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EXPERIMENTAL STUDY OF BEARING CAPACITY IN LAYERED CLAYS

UNE ETUDE EXPERIMENTALE DE LA FORCE PORTANTE DES COUCHES D'ARGILE

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SYNOPSIS The ultimate bearing capacity of foundations resting on clay subsoils has been investigated for the case of a stiff layer overlying a soft layer, and the case of a soft layer overlying a stiff layer. The studies have been based on model tests using circular and strip footings, and using a range of layer thicknesses and clay strengths. The analyses have been carried out in terms of total stresses.

For stiff layers overlying soft layers, failure occurs by the footing punching through the top layer, and with full development of the bearing capacity of the lower layer. For the second case, failure occurs mainly by squeezing of the soft layer between the footing and the stiffer lower layer, with some interaction between the layers as the strength ratio approaches unity. Equations and charts giving the appropriate modified bearing capacity factors are presented, derived from the empirical relationships obtained in the experimental studies.

INTRODUCTION

A fairly common problem in foundation engineering is that of assessing the ultimate bearing capacity of subsoils consisting of two or more layers of clay having significantly different strength and deformation characteristics. Where there are only two such layers, each of which possesses fairly uniform properties, the problem may be resolved into a consideration of two cases, a relatively thin stiff layer overlying a thick soft layer, and a thin soft layer overlying a thick stiff layer. An example of the first type can be found where a stiff crust in the upper portion of a softer deposit has been formed by such agents as desiccation and weathering. An example of the second type may be found where soft post-glacial clays overlie older, stiffer glacial till deposits.

The bearing capacity of homogeneous soils has been the subject of extensive studies. These have yielded results which either are correct for the stated assumptions, such as the Prandtl solution for a perfectly plastic foundation material, or contain approximations which have been found by field experience or model studies to be sufficiently accurate to permit their use with confidence. This situation does not obtain to the same degree in the case of non-homogeneous subsoils. The solutions which have been advanced (e.g. Button, 1953 and Tcheng, 1956) are, for the most part, either admittedly very approximate, or unverified by tests or analyses of full-scale foundations, or else contain assumptions which do not appear to be entirely justifiable.

The purpose of this paper is to present the results of a series of model footing tests carried out on two-layer clay foundations. The problem contains many variables, and the limitations of the study may be seen from the following points, which set forth the scope of the experimental work.

1. All studies were carried out in terms of the undrained shear strength of the clay, using total stress or "$\phi = 0$" analyses.

2. Studies were confined to surface loadings, using rigid strip and circular footings with rough bases.

3. Only one type of clay was used. Therefore, although the strength of the clay was varied, the deformation properties remained essentially constant. This is acceptable in so far as the behaviour of the clay under load, particularly at failure, can be characterized by the undrained shear strength alone. However the extent to which the failure load is governed by the actual stress-deformation behaviour of the soil may impose a limitation on the applicability of the results.

PROPERTIES OF THE CLAY

The clay used in the footing tests was an inorganic clay of medium plasticity, containing about 35 per cent minus 2 micron size, predominantly illite. Failure strains of this material in the remoulded state were too high to permit a satisfactory evaluation of the tests. Accordingly, slaked lime was added to the clay in the proportion of 2 per cent of the dry weight of the soil, in order to make it more brittle.

It is generally appreciated that the undrained strength as measured in triaxial tests does not necessarily predict the undrained strength which may be mobilized in field stability problems. Some of the reasons for disagreement are in situ anisotropy and rotation of principal planes during shear, plane strain effects, relation between peak and post-peak strengths (the progressive failure problem), time dependency of undrained strength, and laboratory sample size and degree of disturbance.

These factors also affect model tests, although to a lesser degree, and required special tests and careful evaluation in the analysis of the footing tests reported herein. In Fig. 1 and Table 1 are summarized some of the strength behaviour properties of the test clay. All compression test specimens were trimmed from blocks of clay compacted in the same way as the footing test clays. The effect of the application of confining pressures to the undrained strength may be noted. Also, unconfined (U) samples failed by vertical splitting or along a single shear plane; undrained triaxial (UU) test samples failed on one, or perhaps two, well defined shear planes. Plane strain tests showed no significant difference in undrained strength or failure strain. Time effects were negligible within the range of testing times which were used. Sample size effects and disturbance due to trimming of specimens were judged to be minimal.

APPARATUS AND TEST PROCEDURE

The raw clay was blended with the required amounts of water and hydrated lime in a Muller mixer, which employs a pair of heavy rolling wheels to knead the clay, and was found to be satisfactorily thorough in mixing. The clay was then packed into the footing test boxes, described below, by hand, using a kneading action, and in

Sample Inclination degrees	$\frac{C_u}{C_{u(vert)}}$	e_f %	$\frac{e_f}{e_{f(vert)}}$
0	1.0	1.60	1.0
45	1.08	0.94	0.59
60	1.10	0.50	0.31
90	1.20	0.75	0.47

Table 1. Results of Undrained Triaxial Anisotropy Tests - Average Values

this way a degree of saturation of about 95 per cent was obtained. The clay was placed in lifts having a compacted thickness of 0.5 to 0.75 inches (ca. 1 to 2 cm), with the interface between the lifts being made deliberately uneven, to reduce bedding effects. The interface between the upper and lower layers was levelled, then "roughened" with a series of grooves so that the clay layers "interlocked" and slipping at the boundary was prevented. The interface was sealed as well as possible against moisture migration by a coating of liquid rubber latex. The clay was cured for 7 days prior to testing.

The test box for strip footing tests was about 27 inches long by 6 inches wide by 12 inches deep, and is illustrated in Fig. 2. It was constructed with 0.5 inch thick (1.31 cm) plate glass sides, which correspond to the intermediate principal planes in a strip footing test. The plate glass was chosen for its rigidity and for minimum friction when greased. Test boxes for the circular tests were square with dimensions approximately 21 x 21 x 10 inches. Footings were made of mild steel, 3 inches wide by

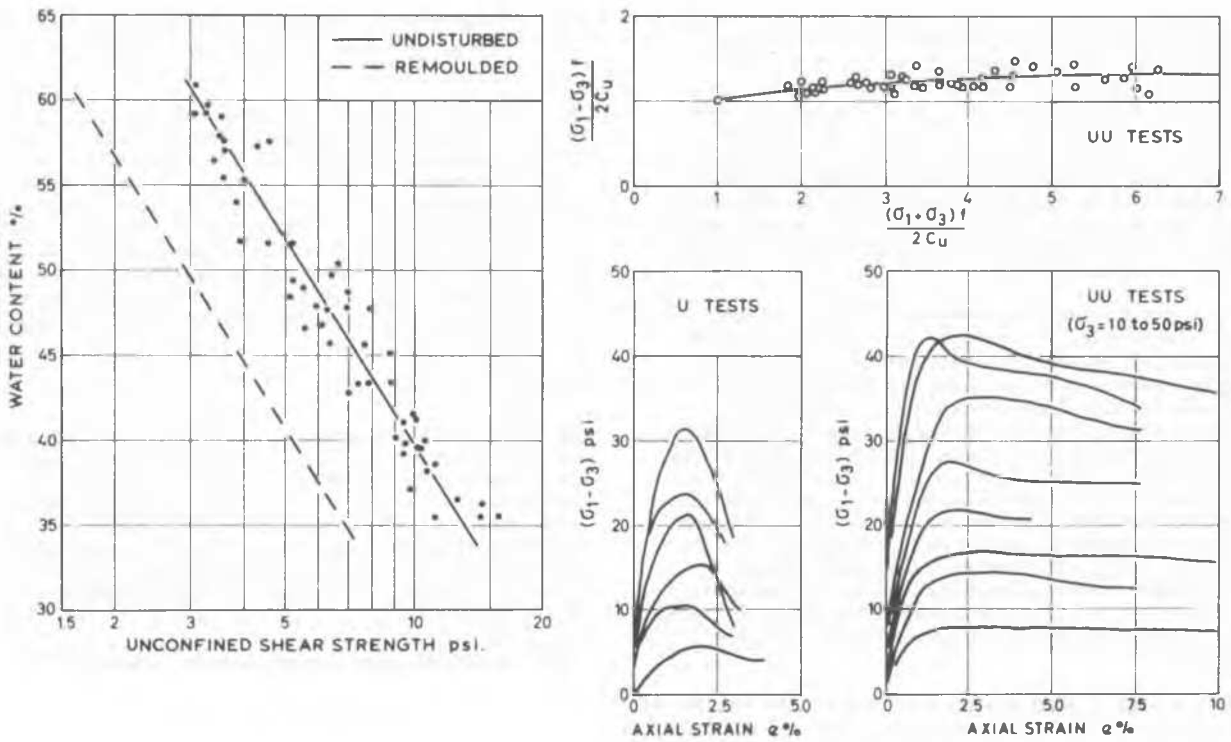


Fig. 1. Undrained shear strength behaviour of the lime stabilized test clay.

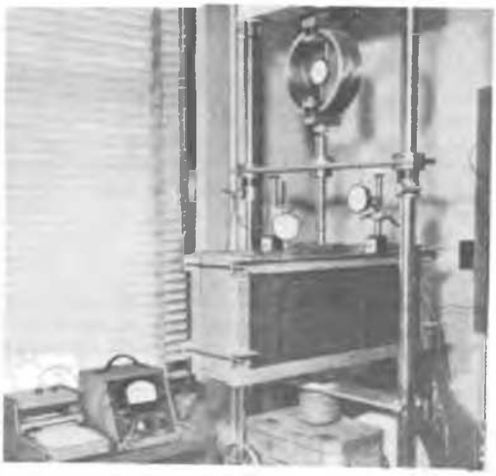


Fig. 2. Typical arrangement for strip footing tests.

6 inches long (7.6 x 15.2 cm) for the strip, and 3 inches diameter (7.6 cm) for the circular footings. The bases were made rough by glueing a layer of medium quartz sand on them.

For the tests, the footings were threaded onto a loading ram which operated through a lubricated ball bearing guide, to ensure that they did not rotate, and the direction of load remained vertical. Loads were generally measured through a proving ring; the amount of guide friction was checked, however, using a load cell just above the footing in some of the tests, and it was found to be negligible.

Tests were run at a rate of 0.15 inches (0.4 cm) penetration per minute; this relatively fast speed was chosen to minimize moisture migration during loading. The mode of failure and slip planes were observed both during the test and afterwards, when the clay was being removed from the box.

FOOTINGS ON HOMOGENEOUS CLAY

A number of strip and circular footing tests were carried out on homogeneous beds of clay. Their chief purpose was to determine the average undrained shear strength, C, mobilized in the clay, so that the inter-relationship of the factors mentioned earlier could be assessed. It was assumed that the classical Prandtl solution for the ultimate bearing capacity, q_f, of strip footings on a rigid-plastic material, which gives a bearing capacity factor, N_c, of 5.14 in the well known expression

$$q_f = C\, N_c \qquad (1)$$

should be obtained in these tests. The corresponding value of N_c for rough circular footings was taken as 6.05, after the analysis of Eason and Shield (1960). The calculated correction factors to be applied to the unconfined shear strength were then found to be 1.10 and 1.21 for the strip and circular footings, respectively. These values were consistent with estimates made of the sum of the individual corrections for anisotropy, confining pressure, progressive failure, etc.

In addition, the general load-settlement behaviour and failure zones of the homogeneous foundations were observed. Typical load-settlement curves are shown in Fig. 3 (a). For strip footings the indicated failure zones consisted of a base wedge with sides inclined at an average angle of 48 degrees to the horizontal, zones of radial shear extending to a depth of 0.85 B, where B was the footing width, at a distance of 0.8 B from the vertical plane of symmetry, and finally, zones of horizontal splitting in the "passive Rankine zones" which extended to the free surfaces. This rupture figure does not agree with that found for a perfectly plastic material $\emptyset = 0$. This disagreement is to be expected, since the clay, although considered in theory to be a perfectly plastic material with $\emptyset = 0$, in fact possesses internal friction, which governs the shape of the rupture figure (Skempton 1948). At the same time, however, neither does the observed rupture surface agree with that postulated for a weightless cohesive-frictional soil, which is governed by the Hvorslev effective angle of internal friction, for this clay between 20 and 25 degrees. The shape of the observed surface suggests a mobilized friction angle of about 10 degrees, which is in general agreement with the arguments put forth by Bjerrum and Kenney (1967) that in brittle soils failure is due to a structural breakdown which occurs before the friction is fully mobilized. The zone of horizontal splitting, adjacent to the free surfaces on either side of the footing, is in turn in agreement with the observed splitting type of failure of the horizontally trimmed unconfined test specimens.

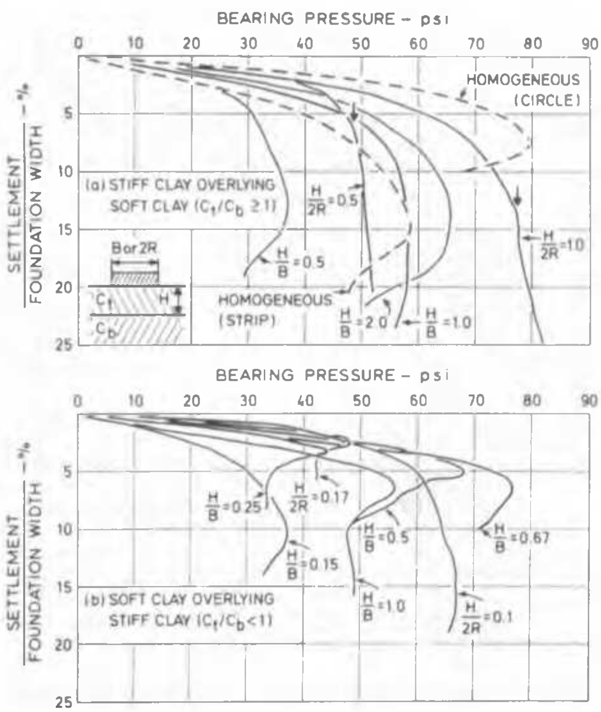


Fig. 3. Typical load-settlement curves of strip (B) and circular (2R) footings, for different top layer thicknesses, H.

From these observations it may be seen that the pattern of failure beneath a footing is a function of the physical mode of rupture of the clay. This mode of rupture, in brittle clays at least, is strongly dependent on the structure of the clay, the failure mechanism of which is not adequately defined by conventional Mohr-Coulomb concepts of cohesion and friction.

FOOTINGS ON STIFF CLAY OVERLYING SOFT CLAY

This configuration consisted of two clay layers, each of constant strength. The ratio of top layer thickness to width or diameter of footing, H/B or H/2R, varied from 0.5 to 3.0, and the strength of the top layer was up to 4 times the strength of the bottom layer. These limits were taken to encompass the probable range of values encountered in practice. Typical load-settlement curves for strip and circular footings are shown in Fig. 3 (a). The point of failures is clearly indicated; for the whole test series, failure occurred at an average penetration of 16 per cent of the footing width in the strip footing tests and 7 per cent in the circular footing tests.

The behaviour of both types of footings was qualitatively similar, and hence both were analyzed in the same way. A variable correction factor, j, derived from the homogeneous test results, was applied to the unconfined shear strength of each layer. A depth factor was also used, to allow for the effect of penetration, s, on the bearing capacity. This factor varied from 1.0 for pure punching, as discussed below, to $(1 + 0.4\,s)$ for fully developed failure zones in the upper layer. The bearing capacity factor, N_m, defined with respect to the unconfined shear strength of the top layer, C_t, is thus given by

$$q_f = j\,C_t\,N_m\,(1 + ks) \tag{2}$$

where q_f is the bearing pressure at failure, and both j and k are variable. If the clay were an ideal rigid plastic material, $j\,C_t$ would be constant and s would be zero, giving the following equations, analogous to equation (1)

$$\text{strip} \quad q_f = C_t\,N_{ms} \tag{3}$$

$$\text{circular} \quad q_f = C_t\,N_{mc} \tag{4}$$

Values of N_m for both strip and circular footing tests are shown in Fig. 4. The numbers accompanying each data point are the thickness-width ratios at failure. It may be noted that different values of j were used in preliminary analyses, and although these changed the calculated values of N_m and C_b/C_t, they did not change the general pattern of the results to any appreciable degree; thus the

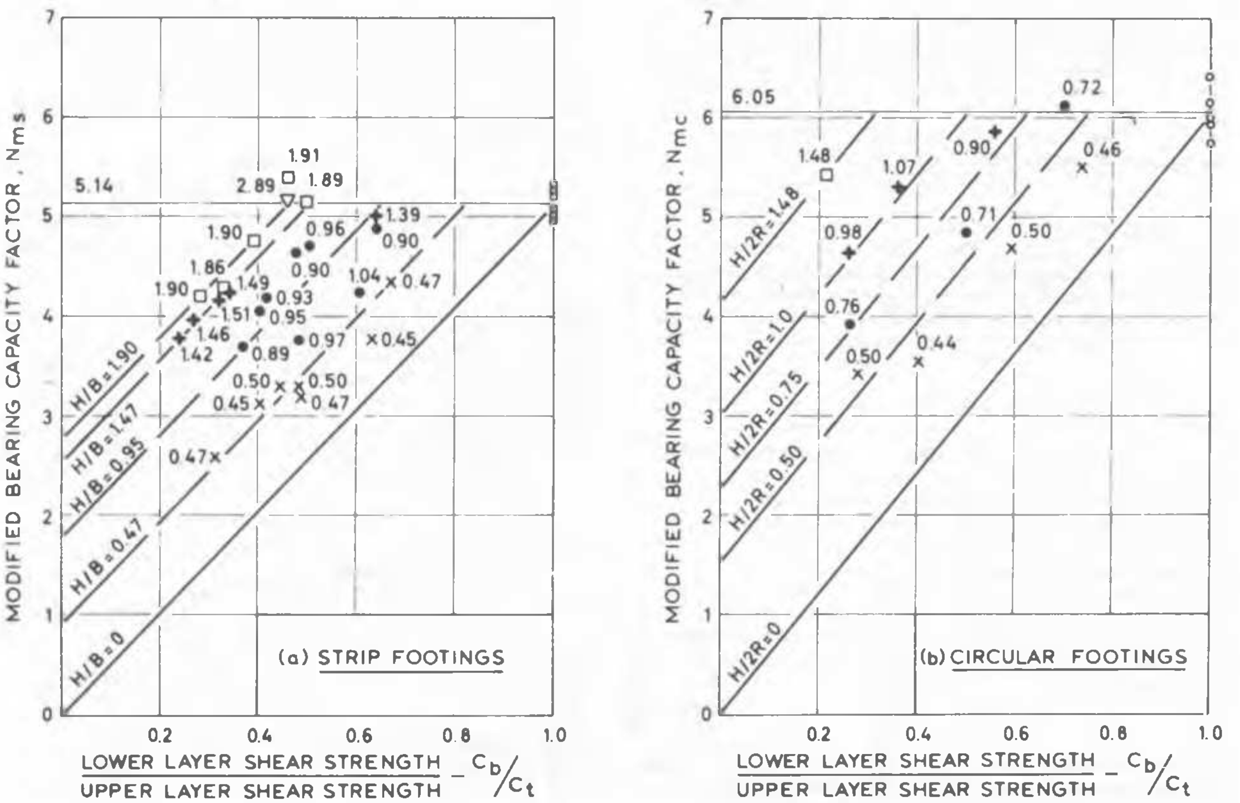


Fig. 4. Experimental values of modified bearing capacity factors for stiff clay overlying soft clay.

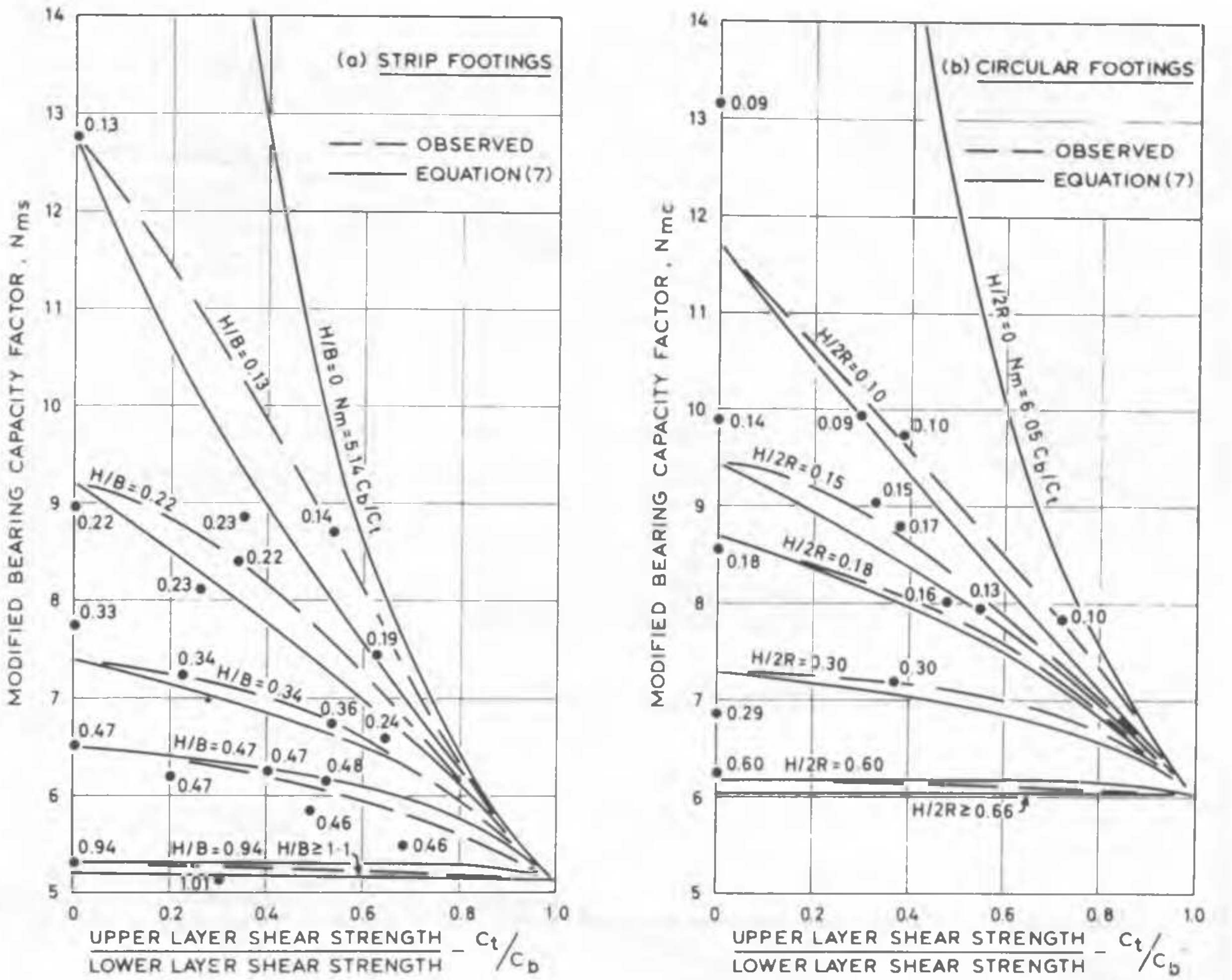


Fig. 5. Experimental values of modified bearing capacity factors for soft clay overlying stiff clay.

equations given below are not sensitive to moderate changes in the correction factor, which was admittedly not rigourously evaluated.

For both sets of tests, the simplest curves which may be fitted to the data are a series of straight lines, which yield the following equations :

$$\text{Strip footings, } N_{ms} = 1.5\frac{H}{B} + 5.14\frac{C_b}{C_t} \leq 5.14 \qquad (5)$$

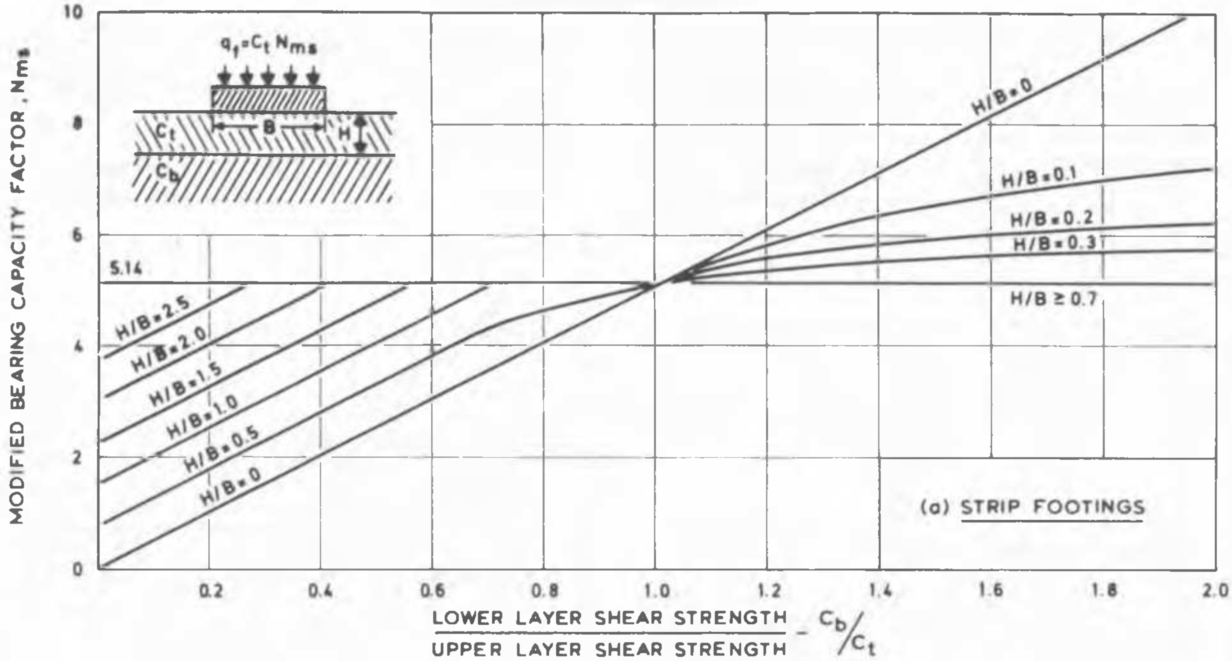
$$\text{Circular footings, } N_{mc} = 3.0\frac{H}{2R} + 6.05\frac{C_b}{C_t} \leq 6.05 \qquad (6)$$

The use of these equations will lead to an over-estimate of about 10 per cent in values of N_m in the range $C_b/C_t \geq 0.7$; the more correct values are indicated by the non-linearity of the curves in these regions in Fig. 6. If these equations are interpreted in terms of physical behaviour, it is seen that the first term is representative of some type of punching shear through the stiff top layer, with the second term suggesting full mobilization of the bearing capacity of the lower layer. This intrepretation is supported by observations made of the failure zones developed in the tests.

For simple shear punching, the first terms in equations (5) and (6) should be 2H/B and 4H/2R, respectively; therefore the full value of punching shear does not seem to be developed.

FOOTINGS ON SOFT CLAY OVERLYING STIFF CLAY

The test configurations were similar to those used in the preceding section. The ratio of the strengths of the layers varied all the way from unity, the homogeneous case, to infinity, where the lower layer is an unyielding, rough and rigid base. Typical load-settlement curves for the strip and circular footings are shown in Fig. 3 (b). Again, the point of failure is clearly indicated; average failure penetrations for the complete series of strip and circular footing tests were 6 and 3 per cent of the footing width, respectively. In almost all cases, failure appeared to be confined to the soft upper layer only, and the observed rupture zones indicated squeezing of the soft clay laterally, under the footing, and a splitting type of failure of the clay below the free surfaces on either side of the footing. A number of tests on thin cylinders and blocks were



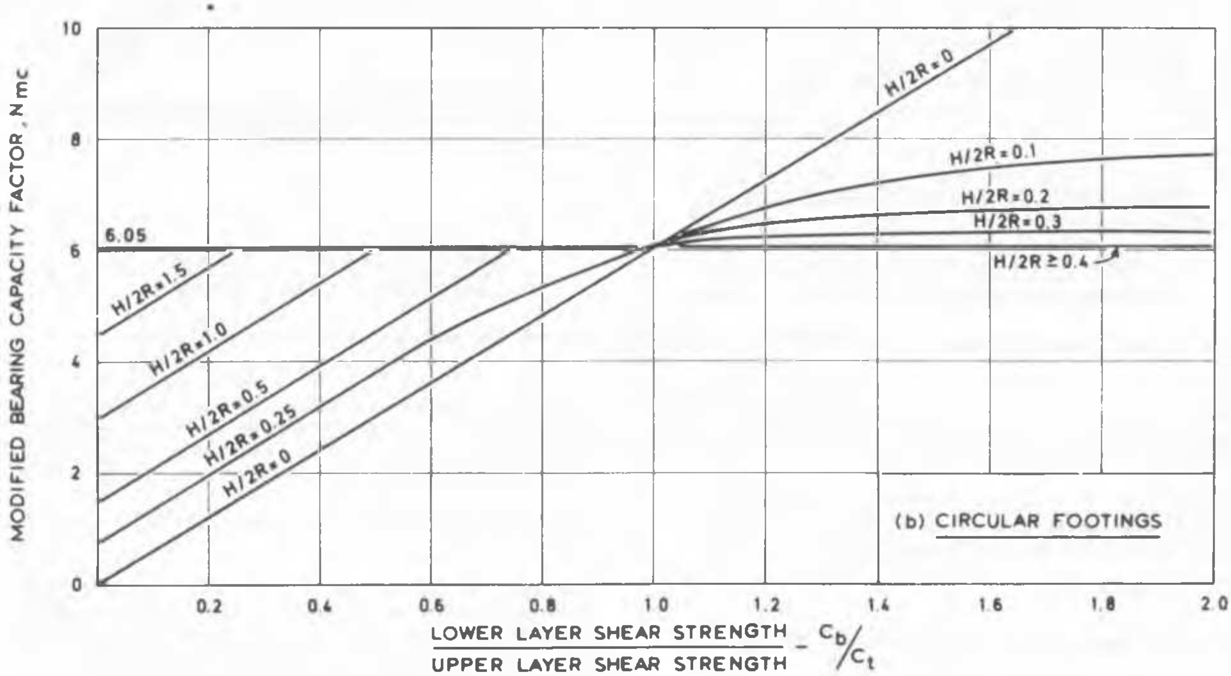


Fig. 6. Relation between modified bearing capacity factors and shear strength ratio, C_b/C_t, for strip and circular footings.

also carried out to assist in the evaluation of the footing tests.

The analysis of the results depended again on an assessment of the shear strength of the clay. The effects of anisotropy, confining pressure and progressive failure were variable, depending on the configuration of the problem, and a precise evaluation of individual cases was not possible. Consideration of all factors led to an average correction factor of the same magnitude as that found in the homogeneous footing tests. The error in the estimated strength associated with this procedure was estimated to be less than $\pm$ 5 per cent.

The modified bearing capacity factor N_m was found from equation (2) for both types of footing, and the results are shown in Fig. 5. The dashed lines represent the graphical best fit. The solid lines have been derived by using an interaction formula of the Rankine type, which is of the form

$$N = \frac{N_1 \; N_2}{N_1 + N_2} \qquad (7)$$

In this equation N_1 and N_2 are the equations of two separate phenomena, which interact in such a way that the equation which relates both phenomena is given by N. This type of empirical equation has found successful application in the interaction between axial compression and buckling in structural columns. In the present application, N_1, represents the equation for N_m obtained from tests where the lower layer is a rough, rigid base, (equations (8) and (9), below), and N_2 represents the equation for N_m where a Prandtl rupture failure involves both the upper and the lower layer. The maximum variation between N_m values given by the best fit and the Rankine formula curves is about 10 per cent.

The results of the tests having a rough, rigid base for the lower layer gave the following values for N_m:

Strip Footings, $B/H \geq 0.9$

$$N_{ms} = 4.14 + 1.1 \; B/H \qquad (8)$$

Circular Footings, $2R/H \geq 1.5$

$$N_{mc} = 5.05 + 0.66 \, (2R/H) \qquad (9)$$

These equations yield values of N_m considerably higher than the corresponding theoretical relationships for a perfectly plastic material (Meyerhof and Chaplin, 1953), which are

$$N_{ms} = 4.14 + 0.5 \; B/H \qquad (10)$$

$$N_{mc} = 5.05 + 0.33 \, (2R/H) \qquad (11)$$

The substitution of epicycloidal slip surfaces for the cycloidal surfaces used in the theoretical derivation increases the second term in equations (10) and (11) by about 30 per cent, which does not produce agreement with the experimental equations. It may therefore be concluded that in the intensely stressed upper layer beneath the footings, the undrained shear strength is not an accurate measure of the shearing resistance. Hence, it is considered that, for purposes of design, the theoretical equations (10) and (11) which are in fact lower limits, should be used in the interaction formula, equation (7). The solution of these equations gives the values for N_{mc} and N_{ms} in Fig. 6.

RECTANGULAR FOOTINGS

The similarities of the modes of failure observed in strip and circular footing tests allows the use of the results to develop equations for rectangular footings. Following Meyerhof, (1951), it is assumed that below the central part of the rectangle the bearing stresses are the same as below a strip, and that at the ends the bearing stresses are essentially those under circular footings. Then it can be shown that the bearing capacity factor for rectangular footings, N_{mr}, is given by

$$N_{mr} = N_{mc} \; B/L + N_{ms} \, (1 - B/L) \qquad (12)$$

N_{mc} and N_{ms} are obtained from Fig. 6 for the appropriate values of C_b/C_t, and H/B. For homogeneous foundations, equation (12) reduces to that given by Skempton (1951),

$$N_r = 5 \, (1 + 0.2 \; B/L)$$

CONCLUSIONS

The results of the investigation are summarized in the charts given in Fig. 6, which may be used in evaluating the bearing capacity of layered clay foundations. The results are essentially experimental, and therefore are rather strongly affected by the characteristics of the test clay. However, it is believed that a conservative interpretation has been put upon the findings, which should permit their use in the range of moderately brittle, slightly sensitive clays.

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