where:

p = bearing pressure,
$P/_A$ = perimeter-area ratio: $2\pi r/\pi r^2 = 2/r$
n = empirical coefficient—pressure intercept, and
m = empirical coefficient—slope of line.

Modified Equation Based on Boussinesq Theory to Fit Settlement Performances:

$$w = \frac{\pi}{2}(1 - \mu^2)\frac{pr}{\left(E_o + 2r\dfrac{\partial E}{\partial z}\right)} \qquad (2a)$$

$$p = \frac{1}{\pi/2(1 - \mu^2)}\left[\frac{2\partial E}{\partial z}w + \frac{E_o w}{r}\right] \qquad (2b)$$

Intercept and Slope

where:

$\pi/2$ = coefficient for rigid circular area,
$\pi/2 \times 1.12$ = coefficient for rigid square area,
μ = Poisson's Ratio,
E_o = soil modulus of surface,
$\frac{\partial E}{z\partial}$ = increase in soil modulus with depth, taken at depth of $2r$,
w = selected permissible settlement, and
p = corresponding bearing pressure.

Proportioning Footings for Equal Settlement:
Find by trial values of p and $1/r$ (or p and $1/b$) to satisfy condition that

Column Load, $P = \pi r^2 p$ (or $P = 4b^2 p$),
where $\left(\frac{1}{b} = 1.12 \times \frac{1}{r}\right)$.

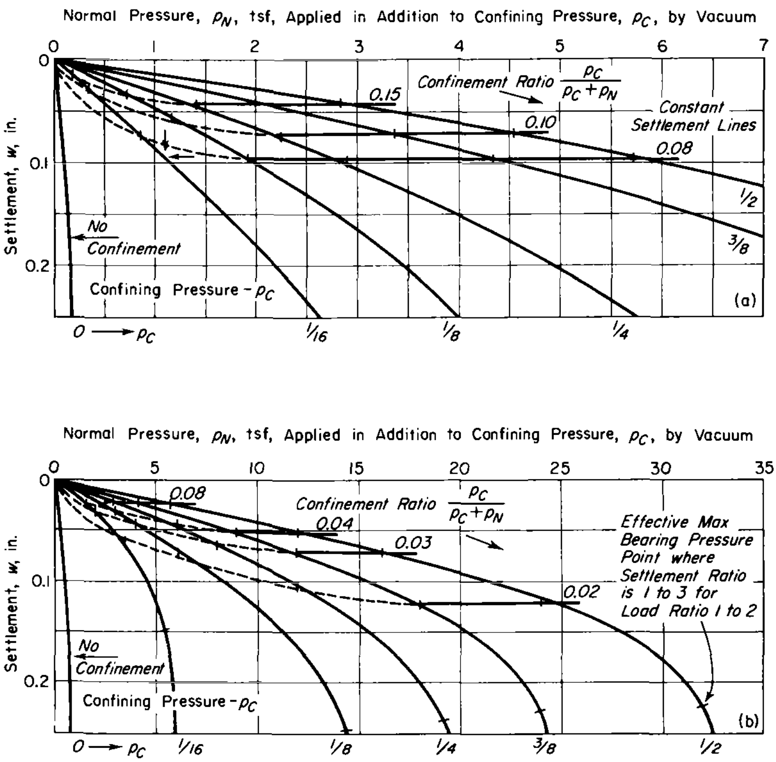
and adequate evaluation of the conditions that control and by a carefully planned and properly executed program of laboratory soil tests or field tests in accordance with the principles of simulated performance and prototype testing of soils.

The first logical and necessary step in treating soils under dynamic loading is prototype static load-bearing tests of soils "in place" to determine significant pressure-settlement relations that are capable of being translated into representative and valid estimates and predictions of full-scale footing performances. Here the important and fundamental concept, first proposed by W. S. Housel[2] also applies, which states that at least two load-bearing tests are required on different sizes of bearing plates in order to determine significantly pressure-settlement relations for predicting full-scale footing performances. This concept of Housel's for the pressure-settlement performances of footings is illustrated in Fig. 1(*b*), but has been modified by the writer to a more fundamental dimensionally correct basis, as indicated by a comparison of Eqs 1 and 2 (see notes under Fig. 1). A permissible bearing value, p is shown to be dependent, first, upon the reciprocal of the radius, $1/r$ of a circular bearing plate or the reciprocal of the half-width, $1/b$ of a square bearing plate, which are the principal plotting arguments, and in Fig. 1(*b*), upon a selected settlement, which may be considered permissible for the type of construction, loading conditions, and soil conditions. The family of selected settlement curves in Fig. 1(*b*) were obtained, for example, from pressure-settlement curves for two sizes of bearing plates from a series of equally spaced selected constant settlement lines, shown on the pressure-settlement

[2] W. S. Housel, "Report of the Symposium Committee on Load Tests to Measure the Bearing Capacity of Soil," Symposium on Load Tests of Bearing Capacity of Soils, *ASTM STP No. 79*, Am. Soc. Testing Mats., p. 2 (1947).

curves of Fig. 1(*a*), which define pressure values for the two plate sizes.

The basic fact established in Fig. 1(*b*) by this concept is the existence of a positive pressure intercept on the pressure axis. The use of a single bearing plate the pressure axis indicates by Eq 2 that an increase in soil modulus must exist with depth. This is in accordance with the established fact by triaxial tests that the strength properties of soil increase with increase in confining stress. Housel's



(*a*) Soil placed in thin layers to a uniform relative density of 30 to 40 per cent.
(*b*) Soil placed in thin layers to a uniform relative density of 70 to 80 per cent.

FIG. 2.—Small Scale Simulated Performance Prototype Bearing Test, Modified CBR Test. Concept: Prototype Performance Criterion of

$$\text{Constant Confinement Ratio} = \frac{p_c}{p_c + p_n} \text{ for Equal Permissible Settlements of Footings.}$$

only and hence the use of a settlement line through the origin would severely penalize the actual more favorable pressure-size-settlement relations by use of too low a permissible bearing value. As indicated in Eq 2, the slope of the family of settlement lines is dependent upon the selected permissible settlement and soil modulus, E_o. The positive intercept on concept is actually an application of the principle of prototype testing of soil.

A third important and fundamental concept, proposed by the writer, states that the confinement of the supporting soil mass directly beneath a footing by placement below the surface of the immediately adjacent ground level and by the presence of the surrounding soil mass above the level of the base of the footing has even more important controlling influences on the pressure-settlement relations for footings. Therefore, the second principle of prototype load-bearing testing of soils requires that the confinement conditions must also be simulated representatively in two load-bearing tests on different sizes of bearing plates. The fundamental nature of this concept and the implications with regard to proper interpretation, evaluation, and representative and valid use of load-bearing test results are illustrated in Figs. 2 and 3.

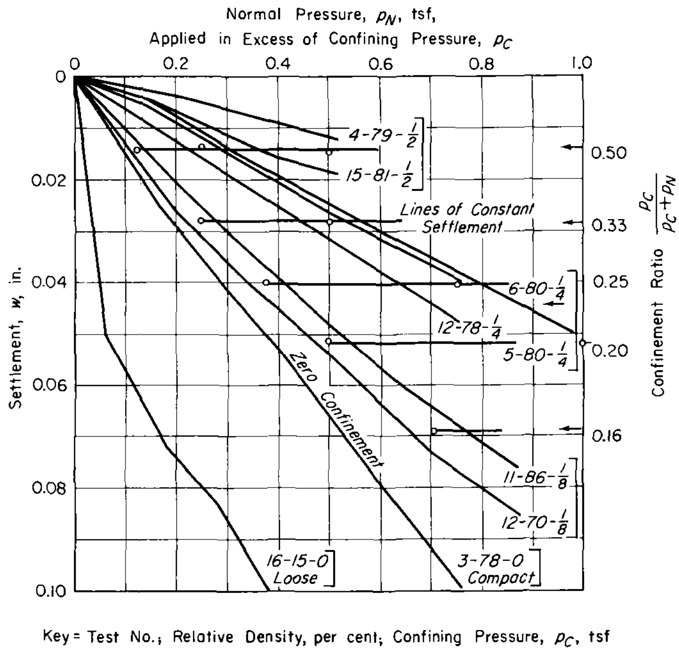


Fig. 3.—Simulated Performance Prototype Bearing Tests on 12 by 18-in. Footing.

Soil Identification—brown coarse to fine SAND, trace — Silt, trace — fine Gravel.
Confining Pressure, p_c, tsf.
Note.—$p_c = \frac{1}{4}$ tsf, equivalent to footing placed at depth of 5 ft.
Normal Pressure, p_n, tsf—applied in addition to confining pressure by dead weights of steel plates, symetrically placed.

In Fig. 2 the results are given for a series of very small-scale penetration tests on a 2-in. diameter disk (actually a modified California Bearing Ratio (CBR) test) for the sand noted in the identification. In Fig. 2(*a*) the soil was placed in thin layers at a uniform relative density of 30 to 40 per cent, which is in the range commonly encountered in many natural deposits near the surface of the ground. In Fig. 2(*b*) the soil was placed in thin layers at a uniform relative density of 70 to 80 per cent, which illustrates the enormous increase and improvement in bearing value achieved by effective compaction. Separate tests were made for each relative density at confining stress levels of zero (no confinement), $\frac{1}{16}$, $\frac{1}{8}$, $\frac{1}{4}$, $\frac{3}{8}$, and $\frac{1}{2}$ tsf (tons per square foot) by vacuum methods applied to a thin rubber membrane sealed to the top of a 6-in. compaction mold.

At zero confinement, an insignificant bearing value is attained, the pressure-settlement performances being completely controlled and dominated by shearing displacements and escape of soil grains from beneath the edges of the 2-in. diameter disk. Furthermore, there is an insignificant improvement in performance with increase in relative density to 70 to 80 per cent at zero confinement. However, with a confining pressure as low as $\frac{1}{16}$ tsf there is relatively an enormous increase in bearing pressure for a constant settlement. A confinement of $\frac{1}{16}$ tsf is equivalent to only $1\frac{1}{4}$ ft of soil surcharge above the level of the base of a footing. The bearing pressure for constant equal settlements consistently increase with increase in confining pressure, but the improvement is enormously greater for the 70 to 80 per cent relative density in contrast to 30 to 40 per cent relative density. These facts have important implications with regard to prototype CBR testing of soils.

In Fig. 3 there are given the results for load-bearing tests on a 12 by 18-in. footing, which is in the range of sizes commonly used for determining pressure-settlement relations in field tests. These prototype load bearing tests were made in a 3-ft diameter by 3-ft high cylindrical container. Theoretical investigations of the stress conditions imposed by the footing loads used showed that the confining influences of the side walls of the container were negligible in comparison with the controlling influences of the surface confinement pressure. This surface confinement pressure increased by consolidation the level of the "earth pressure at rest" with a constant coefficient, K_o. Sand was placed in 6-in. layers at the desired relative density of 70 to 80 per cent. Confinement was attained by vacuum methods through a rubber membrane sealed to the top of the container and controlled at pressure levels of zero (no confinement), $\frac{1}{8}$, $\frac{1}{4}$, and $\frac{1}{2}$ tsf.

A similar pattern of bearing pressure-settlement performances was obtained, as given in Fig. 2, which represents a verification of the validity of this principle of prototype pressure-settlement testing. There is a consistent and significant increase in bearing value at constant settlement with increase in confining pressure. There is, however, an appreciable bearing value for this size bearing area at zero confinement and a somewhat smaller increase in bearing pressure with increase in confinement pressure to $\frac{1}{8}$ tsf. This aspect of the pressure-settlement performances of this larger scale bearing area will be interpreted later. A very significant increase and improvement in bearing values occurs at a confining pressure of $\frac{1}{4}$ tsf which is equivalent to a 5-ft embedment of a footing or surcharge of 5 ft of soil above the base of the footing and is within the range of common footing depths. Thus some idea of the penalty can now be judged that is involved in the usual practice in making load bearing tests on the surface of the ground. At a settlement of 0.03 in., the bearing value under a confinement pressure of $\frac{1}{4}$ tsf or 5-ft surcharge of soil is 2.5 times that of the test at the surface of the ground with zero confinement and at a settlement of 0.05 in. the bearing value for this confinement is 2.36 times. These tests could not be carried by dead weight loading beyond 1.0 tsf, but the results are significant with regard to pressure-settlement performances obtained by prototype testing methods.

On the basis of these prototype pressure-settlement performances, a fourth fundamental concept, and a most important criterion, can be formulated tentatively for similitude of pressure-settlement performances of footings, namely, for a constant selected permissible settlement of a footing, prototype pressure-settlement performances require that the confining pressure and bearing pressure must be in a constant confinement ratio of $p_c/(p_c + p_N)$. Accordingly, higher bearing pressures require higher degrees of confinement for the same settlement in direct proportion to this ratio.

This concept and criterion of similitude of pressure-settlement performances are illustrated in Fig. 3 by the points at pressures satisfying this criterion and located at the average settlement between the curves for each confinement pressure of $\frac{1}{8}$, $\frac{1}{4}$, and $\frac{1}{2}$ tsf. These pressure-settlement points fall almost exactly on the horizontal constant settlement line noted for the given confinement ratio noted at the right hand side of Fig. 3. This concept and criterion have most important implications with regard to prototype load bearing testing, interpretation and evaluation of test results, and for representative and valid applications for predicting full-scale pressure-settlement performances of footings.

On the basis of this criterion, it is now possible more adequately and properly to interpret the results of the penetration tests of Fig. 2, where constant settlement lines for constant confinement ratios are also shown. For tests at 30 to 40 per cent relative density and confinement pressures greater than $\frac{1}{4}$ tsf, the points fall consistently and almost exactly on constant settlement lines. For smaller confinement pressures, the points depart from the constant settlement lines, and higher confinement pressures are necessary to limit settlements to the value indicated by a horizontal settlement line extending back to a confinement pressure curve. For example, for a confinement ratio of 0.08 and the constant settlement line extending back to a confinement pressure of $\frac{1}{16}$ or 0.0625 tsf, a confinement pressure of 0.10 tsf instead of 0.0625 would be required to limit settlements to 0.10 in. for a corresponding bearing pressure of 1.1 tsf. For the much higher bearing pressure levels in Fig. 2(*b*), a confining pressure of $\frac{3}{8}$ tsf is necessary to limit settlements to the values indicated by a constant settlement line. It therefore becomes evident that for confining pressures less than $\frac{1}{4}$ tsf in Fig. 2(*a*) and less than $\frac{3}{8}$ tsf in Fig. 2(*b*) shearing displacements still dominate the pressure-settlement performances of this very small scale 2-in. diameter penetration test, and that the confining pressure levels are not sufficient to prevent the escape by shearing of soil grains from beneath the edges of the bearing disk. However, confining pressures equal to and greater than these critical values are competent to limit shearing displacements from beneath the edges of the bearing disk, for example, $\frac{1}{4}$ tsf in Fig. 2(*a*) and $\frac{3}{8}$ tsf in Fig. 2(*b*), and are sufficient to provide complete confinement. The pressure-settlement performances thereafter fall into a prototype pattern from which representative and valid evaluations and applications can be made on an adequate and reliable basis. In Fig. 3 for a minimum width of footing of 12 in., a confinement pressure of $\frac{1}{8}$ tsf is sufficient to ensure prototype performances, and shearing displacements become negligible. It is for this reason that a 12-in. minimum diameter or width of footing is required for prototype bearing tests.

These facts and concepts have important implications for developing testing methods and procedures for both static and dynamic pressure-settlement performances of soils in order to ensure representative and valid test results that are capable of being interpreted, evaluated, and applied reliably in predicting full-scale performances.

In the laboratory vacuum methods provide the simplest and most effective means of applying prototype confinement conditions up to $\frac{1}{2}$ or $\frac{3}{4}$ tsf pressure, equivalent to 10 to 15 ft of soil surcharge. The marked influences of increase in relative density on pressure-settlement performances show that the soil must be replaced at the desired relative density for each test in prototype testing. The increase in relative density and the important stress conditioning influences caused by previous load-bearing tests always change the pressure-settlement performances sufficiently to make the test results non-comparable and unrepresentative and to appear to be better than they actually are.

In prototype field load-bearing tests, a surcharge of not less than 2 ft of soil ($\frac{1}{10}$ tsf) should be used, which is about equivalent to a footing placed its own thickness below the level of the immediately adjacent ground. Also, this degree of confinement is necessary to limit shearing displacements from beneath the edges of the minimum size 12-in. diameter plate. The bearing plate should be carefully and properly seated, and an initial settlement reading should be taken. The soil surcharge should be placed over the bearing plate and should be extended over an area of not less than 10 ft in diameter. A settlement reading should then be taken, and the loading test should be made in suitable load increments.

Prototype surcharges up to 5 ft of soil can be accomplished by placing a distributing fill 2 ft in thickness over an area not less than 10 ft in diameter. The additional surcharge up to 5 ft of soil can be made up by placing, for example, concrete blocks over the distributing fill. The blocks should be stacked in tiers with a separation all around of about 1 in. so that they will act independently and essentially as a uniform loading without any bridging effects. The surcharge should be in position at least one-half day in order for the surcharged underlying soil to come to stress and strain equilibrium under this confinement condition.

Thus essentially representative prototype load-bearing tests can be made that will satisfy the essential requirements and criteria for obtaining prototype pressure-settlement performances, which can be applied reliably for design purposes in predicting full-scale performances of footings.