pendant of the moisture content, but only of the structure. This would be the case for the permeability coefficient, according to the law of Darcy. If this is not the case, the curves can still be traced but for a determined moisture content. For instance, in the case of the cone-test, the curves will be traced as shown on fig. 3.

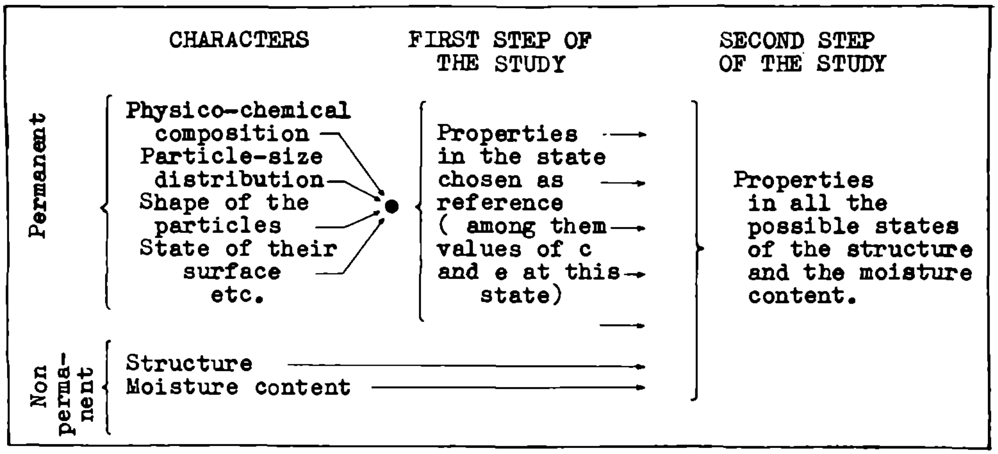
The case is particularly complicated in the study of the internal friction, for which are to be considered the modifications of structure in the sheared zone.

SUMMARY.

The study of the relations between the physical and mechanical properties of the soils and their permanent and non-permanent characters must rationally be conducted according to the scheme below:

The use of the proposed recording system permits to treat clearly and precisely the various questions which arise in this field, in the first step of the study as well as in the second.

As far as the first step of study is concerned, it facilitates the classification of soils based on the values of the compacity and of the moisture content in the state chosen as reference.

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## A METHOD FOR ESTIMATING THE LOAD-SETTLEMENT CHARACTERISTICS AND BEARING VALUE OF CLAYS AND CLAY-SOILS FROM UNCONFINED COMPRESSION AND TRI-AXIAL COMPRESSION TESTS.

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SYNOPSIS

A method is proposed for obtaining estimates of bearing value of soil for footings of bridges and buildings and for airport pavements from laboratory compression tests, which is based upon the concept of a Natural Restraint Coefficient. This involves theoretical interpretations and correlations of laboratory compression tests with load test data. The method is applicable to clays and clay-soils for which the initial stress conditions and strength are approximately constant with depth. Consideration is given to the Test Conditions to be followed in the laboratory compression tests. On the basis of this concept an estimated load-strain curve can be drawn from laboratory compression test data, that can be used directly for estimating the probable load-settlement characteristics, probable bearing value and ultimate failure conditions of full-scale footings.

Load tests have been considered to be the most direct and useful method for determining the load-settlement characteristics and bearing value of foundation soils, and have been used quite extensively for the design of foundations of buildings and bridges and for airport pavements where heavy plane loadings are anticipated. For airport pavements the load tests are full-scale tests, because the size of the bearing plates generally used are equal to the tire imprint areas of the larger planes. A rather large number of load tests may be required to provide adequate information for the design of pavements for a large important airport, where the soil conditions either in cut or fill may be expected to vary considerably over the site. The use of heavily loaded trucks, scrapers, etc. to provide sufficient reaction for the load test and the mobility of such equipment simplifies the problem of load testing to a considerable extent. The major problem of design of airport pavements is to interpret the results of the load tests on the subgrade soils into terms of the load-settlement characteristics and bearing value of the pavement-subgrade system in order to determine the thicknesses of pavement and base course needed to provide adequate support on the different classes of subgrades encountered in the site under the anticipated plane loadings 1).

Of equal or greater importance are the load tests on foundation soils, which are made to provide basic information for the design of spread footings for buildings, highway bridges, and the like. In this case the load tests are practically always small-scale tests. A minimum size of area of 2 feet by 2 feet is commonly required by Building Code Regulations. A total test load of 150 to 200 percent of the proposed design loading is usually applied. This load frequently runs into sizable figures. The handling of the necessary weight of pig iron, concrete blocks, sand bags, etc. is quite a problem. If the subsurface soil conditions are variable over the site of the proposed structure, it would be necessary to establish areas on the basis of the boring records of substantially similar subsurface soil conditions and to make a load test in each such area. The major problem of design is to extrapolate the results of the small-scale load test into terms of the load-settlement characteristics and bearing values for the large full-scale spread footings of the structure.

Because of the expense and time involved in making the load tests required to provide adequate information for the design of footings of buildings and bridges and of airport pavements, it would be desirable to have some method by which fairly reliable estimates of the probable load-settlement characteristics and probable bearing values of foundation soils can be obtained from laboratory unconfined compression tests or triaxial compression tests on undisturbed samples. A method is proposed for such purposes, which is based upon certain basic concepts, involving an interpretation of the theoretical load-settlement equations of the theory of elasticity and correlations of laboratory compression tests results with load test data. The method is applicable to clays and clay-soils for which the initial stress conditions and hence the strength properties are approximately constant with depth in the deposit, whether in the undisturbed state in a natural deposit or in some artificially compacted condition in an embankment.

First of all it must be emphasized that bearing value is not a simple inherent physical property of a soil such as the elastic limit or ultimate strength of steel, but must be determined by an analysis of the controlling physical factors in the particular situation and must be defined in terms of some <u>allowable settlement</u>, which experience has shown to be reasonable and safe for the particular type of structure and type of loading under consideration. Bearing value in any particular case is dependent upon the load-settlement characteristics of the deposit, which are governed not only by the strength properties and stress-strain relationships of the soil in its natural undisturbed state or in some artificially compacted condition, but also to a great extent by the size, shape, and flexibility of the loaded area, whether a bearing plate in a load test, an airplane tire, or a full-scale spread footing of a structure, and by the stratification of the deposit.

The interpretation of the theoretical load-settlement equation of Boussinesq, Eq. 1, is based upon certain fundamental concepts. The first concept is that the load-settlement relations for a footing or bearing plate in Eq. 1 are identical in form to the stress-strain relations in Eq. 2 obtained from a laboratory compression test.

Load-Settlement Equation. $$\Delta = \frac{c(1-\mu^2)}{E} p_f r_f \qquad (1)$$

Laboratory Stress-Strain Equation.

$$\varepsilon = \frac{\Delta}{h_c} = \frac{p_c}{E} \qquad (2)$$

Where Subscripts - f - applies to a footing or bearing plate
c - applies to a laboratory compression test

- $\Delta$ - settlement (in the same units as r)
- $\varepsilon$ - strain
- r - radius of footing, bearing area, or test specimen
- h - initial height of compression test specimen
- p - average pressure on footing or bearing plate in tons per square foot (tsf.)
- E - Modulus of elasticity of soil (tsf.)
- $\mu$ - Poisson's ratio, about 0.40 for many clays and clay-soils
- c - a coefficient depending upon the shape and flexibility of the footing or bearing area

The relationships of the physical factors entering these equations, which are involved in this interpretation, are illustrated in Fig. 1. The second concept, which follows from the first, is that the modulus of elasticity E of the soil in Eq. 2 obtained from a laboratory compression test is essentially identical to that of the soil in the natural deposit, which governs the load-settlement relations in Eq. 1. This parallels the situation for all structural materials and should be considered reasonable, where representative undisturbed samples of good quality are used for the compression tests, which have been obtained by carefully controlled methods from the immediate vicinity of the footing or load test site at a depth equal to about the diameter or width of the bearing are. Sample disturbance will tend to reduce the value of the modulus, E somewhat below that in the natural state, depending upon the degree of disturbance suffered by the sample during the sampling operation and subsequent preparation of the specimens for the laboratory compression tests.

These concepts provide a new approach in interpreting Eqs. 1 and 2. In the laboratory compression tests the height of the specimen is usually made at least equal to twice the diameter or four times the radius of the specimen - $h_c = 4\, r_c$ , as shown in Fig. 1a. In as much

## TABLE I

### Values of Coefficient, C.

| Shape of bearing area | Rigid Entire Area | Flexible, Uniform load Center | Edge | Corner | Average |
|---|---|---|---|---|---|
| Circular(radius, r ) | $\pi$/2=1.57 | 2 | 1.27 | | 1.70 |
| Square (half widt. b) | 1.76 | 2.24 | 1.56 | 1.12 | 1.90 |
| Rectangle x)(half least width, b ) | | | | | |
| Side Ratio 1.5 | | | | | 2.31 |
| 2.0 | | | | | 2.60 |
| 3.0 | | | | | 3.05 |
| 5.0 | | | | | 3.66 |
| 10.0 | | | | | 4.50 |

x) Coefficiente adopted from "Theory of Elasticity" by S. Timoshenko, p. 338, McGraw-Hill Book Company, New York, 1934.

as theory shows that 75 percent of the total settlement occurs within a depth equal to four times the radius of the bearing area, as shown in Fig. 1b, and because of the fact that the major part of the lateral bulging effect and displacements also occur within this depth, the load-settlement Eq. 1 is re-written in the form of a stress-strain relation similar to that in Eq. 2 for the compression test by assuming an effective height of a cylindrical mass of soil beneath the bearing area $h_f = 4\ r_f$ in Fig. 1b, as given in Eq. 3.

Load-Settlement Equation

$$\Delta = \frac{c(1-\mu^2)}{E} p_f \frac{4r_f}{4}$$

Load-Strain Equation (3)

$$\epsilon = \frac{\Delta}{4r_f} = \frac{c}{4}(1-\mu^2)\frac{p_f}{E}$$

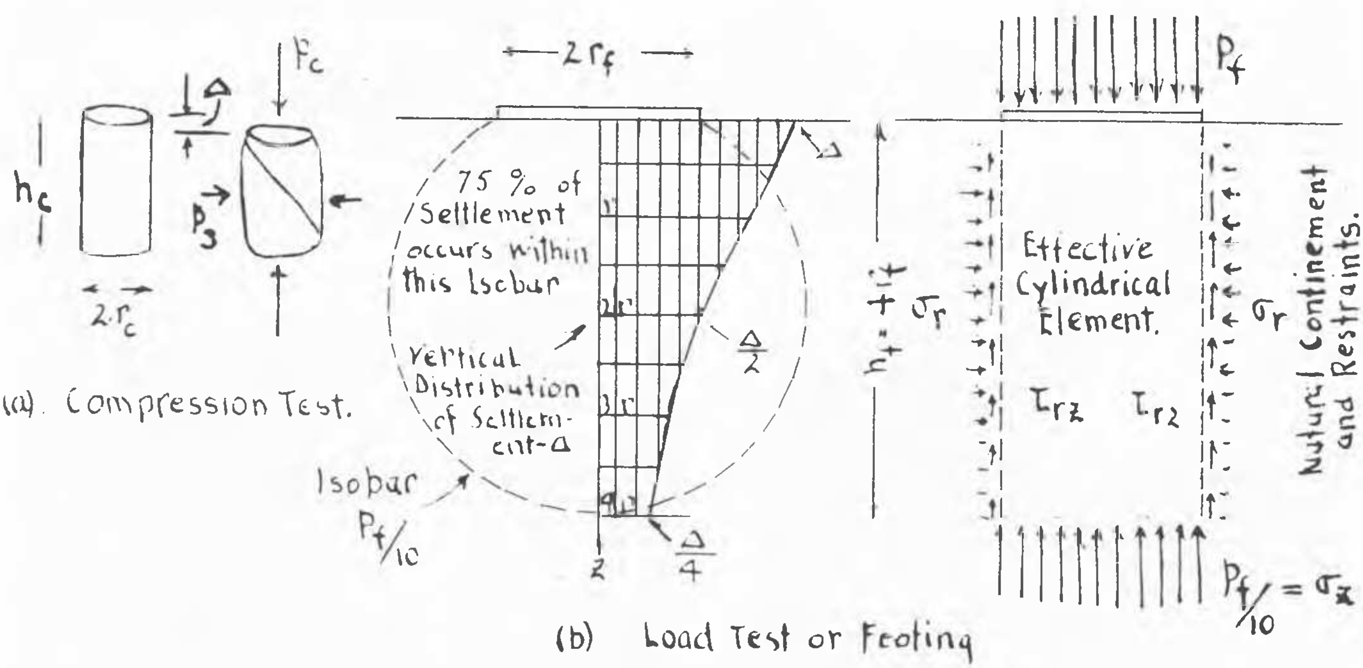
It follows that for equal strains the relation between the corresponding pressures for the footing, $p_f$ and for the compression test, $p_f$ from Eqs. 2 and 3 becomes

$$\frac{p_c}{E} = \left[\frac{c}{4}(1-\mu^2)\right]\frac{p_f}{E} \qquad (4)$$

$$\text{or}\quad p_f = \left[\frac{4}{c(1-\mu^2)}\right] p_c = R\ p_c$$

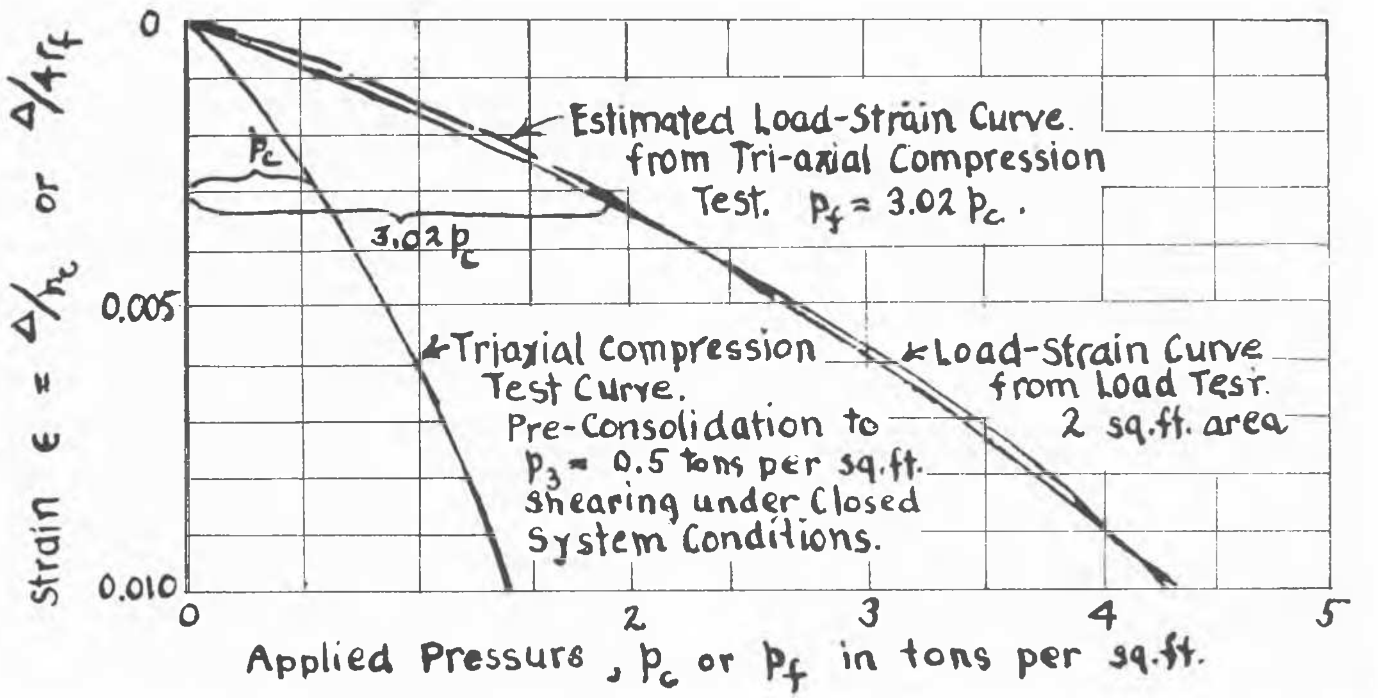
where $R = \left[\frac{4}{c(1-\mu^2)}\right]$ is termed the Natural Restraint Coefficient.

This leads to the third concept that the natural confinement and restraints of the nature of a passive resistance offered by the mass of soil surrounding the hypothetical cylindrical element beneath the bearing area have a most important influence on the load supporting capacity of the soil in natural deposits, as stated in Eq. 4; and accounts for the marked difference observed in Fig. 2 in the stress-strain relationships and in the apparent value of the modulus of elasticity E of the soil in the natural deposit as compared with that in the laboratory compression




Relative dimensions of footing and compression test.

FIG.1




Estimates of load-settlement relations of footings from compression tests.

**FIG.2**

test. The importance of the natural confinement and restraints to lateral and vertical displacements expressed in terms of the Natural Restraint Coefficient, R is evident from Eqs. 3 and 4, which show that for equal strains the pressure, $p_f$ for the footing loaded at the surface of the ground may be R times grater that the value, $P_c$ obtained in the laboratory compression test on specimens of the same soil. The values of the Natural Restraint Coefficient, as may be expexted, depend upon the shape and flexibility of the bearing area and upon the value of Poisson's ratio, which expresses the relation between the strains in the horizontal and vertical directions, as given in Table 2.

Table 2.

Values of the Natural Restraint Coefficient,

$$R = \left[\frac{4}{c(1-\mu^2)}\right]$$

For clays and clay-soils Poisson's ratio is taken to be of the order $-\mu = 0.40$

Center deflection or settlement of bearing area

| Shape of Bearing Area | Rigid | Flexible, Uniform load |
|---|---|---|
| Circular | 3.02 | 2.38 |
| Square | 2.71 | 2.13 |

On the basis of this concept of a Natural Restraint Coefficient an estimated stress-strain curve can be obtained from the laboratory compression test curve that can be used directly for estimating the probable load-settlement characteristics and the probable ultimate failure conditions of a footing or of a load test bearing plate by multiplying the laboratory compression test stresses by the appropriate value of the Natural Restraint Coefficient for a sufficient number of strain values to define the curve over the range of values desired, as shown in Fig. 2., where the estimated load-strain curve is superimposed on the actual load-strain curve of a load test on a 2 square foot circular bearing area. It is to be noted that the agreement between the actual and the estimated load-strain curves is reasonably good. Similar agreements have been found by such an analysis of data presented by other investigations.

The probable safe bearing value corresponding to some selected safe or reasonable limiting value of settlement can then be estimated directly from the estimated load-strain curve for any size of bearing area, since the strain for a given size of bearing area is given by Eq. 3

Circular Bearing Area - $\Delta = 4r_f\varepsilon$

$$\qquad (5)$$

Square Bearing Area - $\Delta = 4p_f\varepsilon$

The method is applicable to individual footings on clays and clay-soils for which the initial stress conditions in the deposit and hence the strength properties are approximately constant to a depth of at least 2 and possibly 4 times the diameter or width of the bearing area, whether in the natural undisturbed state or in some artificially compacted condition. Experience has shown that deposits of clays and many deposits of clay-soils conform reasonably closely to this condition, and hence follow reasonably closely the theoretical load-settlement Eq. 1. The estimates of the probable settlement from the load-strain curve by Eq. 3 and 5 is that which occurs almost immediately after the loading, due to elastic and plastic displacements, but does not include the subsequent settlement due to gradual consolidation of the clay, either directly beneath the bearing area or at greater depths under the long-time loading. The estimates of the settlement due to consolidation must be added to the above immediate settlements to obtain the probable total settlement to be expected ultimately.

An analysis is now made of the probable ultimate failure load to determine if it is in reasonable agreement with theory and with experience. This follows directly from the three concepts stated previously and involves the question whether the assumed effective height of the cylindrical element directly beneath the bearing area in Fig. 1b is of the right order of magnitude for use in the proposed method or whether some other value should be used. Analyses of the failures of footings on clay by a number of investigators have shown that the failure load, varies from about 4 to 6 times the maximum shearing strength, of the clay, which is given by Equation 6 for the unconfined compression test.

Unconfined Compression Test

$$s_{(max)} = \frac{p_{c(max)}}{2} \qquad (6)$$

Depending upon the value of the Natural Restraint Coefficient, which is a function of the shape and flexibility of the bearing area and of Poisson's ratio, the probable failure load is given by Eq. 7 and is tabulated in Table 3.

$$\text{Failure load } p_{f(max)} = Rp_{c(max)} = 2Rs_{(max)} \qquad (7)$$

Table 3.

Failure Conditions - Probable Values of $p_{f(max)}$ expressed in terms of $2\,RS_{(max)}$

| Shape of Bearing Area | Rigid | Flexible, Uniform load |
|---|---|---|
| Circular | 6,04 $S_{(max)}$ | 4.76 $S_{(max)}$ |
| Square | 5,42 $S_{(max)}$ | 4.26 $S_{(max)}$ |

Since these values are of the order of magnitude indicated by theory and by experience for the ultimate failure load, and since there is a reasonable agreement between the estimated and the actual load-strain curves in Fig. 2, they give support to the concepts and method proposed.

The agreement attained between the actual load-strain curve and that estimated from a laboratory compression test curve both as to magnitude of the stress-strain values, the shape of the curve, and the ultimate failure load, will depend upon whether representative undisturbed samples of good quality are used for the compression tests and upon the testing technique. The best and least disturbed samples obtainable are those carefully cut out by hand from a test pit in the immediate vicinity of the footing or load test site at a depth about equal to the diameter or width of the bearing area. Sample disturbance will tend to increase the strains and to reduce the ultimate strength, depending upon the degree of disturbance suffered by the sample during sampling operations and subsequent preparation of the specimens for the laboratory compression tests. Careful consideration should be given to the Test Conditions, which should be set up to approximate as closely as possible the natural conditions of initial stresses and of drainage, that may be expected to obtain.

In the natural state clay acts temporarily as an almost incompressible material under closed system conditions with little or no free drainage possible. Under such conditions the immediate settlement under a rapid loading is due almost entirely to lateral displacements of elastic and plastic character at constant volume, and the settlement due to consolidation is considerably delayed. If the natural undisturbed consistancy of the clay or clay-soil is Hard or better, that is, a shearing strength greater than 2.0 tons per square foot (See Eq. 6), both the immediate settlement due to lateral displacements and the subsequent settlement due to consolidation would be relatively small under loadings not exceeding possible 30 percent of the ultimate load. In such a case the unconfined compression test may be expected to give reasonably satisfactory information. If on the other hand the natural consistency is Stiff or Medium Hard (shearing strength 0.5 to 1.0 and 1.0 to 2.0 tons per square foot, respectively) the immediate settlements and the subsequent consolidation settlements, particularly for the stiff consistency, may be considerably larger and reach objectionable magnitudes. In this case both tri-axial compression and consolidation tests would be needed in order to estimate the probable immediate and the total subsequent settlement. First, it is necessary to determine as closely as possible the initial state of stress on the clay. The value of the initial vertical stress, $p_V$ can be estimated from the consolidation test curve (the so-called preconsolidation pressure). The first test condition in the tri-axial compression test is to re-establish the probable initial lateral stress, $p_H$ on the specimen by pre-consolidating the specimen with open system conditions under this lateral pressure, $p_3 = p_H$, which may be estimated approximately form Eq. 8.

$$p_3 = p_H = \left(\frac{\mu}{1-\mu}\right) p_V \qquad (8)$$

where $p_3 = p_H$ - is the approximate lateral pressure to be applied in the triaxial compression test.

$p_V$ - is the initial vertical stress, usually larger than the overburden stress.

$\mu$ - is Poisson's ratio, approximately 0.40 for clays.

After the tri-axial compression test specimen is fully pre-consolidated under this lateral pressure, the tri-axial compression test is run under Closed System conditions with drainage cut off, as the second test condition. A comparison of the results of the triaxial compression test with those of an unconfined compression test will indicate whether or not the proposed method is applicable or not. If the stress-strain characteristics and the ultimate strength of the clay or clay-soil are found to be only a small function of the lateral pressure, the proposed method is applicable, but if they are found to be a rather large function of the lateral pressure, where the clay-soil is quite silty or sandy in character or is not saturated, the method may not be directly applicable, because the initial stresses are not constant with depth, but a function of depth and Eq. 1 does not apply directly.

Comparisons of the estimated and actual load-strain curves obtained from laboratory compression tests and load tests, and actual observations of the settlements of full-scale footings will indicate the realm of validity of the theory and the types and consistencies of clays and clay-soils for which reasonably close estimates may be expected.

REFERENCE.

1) D.M. Burmister - The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways, Proc. Twenty-Third Annual Meeting of the Highway Research Board, November, 1943.

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