Studies of Shear Strength and Bearing Capacity of some Partially Saturated Sands

Études de la Résistance au Cisaillement et de la Capacité Portante de Certains Sables Partiellement Saturés

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Summary

The results of some shear tests on partially saturated loose silty sands are presented and the importance of careful laboratory procedure is pointed out. Curves of field loading tests are given together with comparisons of actual and predicted bearing capacities. The use of a 'settlement ratio' is suggested for a clearer understanding of the behaviour of different sized footings on various soils.

Sommaire

Cette communication présente les résultats de certains essais de cisaillement sur des sables d'alluvion meubles partiellement saturés et souligne la nécessité d'apporter un soin méticuleux aux recherches de laboratoire. On donne des courbes d'essais de charge *in situ* avec des comparaisons de capacités portantes théoriques et réelles. On propose de recourir au 'taux de tassement' afin de mieux comprendre le comportement des semelles de différentes dimensions sur des sols de nature différente.

Introduction

The results presented in this paper were obtained in the course of investigations into the causes of distress in structures situated in different localities but all founded near the surface of partially saturated loose silty sands. Excessive foundation movements had occurred and studies of shear strength and ultimate bearing capacity were required, although the sudden settlement, or 'collapse', due to saturation of the soil might have been the most important factor involved. The sudden settlement and tests for the recognition of this phenomenon have been described by JENNINGS and KNIGHT (1957).

Site A

Description of soil—The soil encountered was a fine silty sand subjected to sufficient wind action to cause partial rounding of the grains and containing iron oxide which imparted a reddish colour. The material had about 15 per cent silt and 15 per cent clay with a porosity of 43 per cent in the undisturbed state.

Laboratory tests—In order to obtain an idea of the bearing capacity, shear box tests were carried out on undisturbed samples trimmed from blocks taken in the field. Complete drainage was possible during the tests as the normal rate of strain of 0·036 in. per minute was found to be slow enough, from considerations of the consolidation characteristics of the soil. Confirmation of full drainage was afforded by parallel tests at 0·012 in. per minute which gave very similar results.

The following procedures were adopted to study the effect of saturation on shear strength: (1) drained shear box test at natural moisture content; (2) drained shear box test performed after application of required load and gradual inundation of the sample; (3) drained shear box test performed after first inundating the sample carefully while under the light load of the top plate only, and then applying the required load; (4) drained test carried out in the triaxial apparatus, after first applying the required ambient pressure and then allowing saturation under a small head.

The results of these tests are presented in Fig. 1, a normal shearing resistance diagram, and indicate that the angles ϕ for the procedures (2) and (3) for saturation differ by 5 degrees. The lower angle of shearing resistance of 33 degrees would be anticipated from a sand in such a loose state of packing. A possible explanation of this discrepancy is that the structure of the sand depends on conditions prior to the actual test (BISHOP and ELDIN, 1953). There appears to be little difference, however, between the triaxial test (4) and the shear box test (3).

Typical stress, strain and volume change relationships are given in Fig. 2, showing the additional settlement after saturation of the sample.

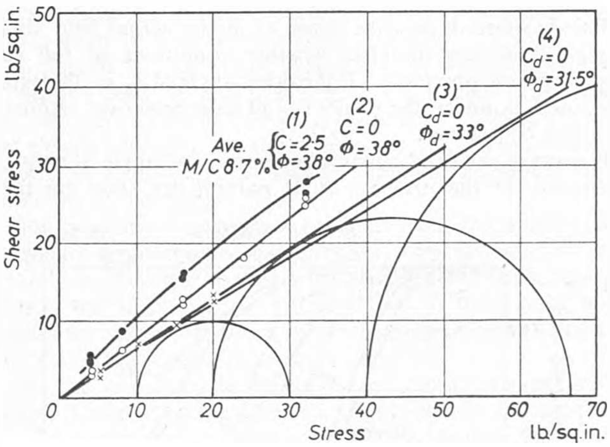

Fig. 1 Results of shear tests—Site A
Résultats des essais de cisaillement—Emplacement A

Bearing capacity tests—Several load tests were performed in the field using a concrete block 2 ft. square founded at a depth of 2 ft., giving the same D/B ratio as the actual foundations of the structure. The results are shown in Fig. 3. The curves AN and LN represent tests carried out after rain over the area, when the bulk density was about 108 lb./cu. ft., the water table being at great depth. The bearing capacities in each case were about 3300 lb./sq. ft. The curves AS and LS represent similar tests carried out after artificially flooding the area and allowing 15 hours for natural drainage before testing—bearing capacities were 2250 lb./sq. ft. and 1500 lb./sq. ft. respectively.

Calculations based on Terzaghi's equation

$$q_f = 1{\cdot}3\,.\tfrac{2}{3}c\,.N_c' + \gamma D N_q' + 0{\cdot}4\,\gamma B N_\gamma'$$

give, in cases AN and LN using $c_d = 0$, $\phi_d = 33°$, $\gamma = 108$ lb./cu. ft.

$$q_f = 108 \times 2 \times 12 + 0{\cdot}4 \times 108 \times 2 \times 8$$
$$= 3280 \text{ lb./sq. ft.}$$

In cases AS and LS if $\gamma' = 58{\cdot}4$ lb./cu. ft. is used

$$q_f = 58{\cdot}4 \times 2 \times 12 + 0{\cdot}4 \times 58{\cdot}4 \times 2 \times 8$$
$$= 1775 \text{ lb./sq. ft.}$$

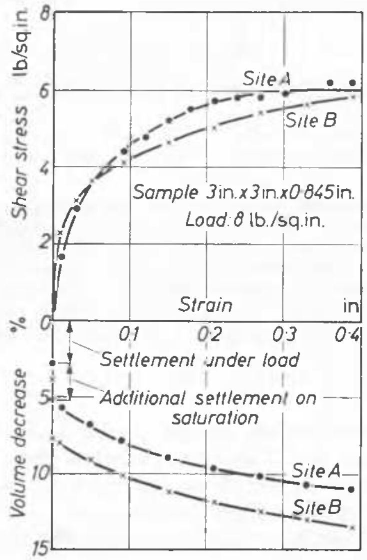

Fig. 2 Typical stress/strain curves in shear box tests
Contrainte caractéristique contrainte/déformation des essais dans la boîte à cisaillement

This is more or less the same as in the actual field values though it is very doubtful whether conditions of full submergence were obtained. If the other angle of $\phi_d = 38$ degrees were used, however, the values would have been over-estimated by about 60 per cent.

If average values of apparent cohesion and angle of shearing resistance for the material in its natural dry state are used,

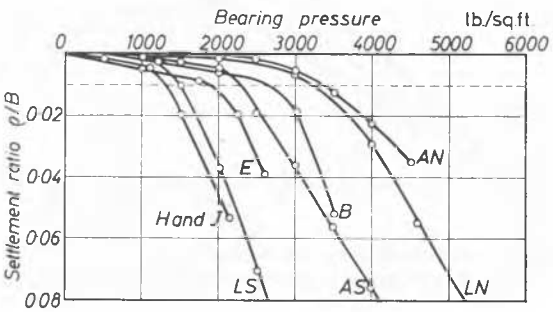

Fig. 3 Field loading tests
Essais de charge *in situ*

$c = 2{\cdot}5$ lb./sq. in., $\phi = 38$ degrees, $\gamma = 102$ lb./cu. ft. and applied to footings 5 ft. square and about 5 ft. deep, $q_f = 22{,}000$ lb./sq. ft. Thus the design pressures of 4000 lb./sq. ft. would appear to be quite safe, giving an anticipated settlement of $\frac{1}{8}$ in., i.e. an amount proportional to the anticipated settlement at failure. On inundation, however, shear failure might occur since the calculated bearing capacity would then be $q_f = 4350$ lb./sq. ft. In the first structure to show signs of distress cracking of reinforced concrete beams occurred when the bearing pressures were between 1000 lb./sq. ft. and 3000 lb./sq. ft.

A safe bearing pressure allowing for conditions of submergence and a factor of safety of 3 would have been $q_a = 1{,}450$ lb./sq. ft. which would have pointed to the use of a raft foundation, making provision for a basement, or founding at greater depth using under-reamed piles. For another structure to be built in the vicinity it is possible to found on a hard stratum at about 20 ft. depth and it was considered economical to excavate by hand and cast square footings and a column before backfilling.

Site B

The material here consisted of fine, loose, orange-coloured aeolian sand, with a typical dry density of 86 lb./cu. ft. at 8·7 per cent moisture content. A series of shear box tests was carried out in order to get an idea of the bearing capacity. The following procedures were adopted: (1) drained test at natural moisture content; (2) drained test after first applying the load and then slowly inundating the sample; and also (3) and (4) undrained triaxial test at natural moisture content, with measurement of pore pressure.

The results are given in Fig. 4. It was noted that the shearing resistance was decreased by about 20 per cent on saturation.

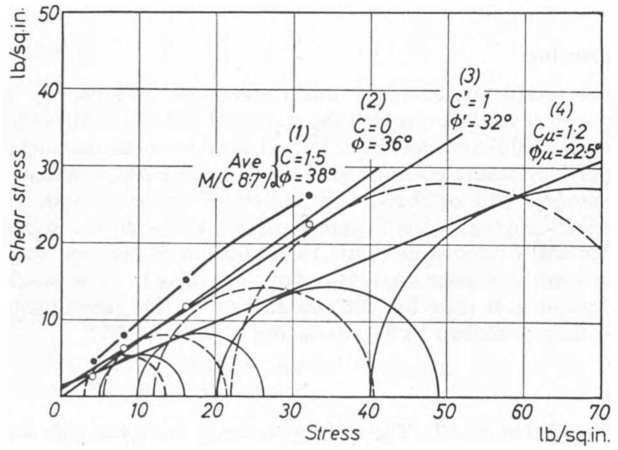

Fig. 4 Results of shear tests—Site **B**
Résultats des essais de cisaillement—Emplacement **B**

Further, the angle ϕ' was lower by 3 degrees than ϕ given by procedure (2) above, whereas the two angles should be very nearly equal in a loose sand as the rate of volume change at failure is practically zero in both tests.

Again, an explanation of this discrepancy might lie in the structure of the sand before testing, as the effect of air content would have been to impart an apparent cohesion if full saturation were not obtained under procedure (2).

In regard to drained tests on partially saturated soils, it should be noted that the applied stresses are not effective stresses, as in the saturated condition, because of the existence of additional capillary stresses. In all cases a curved line of shearing resistance was observed in slow tests on the natural soil, which suggested that the capillary stresses varied for each single test. To verify this observation an undisturbed sample 3 in. in diameter and $1\frac{1}{2}$ in. high contained in a cutter was placed on a sintered glass base, through which the soil suction was measured, and loads were applied through a perforated top plate. It was found that the soil suction increased with increasing load or density, as indicated in Fig. 5, the moisture content remaining constant. While this effect appears rather strange at first sight, it can be correlated with information given by CRONEY and COLEMAN (1954) where curves of pF/moisture content are presented for high and low densities.

It is interesting to note that the value of soil suction obtained is of the same order as the horizontal difference between curves (1) and (2) of Fig. 4. A direct comparison at equal applied stresses cannot be made, however, because the moisture content of the suction test sample was lower than that of the shear strength sample and also because there was some change in density during initial shear strain.

The effect of soil suction on shear strength may be an important consideration, particularly in the case of partially saturated clays. Also, the density/soil suction relationship at constant moisture content deserves further study, though a different technique from that mentioned above may be required even for low values of pF as the experimental difficulties were extreme and only limited information was obtained.

Bearing capacity tests—A number of plate-bearing capacity tests were performed in the field using different sizes of plate, but no correlation between ultimate failure and size of plate was found. The natural moisture content, however, varied from about 6 to 16 per cent over the site so that the lack of correlation was not surprising in view of the large change in shear strength found during laboratory testing. The results are given in Fig. 3. Two of these tests, marked H and J, were performed after the material in the field had been soaked for 16 hours. The ultimate bearing capacity in each case was

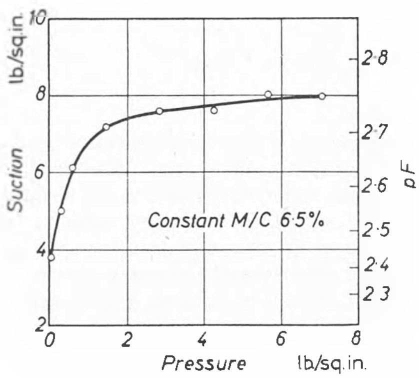

Fig. 5 Soil suction/pressure relationship at constant m/c

Aspiration du sol en fonction de la pression avec teneur en eau constante

about 1300 lb./sq. ft., and if the submerged density is used in calculation with $\phi_d = \phi' = 32$ degrees

$$q_f = 53{\cdot}4 \times 2 \times 10 + 0{\cdot}4 \times 53{\cdot}4 \times 1{\cdot}4 \times 6$$
$$= 1245 \text{ lb./sq. ft.}$$

This is very fair agreement, but if an angle of ϕ of 35 degrees were used the result would have been over-estimated by about 35 per cent. It is concluded, therefore, that the shear testing technique of applying a load before saturation should not be used, and that a value of ϕ_d (with initial saturation before loading), or ϕ' in loose sands, be taken instead.

If an average value were taken for $c_d = 1{\cdot}5$ lb./sq. in. and $\phi_d = 38$ degrees for the natural soil

$$q_f = 10{,}000 \text{ lb./sq. ft.}$$

In the field, values of up to 9000 lb./sq. ft. were obtained for plates 1 ft. or 2 ft. square. The value under these conditions, however, should not be used in design unless inundation due to rain, which is often aggravated by drainage from any structure, can be prevented.

Interpretation of Field Tests

The interpretation of field bearing capacity tests is always a difficult matter, particularly when local shear failure occurs and the ultimate load is not clearly defined. In Fig. 3 some typical results of field loading tests on the two sites are plotted in the relationship $q : \rho/B$ and it will be noted that 'failure' may be taken to occur at a ratio of $\rho/B = 0{\cdot}01$ in nearly every case. Because the field moisture contents varied under every load test it was not possible to correlate the breadth B with load intensity at failure q_f, although such correlation is an accepted fact as regards any homogeneous sand.

The use of this 'settlement ratio' has been found convenient by SKEMPTON (1951) and MEYERHOF (1953) in the study of bearing on clays with respect to prediction of settlement, but from their work and that of others it appears that the ratio is significant in regard to ultimate failure as well. For the clays mentioned a typical value at failure is 6 per cent. For other materials this point has already been brought out by WILSON (1950), who stated that for deep footings in loose sands at failure ρ/B was about 1·5 per cent and for dense sands 3·5 per cent.

This ratio may give some idea of the shearing strain that occurs beneath a footing and it is noteworthy that the present test results give a value of about 1 per cent which appears rather low and may be a characteristic of this type of loose deposit.

Conclusions

(1) Several instances of distress in structures founded on partially saturated sands in South Africa and Rhodesia have resulted from the effects of inundation due to rain or concentrated drainage. While these soils appear to be very firm in their natural fairly dry state it appears that there is a great loss of strength on saturation.

For the design of foundations in such instances the most severe conditions should be taken into account since it is difficult to prevent some ingress of rain water or the accumulation of moisture under a covered surface. Thus, in laboratory testing, the samples should be saturated and drained tests performed. It is thought that most of the windblown deposits encountered will be permeable enough for drainage to occur during shear in the field—coefficients of permeability were measured at 10^{-3} cm/sec or more for Site A.

In regard to the shear parameters to be obtained from laboratory tests for any interpretation it is considered that if the coefficient of permeability, k_w, exceeds 10^{-3} cm/sec a drained test should be used. On the other hand, if k_w is less than 10^{-5} cm/sec an undrained test should simulate closely conditions obtaining in the field.

For loose sands the angle ϕ' obtained from undrained triaxial tests may be used as equivalent to ϕ_d. In general, the most severe condition will be that of saturation, when the submerged density should be used in calculation or, if the ground water conditions are known more accurately, the treatment given by MEYERHOF (1955) should be employed.

(2) The laboratory technique for saturating an undisturbed sample is important and from the results of this investigation it appears preferable, in a shear box test, to saturate the samples before applying any load. In the field, saturation will usually occur after loading, but the laboratory procedure will give the worse condition. Further, the validity of assuming full submergence in regard to density calculations may be doubted, but this will give the most severe case.

(3) It is useful, when interpreting the results of bearing capacity tests, to plot the information in the form of $q/\rho/B$. With experience it may be found that the criterion of failure is easier to define in the light of a limiting 'settlement ratio' than as the point at which the settlement curve becomes steep. For loose sands ρ/B appears to be of the order of 1 per cent, while for other materials it may approach 6 per cent.

(4) While the above comments refer to shear failure this may not be the criterion for design of foundations as the amount of

settlement should also be considered. In the loose sands described, which appear to be unusual materials, the collapse of soil structure on saturation may be the governing factor and prediction of this sudden settlement should be based on the results of consolidation tests.

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References

Bishop, A. W. and Eldin, A. K. G. (1953). The effect of stress history on the relation between ϕ and porosity of sand. *Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, p. 100

Croney, D. and Coeman, J. D. (1954). Soil structure in relation to soil sunction (*pF*). *J. Soil Sci.*, **5**, No. 1.

Jennings, J. E. and Knight, K. (1957). The additional settlements of foundations due to a collapse of structure of sandy subsoils on wetting. This volume

Meyerhof, G. G. (1955). Influence of roughness of base and ground-water conditions on the ultimate bearing capacity of foundations. *Géotechnique*, **5**, No. 3, 227

— and Murdock, L. J. (1953). An investigation of the bearing capacity of some bored and driven piles in London clay. *Géotechnique*, **3**, No. 7, 267

Skempton, A. W. (1951). The bearing capacity of clays. *Proc. Building Research Congress*, Vol. 1, p. 180

Wilson, G. (1950). The bearing capacity of screw piles and screwcrete cylinders. *J. Inst. Civ. Eng.*, **34**, No. 5