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EDITORIAL ADVISERS

The Editor of *Geotechnical Engineering* feels that, in order for the Journal to achieve and maintain a high standard in its technical papers, a need exists for advice and guidance on editorial policy from a number of learned and experienced men in the geotechnical field. As from the next issue, therefore, a small panel of Editorial Advisers will fulfil this need. The following well-known personalities have kindly consented to help in this way.

Prof. E. De Beer

Prof. E.H. Davies

Prof. T. William Lambe

Prof. Victor F.B. de Mello

Prof. J.K.T.L. Nash

Prof. Ralph B. Peck

With the guidance of such an august body of Editorial Advisers, it is felt that this journal will have the best possible chance of achieving its aim of excellence, and the Editor looks forward to a long and fruitful association.

UNDRAINED SHEAR STRENGTH CHARACTERISTICS OF SAND

ALAGAI AH THURAIRAJAH† and BRIAN LELIEVRE*

SYNOPSIS

The undrained deformation behaviour of a saturated sand was investigated under non-cavitating conditions using strain-controlled triaxial tests. Compression and extension tests were carried out, with back pressure, over a wide range of void ratio and consolidation pressure. At small strains, loose samples reached a peak deviator stress followed by a decrease in deviator stress to a minimum value. With further strain the deviator stress increased with shear strain at an increasing rate and a negative pore water pressure was developed. This unstable behaviour was absent in dense samples.

The limiting equilibrium states for the sand determined from undrained compression and extension tests are compared to investigate the failure criterion applicable to sand. The Mohr-Coulomb criterion describes failure much closer than the extended von Mises or the extended Tresca criterion. The angle of internal friction determined from extension tests for any void ratio is a few degrees higher than that determined from compression tests. Thus, the Mohr-Coulomb criterion underestimates the strength of soil in many field problems.

INTRODUCTION

Most of the stability problems and deformation problems encountered in practice in saturated cohesionless soils occur under drained conditions, since the pore water pressure developed with change in stress is dissipated almost immediately due to the high permeability of the medium. Hence, the strength and deformation behaviour of cohesionless media under drained conditions have been extensively studied using different types of shear apparatus. The standard triaxial (axi-symmetric) apparatus, in which the intermediate principal stress is equal to one of the other two principal stresses, has been used by many research workers to investigate whether the widely used Mohr-Coulomb failure criterion, which considers that the intermediate principal effective stress has no influence on strength, is applicable to soils.

If the Mohr-Coulomb failure criterion is valid, the angle of internal friction, ϕ' , determined from triaxial compression tests, in which the intermediate principal stress is equal to the minor principal stress, will be equal to ϕ' determined from triaxial extension tests, in which the intermediate

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Discussion on this paper is open until 1 November 1972.

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principal stress is equal to the major principal stress. The majority of investigators (BISHOP & EL-DIN, 1953; CORNFORTH, 1964; KIRKPATRICK, 1957) have concluded that the value of ϕ' determined from triaxial extension tests is equal to that determined from triaxial compression tests for any state of packing, though some (GREEN & BISHOP, 1969; ROSCOE et al, 1963) have reported that ϕ' in extension is slightly greater than ϕ' in compression. Drained shear tests carried out in a shear apparatus where the intermediate principal stress lies between the major and minor principal stresses, however, show that ϕ' is influenced by the intermediate principal stress and is a few degrees higher than ϕ' determined from standard, drained triaxial compression tests (GREEN & BISHOP, 1969; KIRKPATRICK, 1957; KO & SCOTT, 1968).

Because of sudden loading, as in earthquakes and blasts, a mass of saturated, cohesionless soil can deform and fail in such a way that the pore water pressure developed as a result of the imposed load is not dissipated during the short duration of load application. The deformation behaviour of this medium under undrained conditions needs to be investigated to analyse such problems. The research that has been carried out into the undrained shear characteristics of sand, especially in loose states of packing, is rather limited (SCHOFIELD & WROTH, 1968; SEED & LEE, 1967), although the undrained strength of sand when subjected to cyclic loading has been studied recently to a greater extent.

The behaviour of a fine sand during undrained shear tests has been studied herein using triaxial (axi-symmetric) compression and extension tests. The tests described were performed without allowing any cavitation to occur in the sand from the negative pore water pressures developed during the shearing process.

MATERIAL

The material tested was the portion of fine Ottawa sand passing a No. 60 sieve (0.25 mm) and retained on a No. 200 sieve (0.074 mm). 21% of this material passed a No. 100 sieve (0.149 mm). The sand grains had a specific gravity of 2.65. A fresh sample of sand was used for each triaxial test, since shearing might have caused fracturing of some sand grains.

EXPERIMENTAL PROGRAM

The triaxial samples tested were 2 in. in diameter and 4 in. in height. The conventional end platens of the triaxial sample were replaced with enlarged stainless steel platens of 2.25 in. diameter, which had been carefully ground

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and polished. The friction at the end platens was reduced by placing a 0.01 in. thick rubber disc on each platen with a thin layer of silicone grease between the rubber disc and the polished surface of the platen. This arrangement produced a more uniform deformation throughout the length of the sample (BISHOP & GREEN, 1965; ROWE & BARDEN, 1964). Each sample was free to drain from the bottom end only through a 0.5 in. diameter porous stone fixed to the centre of the polished end platen.

The error in measured axial load due to friction between the loading piston and the top of the triaxial cell in the conventional apparatus was eliminated by measuring the load internally using an electrical resistance strain gauge type load cell fixed to the bottom end of the loading piston. The pore water pressure was measured at the base of the sample using a *Dynisco* pore water pressure transducer.

In triaxial samples where membrane penetration can occur (NEWLAND & ALLELY, 1959), the penetration increases with increase in the pressure difference between the cell fluid and the pore fluid. Hence, a conventional undrained test at constant cell pressure becomes a partially drained test due to the change in volume produced by membrane penetration (THURAIRAJAH & ROSCOE, 1965). In this series of tests, the membrane penetration was reduced to a negligible value by testing a fine sand and using 0.015 in. thick rubber membranes.

Saturated triaxial samples were obtained in a loose state of packing by spooning freshly boiled sand into a sand former (specially designed to accommodate the enlarged end platens) filled with water. Denser samples were obtained by tamping and mechanical vibration. This method of forming samples produced void ratios which varied from about 0.7 in the loose state to about 0.5 in the dense state.

Undrained compression and extension tests were conducted at constant cell pressure on samples consolidated under isotropic pressures varying from 2 to 80 lb/in.². Dense sand samples undergo reductions in pore water pressures during undrained tests which lead to cavitation. Once water vapour or dissolved air starts coming out of solution, the tests become partially drained tests. In the undrained tests discussed in this paper, all samples were tested with back pressures and with a cell pressure of 110 lb/in.²; a test was stopped when the excess positive pore water pressure in the sample became zero. Wherever cell pressure is given subsequently in this paper, the effective value in excess of the back pressure is quoted.

The tests were carried out under strain-controlled conditions at an axial deformation rate of 0.003 in./min.

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UNDRAINED COMPRESSION TESTS

During a triaxial compression test, the deviator stress, q , given by $(\sigma'_1 - \sigma'_3)$, where σ'_1 and σ'_3 are the axial and radial effective stresses respectively, is estimated on the assumption that the sample remains a right circular cylinder. In this investigation, this assumption was observed to be reasonable, except for tests on loose sand consolidated under a low cell pressure when some bulging occurred in the middle portion of the sample under the large axial strains required for failure. For example, a sample of loose sand (void ratio = 0.685) consolidated under a cell pressure of 2 lb/in.² was subjected to a conventional axial strain of 16.9 % before the excess pore water pressure was reduced to zero, while a sample of dense sand (void ratio = 0.525) consolidated under the same pressure needed only a conventional axial strain of 3 %. Hence, the deviator stress estimated from a test on loose sand was subject to some error because of the area correction applied.

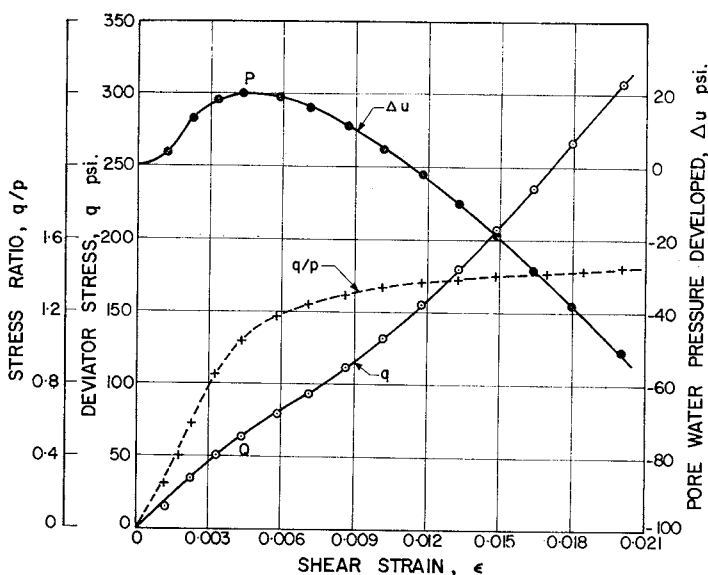


Fig. 1. Undrained compression test on dense sand.

Stress-Strain Relationship

Figure 1 shows the variation of the deviator stress, q , the pore water pressure developed during shearing, Δu , and the stress ratio, q/p , with the natural shear strain ϵ (equal to the natural axial strain for undrained tests) for a typical dense sample of sand (void ratio = 0.526) consolidated under a cell pressure of 60 lb/in.² p is the mean principal effective stress given by $(\sigma'_1 + 2\sigma'_3)/3$. For the choice of the stress and strain parameters used here, 104

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see the paper by ROSCOE & POOROOSHASB (1963). During the initial stage of the test on dense sand, a positive pore water pressure is developed with increase of q , and the rate of development of Δu with ϵ becomes zero at a shear strain corresponding to point P in Fig. 1. A reduction in pore water pressure is developed with further application of shear strain, and the excess pore water pressure continues to decrease at an increasing rate until it becomes zero. During this stage of the test, q increases with ϵ at an increasing rate, showing that under undrained conditions the material is progressively locking. The point Q on the q - ϵ curve is a point of inflexion and occurs when the positive pore water pressure developed is a maximum, corresponding to point P on the Δu - ϵ curve.

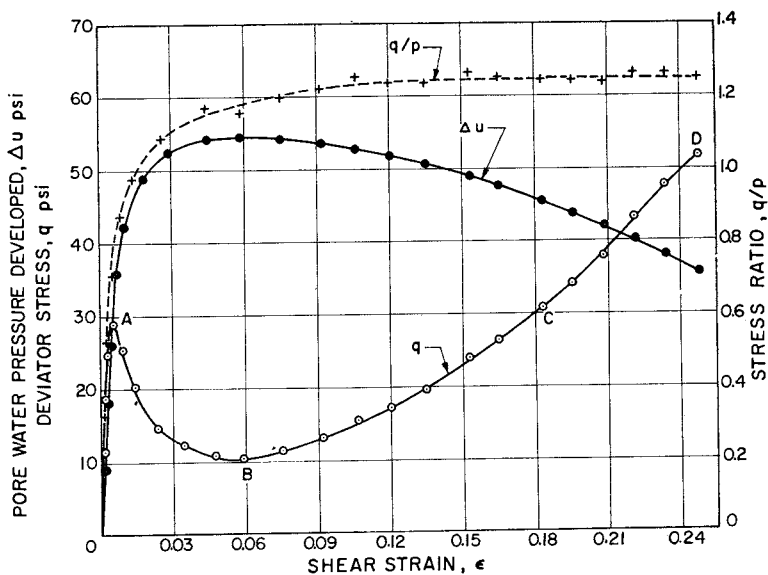


Fig. 2. Undrained compression test on loose sand.

The variation of q , Δu and q/p with ϵ for a typical loose sample (void ratio = 0.741), also consolidated under a cell pressure of 60 lb/in.², is presented in Fig. 2. q increases with ϵ up to point A when unstable conditions are reached. With further increase in ϵ , q decreases to a minimum value at point B and again increases at an increasing rate along the curve BCD . If the test were done under stress-controlled conditions instead of the strain-controlled conditions used here, the sample would deform suddenly once the state A were attained, and would reach the state C with a large shear deformation. q/p is found to increase continuously with ϵ to reach a steady value. The sample develops positive pore water pressure during the initial stage of the test, and the rate of change of Δu with ϵ becomes zero when q

reaches the minimum value at point *B*. A reduction in pore water pressure is developed in the sample with further shear strain. The excess pore water pressure continues to decrease even at the maximum shear strain of 25% (conventional axial strain of 22%).

Samples of sand at all states of packing developed positive pore water pressures during the initial stages of the tests before developing negative pore water pressures with further strain; the rate of development of negative pore water pressure with ϵ was greater for denser samples. The critical state (ROSCOE et al, 1958), at which further shear strain applied does not produce any change in p , q or Δu during an undrained test, was not attained for any of the tests discussed herein; these were discontinued either when the excess pore water pressure became zero or when the conventional axial strain reached a value of 22%.

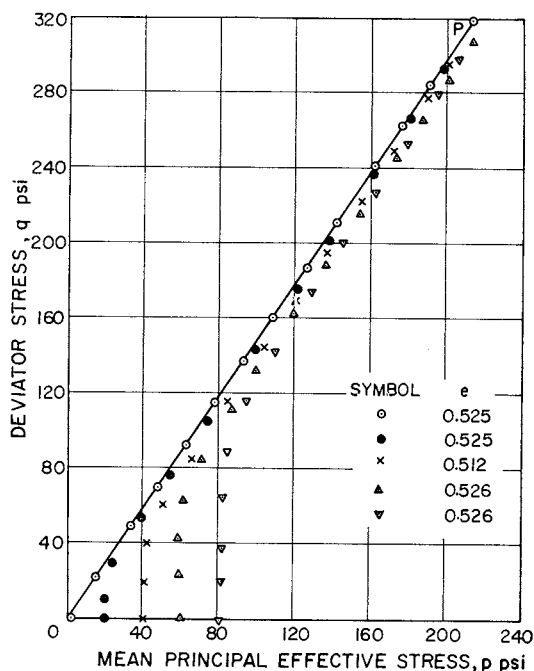


Fig. 3. Stress paths for undrained compression tests on dense sand.

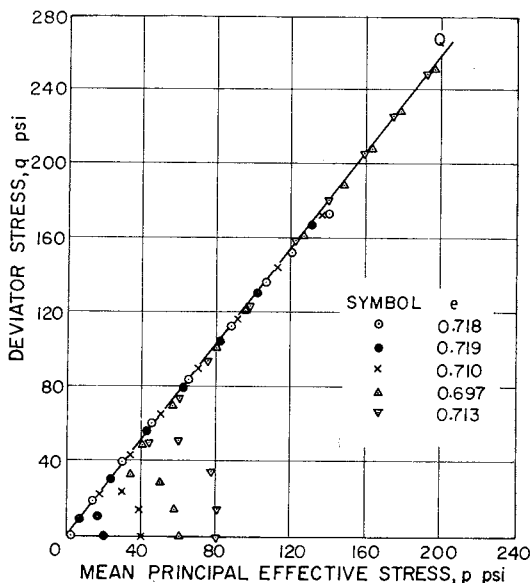


Fig. 4. Stress paths for undrained compression tests on loose sand.

Effective Stress Paths

The effective stress paths on the (p , q) plane of undrained compression tests carried out on dense samples consolidated under different cell pressures are presented in Fig. 3. The effective stress path for the sample consolidated under the small pressure of 2 lb/in.² lies on the straight line *OP* passing

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through the origin, while the stress paths for samples with higher consolidation pressures rise almost vertically during the initial stage of the tests and tend to approach the line OP at higher values of p . It is not possible for the sand at this void ratio to have a stress state point lying above the line OP under undrained conditions. This line will be referred to as the *limiting state line* corresponding to that particular void ratio.

Figure 4 shows the effective stress paths on the (p, q) plane for undrained compression tests on loose sand consolidated under different cell pressures. Here again, the effective stress path of the sample consolidated under 2 lb/in.² lies on the straight line OQ passing through the origin. The effective stress paths of the samples with higher consolidation pressures follow a curved path towards the q -axis until they reach the line OQ , when they turn to remain on this line.

The slope, α , of the limiting state line obtained for any void ratio decreases with increase in void ratio. The variation of α with void ratio, e , as determined from the undrained compression tests, is shown by the curve AB in Fig. 5.

The effective stress paths followed by five samples with different initial states of packing and consolidated under a pressure of 60 lb/in.² are plotted in Fig. 6. The stress paths followed during the initial stages of the tests move

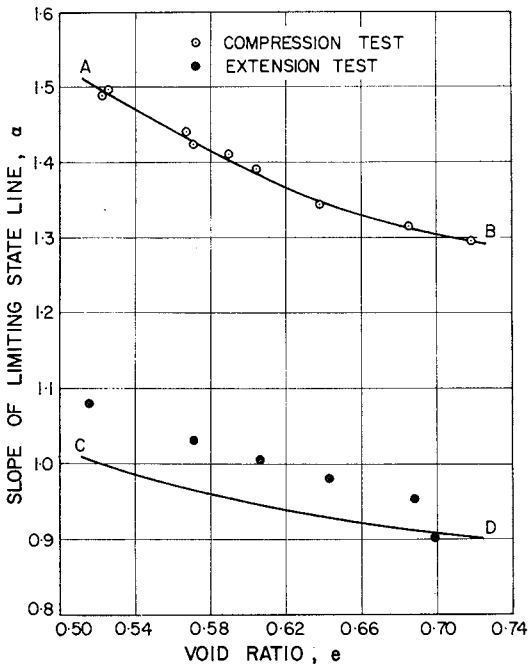


Fig. 5. Variation of slope of limiting state line with void ratio.

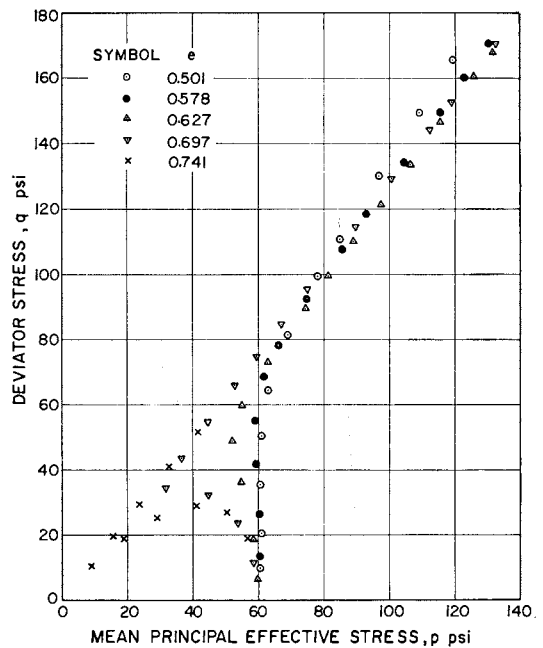


Fig. 6. Stress paths for undrained compression tests.

closer to the q -axis as the void ratio increases. During the later stages of the tests, the stress paths lie on the limiting state line for loose samples and approach the limiting state line for dense samples. Yielding occurs during these stress paths, producing irrecoverable shear strains.

Limiting State Surface

In the three dimensional stress-void ratio (p, q, e) plot used by ROSCOE et al (1958), the limiting state lines obtained from undrained tests with small consolidation pressures lie on a surface passing through the e -axis, represented by $ABCDEF$ in Fig. 7. The intersections of this surface by $e = \text{constant}$ planes give straight lines whose gradients decrease with increase in void ratio. It is not possible for the sand to be in equilibrium under undrained compression conditions for a state point lying outside the domain contained by this surface and the $q=0$ plane. This surface will be referred to as the *limiting state surface*.

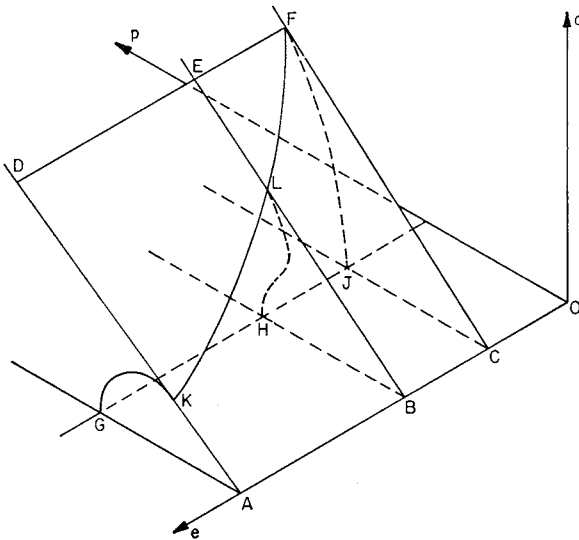


Fig. 7. Isometric view of the limiting state surface.

Yield Surface

Consider now the undrained stress paths of three samples of sand in loose, medium dense and dense states, all consolidated under the same cell pressure. The loose sample follows the curved stress path GK shown in Fig. 7 moving closer to the (q, e) plane, during which q first increases and then decreases to a minimum value, and meets the limiting state line AD at K . q increases with further strain, and the stress path turns back and lies on

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the limiting state line KD . On the other hand, the dense sample follows the stress path JF , which moves away from the (q, e) plane and meets the corresponding limiting state line CF at F . With further strain, the stress path lies on the limiting state line CF extended. The stress path HL for a medium dense sample, which moves in the direction of the (q, e) plane during the initial stage of the test, turns and moves away while still lying below the limiting state line. It then meets the limiting state line BE at L , and lies on the portion of the limiting state line LE for further strain.

The effective stress paths of samples having different void ratios and consolidated under the same cell pressure, therefore, lie on a surface similar to $GHJKLF$ in Fig. 7 until they meet the limiting state surface, when they continue to lie on the limiting state surface. As the samples are subjected to irrecoverable shear strains when the effective stress paths lie on surfaces like $GHJKLF$, these surfaces will be referred to as *yield surfaces*. A series of such yield surfaces exist, each one corresponding to a particular consolidation pressure.

It should be noted that the critical state condition was not attained during any of the tests reported here, but the critical state line for this sand is expected to lie on the limiting state surface further away from the line KLF , similar to that reported for silt by ROSCOE et al (1958).

UNDRAINED EXTENSION TESTS

In a manner similar to that in the compression tests, the triaxial samples deformed uniformly and remained approximately right circular cylinders during undrained extension tests, except in the cases of loose samples consolidated under small cell pressures; this was due to the large axial strains required. For example, a loose sample (void ratio = 0.699) consolidated under 2 lb/in.² was subjected to a conventional axial strain of 20.4%, and the excess pore water pressure decreased from an initial value of 108 lb/in.² to a value of 69 lb/in.². A dense sample (void ratio = 0.516) consolidated under 2 lb/in.², however, attained zero excess pore water pressure with only a 2.9% conventional axial strain. Because of the large axial strains imposed on a loose sample, a neck is formed, generally at mid-height of the sample, at these large strains. This leads to significant error in the deviator stress, q , given by $(\sigma'_3 - \sigma'_1)$, estimated using the conventional area correction (ROSCOE et al, 1963). Hence, results of undrained extension tests on dense samples are more reliable than those on loose samples.

Stress-Strain Relationship

The variation of q , Δu and q/p with ϵ during a typical undrained extension

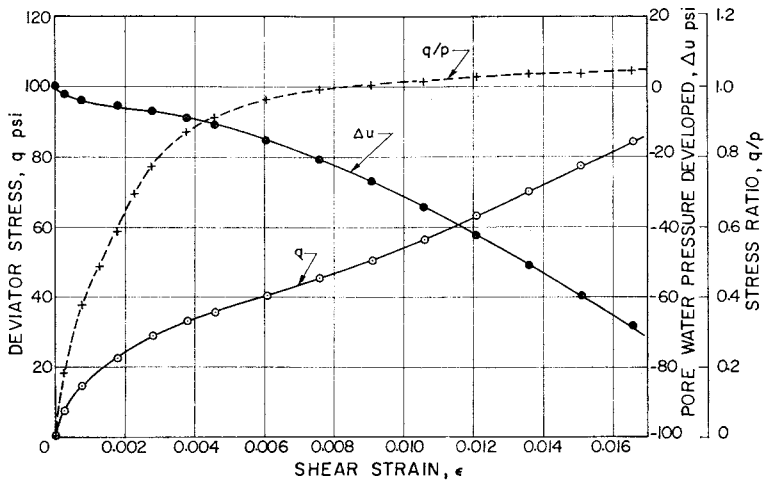


Fig. 8. Undrained extension test on dense sand.

test on a dense sample of sand (void ratio = 0.531) consolidated under a cell pressure of 40 lb/in.² is presented in Fig. 8. Unlike compression tests on dense sand, a reduction in pore water pressure is developed throughout the test. The pore water pressure decreases with ϵ at a decreasing rate up to a certain strain, and then at an increasing rate with further strain, until it becomes zero. q increases continuously with ϵ , initially at a decreasing rate and then at an increasing rate. Points of inflexion can be noticed on the $q-\epsilon$ curve and on the $\Delta u-\epsilon$ curve for the same value of ϵ .

Figure 9 shows q , Δu and q/p plotted against ϵ for a typical undrained extension test on loose sand (void ratio = 0.681) consolidated under 40 lb/in.². The $q-\epsilon$ curve obtained shows unstable conditions similar to the behaviour observed during undrained compression tests on loose sand (see Fig. 2). q increases with ϵ initially, then decreases to a minimum value and increases again at an increasing rate, while q/p increases continuously with ϵ and reaches a constant value. The drop in value of q/p at 9% shear strain is due to the formation of failure planes in the region of the neck formed in the sample. Negative pore water pressure is developed initially which becomes positive after a small strain, until q reaches the minimum value. With further strain, the excess pore water pressure decreases with ϵ at an increasing rate until failure planes are formed.

All samples tested developed negative pore water pressures during the initial stages of the tests. The critical state was not reached in any of the undrained extension tests and pore water pressure continued to decrease to the end of each test.

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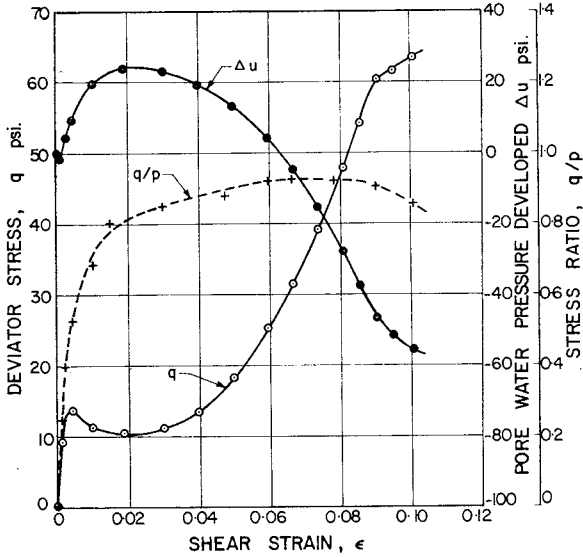


Fig. 9. Undrained extension test on loose sand.

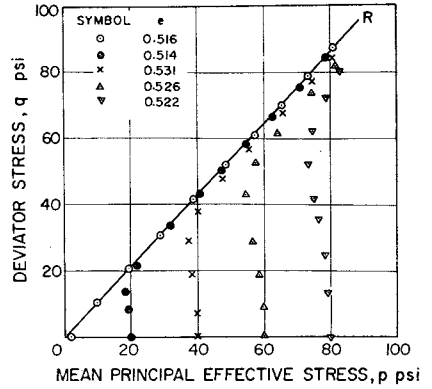


Fig. 10. Stress paths for undrained extension tests on dense sand.

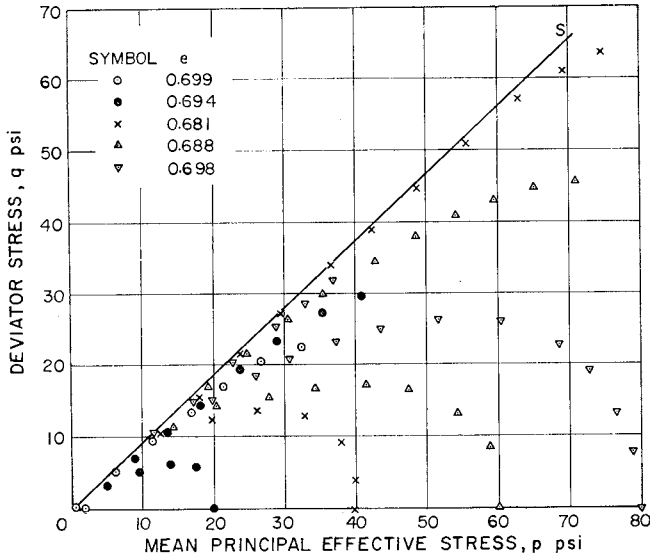


Fig. 11. Stress paths for undrained extension tests on loose sand.

Effective Stress Paths

The effective stress paths on the (p, q) plane of undrained extension tests on dense samples consolidated under different cell pressures are plotted in Fig. 10. As in the undrained compression tests, the stress path for the sample consolidated under 2 lb/in.^2 lies on a straight line OR passing through the origin. OR is the limiting state line corresponding to this void ratio. The

stress paths for high consolidation pressures rise almost vertically during the initial stage of the tests and tend to approach this line with increase of p .

Figure 11 shows the effective stress paths on the (p, q) plane followed by a series of tests on loose sand. The stress paths rise and move towards the q -axis initially, and then turn away from it. OS is the line representing the limiting stress state for these loose samples. The stress paths depart from this line, unlike for dense samples, due to the formation of a neck at higher strains which leads to significant errors in estimated values of q .

The slope, α , of the limiting state line determined for any void ratio decreases with increase in void ratio, as in the compression tests. The variations of α with void ratio for the undrained extension tests are also shown in Fig. 5. Values of α obtained from compression tests are found to be much larger than those obtained from extension tests.

The effective stress paths of four samples consolidated under a pressure of 60 lb/in.² and with different void ratios are plotted on the (p, q) plane in Fig. 12. Their behaviour is similar to those for the undrained compression tests.

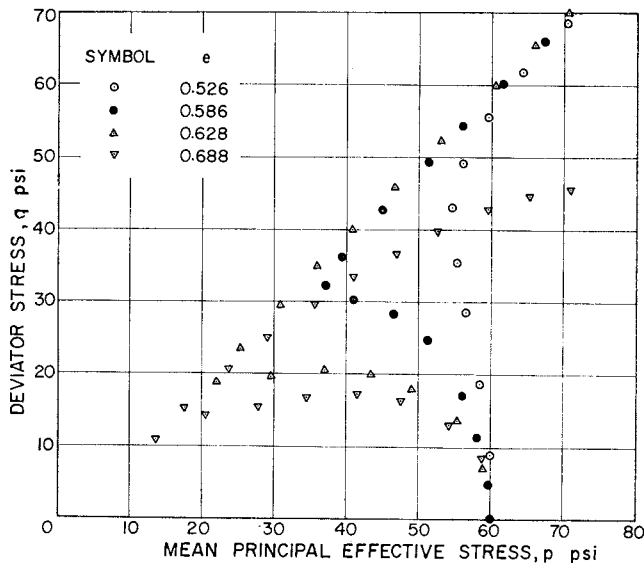


Fig. 12. Stress paths for undrained extension tests.

These results show that, in the three dimensional (p, q, e) space, a limiting state surface, and yield surfaces similar in shape to those shown in Fig. 7 for undrained compression tests, are obtained for undrained extension tests. The intersections of the limiting state surface by $e = \text{constant}$ planes give straight lines whose slopes decrease with increase in void ratio.

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COMPARISON OF COMPRESSION AND EXTENSION TESTS

Limiting State Surface

The limiting state surface shown in Fig. 7 gives the state of the sand when it is in limiting equilibrium under undrained conditions. It is not possible for the sand to have a state point lying above this surface when sheared under undrained conditions. The limiting state surfaces obtained from undrained compression and extension tests will now be compared to investigate which failure criterion is applicable to sand.

The three classic failure criteria that are commonly applied to soils are the extended von Mises, the extended Tresca and the Mohr-Coulomb (BISHOP, 1966; ROSCOE et al, 1963). If the extended von Mises or the extended Tresca criterion is applicable, the value of α determined for any void ratio from undrained compression tests would be equal to the value of α determined from undrained extension tests. From Fig. 5, it is evident that α for compression is much higher than α for extension, showing that the above two criteria are not applicable to sand.

On the other hand, if the Mohr-Coulomb failure criterion is valid, the ratio between the major and minor principal stresses for both types of tests would be equal at the limiting state for the same void ratio. It can be shown that, if α_c is the value of α obtained from compression tests, then α_e determined from extension tests should be $3\alpha_c / (3 + \alpha_c)$ in order to satisfy the Mohr-Coulomb criterion. The curve *CD* in Fig. 5 gives the values of α for extension tests that have been predicted from curve *AB* using the above expression. The closed points obtained from actual experiments show that the Mohr-Coulomb criterion describes failure more closely than the extended von Mises or the extended Tresca criteria.

The limiting equilibrium states determined from the undrained compression and extension tests are again compared in Fig. 13, in terms of the angle of internal friction ϕ' . The curves *KL* and *MN* show the variation of ϕ' with void ratio as determined from undrained compression and extension tests respectively on samples consolidated under a pressure of 2 lb/in.². For any void ratio, ϕ' determined from extension tests is higher than ϕ' determined from compression tests, the difference varying from 4° in the dense state to 2° in the loose state. These results show that the Mohr-Coulomb failure criterion underestimates the strength of soils in many field problems.

Similar results have been reported by GREEN & BISHOP (1969) who found from drained triaxial (axi-symmetric) tests on sand that ϕ' in extension is 2° higher than ϕ' in compression.

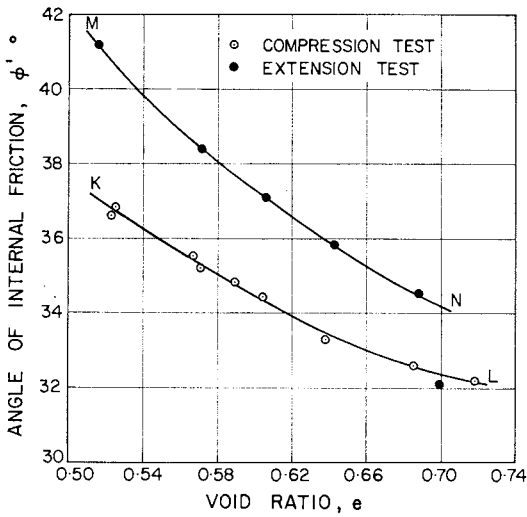


Fig. 13. Comparison of ϕ' determined from undrained compression and extension tests.

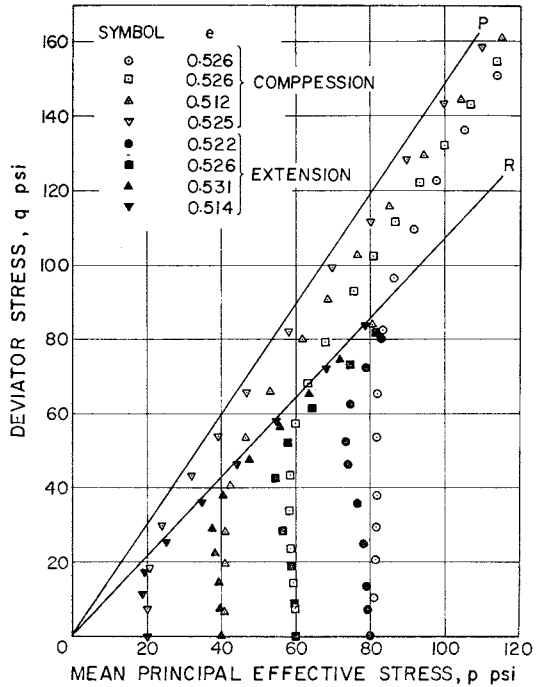


Fig. 14. Comparison of stress paths for undrained compression and extension tests on dense sand.

Yield Surface

In Fig. 14, the effective stress paths of undrained compression and extension tests on dense samples of sand with approximately equal void ratios are compared for four consolidation pressures. *OP* and *OR* are the limiting state lines, corresponding to this density of packing, for compression and extension tests respectively. The stress paths obtained for compression and extension tests appear to lie close to each other until they get near the limiting state line *OR*. The effective stress paths for loose samples are compared in Fig. 15, where *OQ* and *OS* are the limiting state lines for compression and extension tests respectively. The compression and extension stress paths are now widely different from each other, the extension stress path lying much closer to the q -axis than the compression stress path.

Results on normally consolidated clay show that the yield surfaces obtained from triaxial compression and extension tests lie close to each other till the Mohr-Coulomb rupture condition is reached (ROSCOE & BURLAND, 1968). Data presented herein, however, illustrate that the yield surfaces determined from undrained compression and extension tests on sand for any consolidation pressure differ widely from each other.

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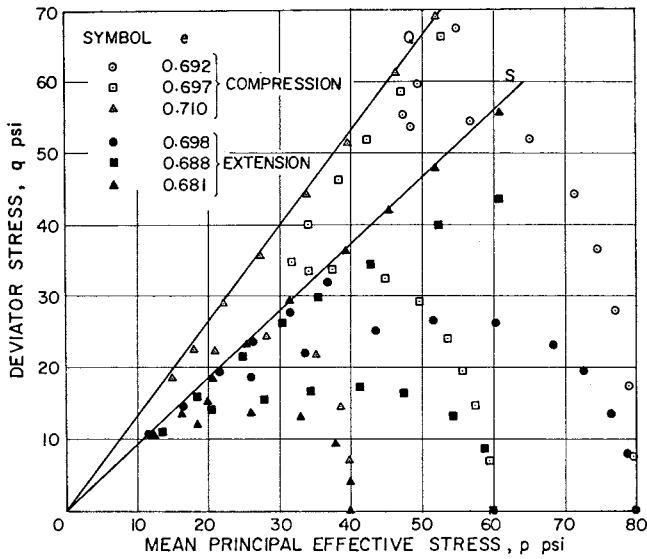


Fig. 15. Comparison of stress paths for undrained compression and extension tests on loose sand.

CONCLUSIONS

Undrained triaxial compression and extension tests on saturated, dense sand (no cavitation allowed to occur) exhibit stress-strain relationships in which q increases with ϵ at an increasing rate, developing negative pore water pressure in the sample, showing that the material progressively locks under undrained conditions. On the other hand, loose samples reach a peak value of q at a small strain, and q decreases with further increase in ϵ until it reaches a minimum value. q then increases with ϵ at an increasing rate, developing negative pore water pressure. This unstable condition has practical significance in the field, since a mass of sand at this state of packing can liquefy under undrained conditions due to shock loading, with very large consequent deformation.

The critical state, at which no change in p , q or Δu takes place with any increase in shear strain, was not attained in any of the tests conducted, the maximum conventional axial strain applied being about 22%.

Limiting state surfaces in (p, q, e) space were established for undrained compression and extension conditions by carrying out tests on samples consolidated under a very small pressure. These surfaces pass through the e -axis and the intersections of the surfaces by $e = \text{constant}$ planes give straight lines whose slopes decrease with increasing void ratio. An element of sand is in limiting equilibrium when its state lies on this surface.

When the limiting state surfaces obtained from compression and extension tests are compared, the results are found to be much closer to the Mohr-Coulomb failure criterion than to either the extended von Mises or extended Tresca criterion. The angle of internal friction, ϕ' , determined from the extension tests for any void ratio is higher than that determined from the compression tests, the difference varying from 4° for dense sand to 2° for loose sand. It is apparent from this result that the intermediate principal stress has some influence on the failure strength. The widely used Mohr-Coulomb failure criterion underestimates the strength of soil in stability problems encountered in practice, most of which can be approximated closely to plane strain conditions.

The stress paths of samples of sand consolidated under higher pressures lie within the domain contained by the limiting state surface and the $q = 0$ plane in (p, q, e) space at small strains, and lie on the limiting state surface at higher strains. The portions, which lie below the limiting state surfaces, of the stress paths for undrained compression and extension tests on sand samples with the same void ratios and consolidation pressures are widely different except for very dense sand, unlike the observed behaviour of normally consolidated clays.

ACKNOWLEDGEMENT

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ANALYSIS OF UNCERTAINTY IN SETTLEMENT PREDICTION

J. NEIL KAY* and RAYMOND J. KRIZEK†

SYNOPSIS

A method is presented to analyze the uncertainty associated with the settlement prediction determined by use of the conventional deterministic approach. Probability distribution functions, instead of individual values, are used for the variables in the formula to obtain a probability distribution function for the settlement due to primary consolidation. The distributions for soil properties are established from a statistical treatment of test results, while those for the stress parameters are deduced on a more subjective basis. The study is generalized by repeating the process for a large number of cases. Values calculated therefrom are then used in regression analyses to develop empirical equations which provide a measure of the uncertainty associated with a wide range of input parameters for this particular problem. Finally, these equations are represented in the form of nomographs which facilitate their usage.

INTRODUCTION

The deterministic approach to design problems in soil mechanics and foundation engineering usually provides a single answer which is frequently construed as "the correct answer". With the advent of modern, high-speed, digital computers which may readily handle theoretical complexities, there has been an increasing tendency by many to accept this "correct answer". However, there are others who fully recognize that the greatest current limitation is the inability to determine accurately the soil properties and other basic parameters of the analytic model. This latter shortcoming may have serious consequences on the accuracy of the results obtained in any given situation, and even the simplest determination of settlement may be only a relatively crude estimate of the actual final settlement which may occur. In calculations leading to the prediction of settlement of structures built on compressible soils, a degree of uncertainty is involved in the values assigned to both soil properties and soil stresses. Consequently, any settlement value determined on the basis of these parameters also displays some degree of uncertainty. Through the application of probabilistic methods, it is possible to obtain an insight into the ranges of uncertainty which may be expected for particular types of settlement problems. The work presented herein is restricted specifically to classical one-dimensional consolidation

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problems in which the settlement is associated entirely with the virgin portion of the conventional e -log p curve, and any settlements resulting from stresses below the preconsolidation stress and from secondary compression are neglected.

APPLICATION OF PROBABILITY THEORY

The settlement, S , associated with the primary consolidation of normally or overconsolidated soils is usually determined from the equation :

$$S = H \frac{C}{1 + e} \log \frac{p_o + \Delta p}{p_c} \dots\dots\dots (1)$$

in which H is the thickness of the compressible layer, C is the compression index of the soil, e is the initial void ratio of the soil in place, p_o is the vertical stress at the mid-height of the compressible layer before loading, Δp is the increase in vertical stress at the mid-height of the compressible layer due to the applied load, and p_c is the preconsolidation stress at the mid-height of the compressible layer; in the case of a normally consolidated clay, p_c equals p_o . For convenience, Eq. (1) may be rewritten in the form :

$$R = \frac{S}{H} = \frac{C}{1 + e} \log \frac{T}{P} \dots\dots\dots (2)$$

in which R is the settlement-layer thickness ratio, T is the final vertical stress at the mid-height of the layer ($T = p_o + \Delta p$), and P is the preconsolidation stress at the mid-height of the layer.

If, instead of specific values, the parameters (C , e , T and P) in Eq. (2) are considered to be distribution functions [$f_C(C)$, $f_e(e)$, $f_T(T)$, and $f_P(P)$], which reflect some degree of uncertainty, an associated distribution function [$f(R)$] may be determined for the dependent variable R . Although a closed-form analytical solution is possible for certain special cases of distribution functions, the use of a numerical technique is necessary in most instances. One simple and convenient sampling procedure, which may be employed to perform the required computations, is the Monte Carlo technique (WARNER & KABAILA, 1968). By means of random number generation on a digital computer, it is possible to produce values which are characteristic of any specified distribution for each of the independent parameters in Eq. (2); this may be considered a simulated sampling procedure. If the generated values for the independent parameters are substituted into Eq. (2), a value for the dependent variable, R , may be determined; repetition of this process a large number of times will yield the dependent distribution function, $f(R)$. This latter

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function is unique for a given set of independent parameter distribution functions, and the accuracy to which it can be determined is limited only by the computer time which is required to generate a sufficient number of test cases.

In order to characterize the spread of the resulting dependent distribution function, the use of a 90% confidence level seems appropriate for settlement prediction; that is, the actual settlement has a 90% chance of lying within the range of settlements predicted. As shown in Fig. 1, a reference value of R , designated \bar{R} , is determined by substituting into Eq. (2) the mean values, \bar{C} , \bar{e} , \bar{T} and \bar{P} , of the independent parameter distributions, and the upper and lower 5% levels of the generated distribution are obtained. From the differences between \bar{R} and the upper and lower 5% levels (H and L , respectively), the ratios H/\bar{R} (designated H_R) and L/\bar{R} (designated L_R) may be determined, and these parameters are utilized subsequently.

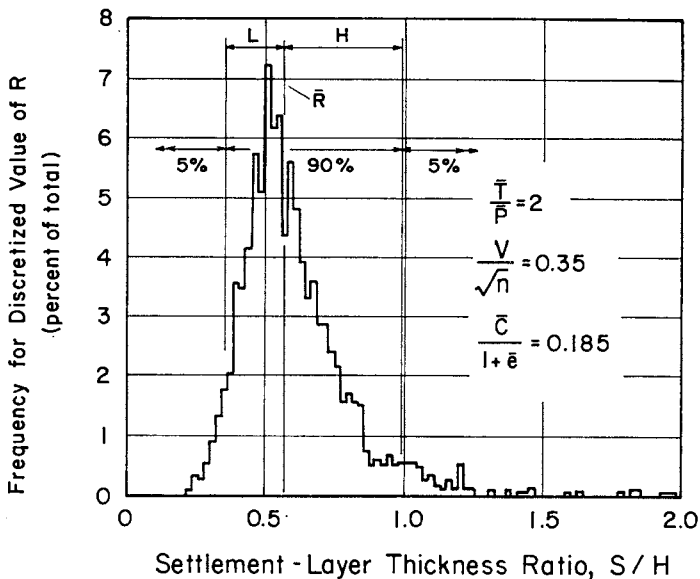


Fig. 1. Typical probability distribution function for settlement.

DISTRIBUTION FUNCTIONS FOR INPUT PARAMETERS

Compression Index

The uncertainty associated with the selection of the compression index, C , may be approached on the basis of its coefficient of variation, V (defined as the ratio of the standard deviation to the mean). A number of investigators (LUMB, 1966; HOOPER & BUTLER, 1966; and others) have demonstrated the usefulness of the coefficient of variation as a statistical parameter to describe

soil properties. Furthermore, if the soil within a given layer is of a similar type and has a reasonably consistent geologic origin, there is evidence to indicate that V may be restricted to rather narrow limits, even though the mean value of the property under consideration may vary somewhat from one location to another. However, owing to the human factor involved in determining this parameter, it must be used with caution; for example, HOOPER & BUTLER (1966) have shown that poor sampling techniques can cause a considerable increase in the coefficient of variation for shear strength, and a similar effect is likely for the compression index. Ideally, the selected value for the coefficient of variation should be based on a large number of previously obtained test results from soils in the vicinity of the project site; alternatively, a conservative estimate may be made on the basis of past experience in measuring the variation in the compression index.

Based on the coefficient of variation, V , the probability distribution function for the coefficient of compressibility, $f_c(\hat{C})$, may be generated from:

$$\hat{C} = \frac{\bar{C}}{\left(1 - \frac{zV}{\sqrt{n}}\right)} \dots\dots\dots (3)$$

in which \bar{C} is the mean compression index for the samples tested, z is the standard normal deviate, n is the number of tests, and \hat{C} represents an estimated value of the population mean (KAY & KRIZEK, 1971). The forms of the distributions for a soil with an average compression index of 0.2 and values of 0.1 and 0.2 for V/\sqrt{n} are shown in Fig. 2. Note the considerable improvement in the estimate with relatively few additional tests.

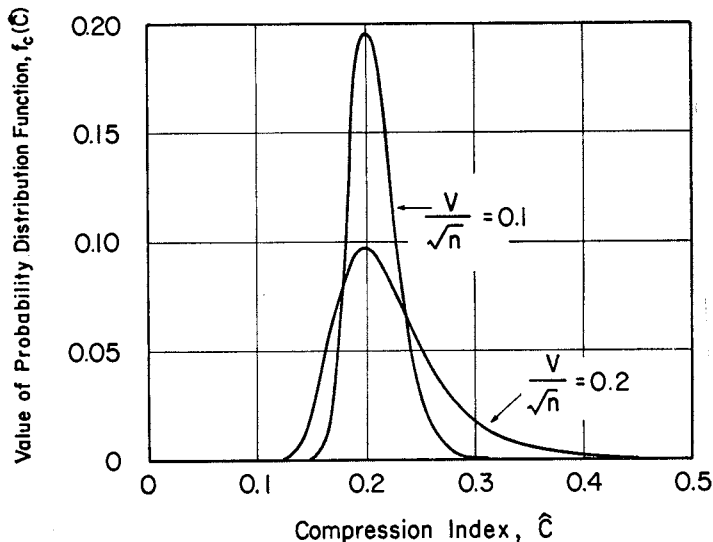


Fig. 2. Probability distribution functions for the compression index.

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Void Ratio

The void ratio, e , of the soil cannot be considered independent of the compression index since, as indicated in Fig. 3 (ELNAGGAR & KRIZEK, 1970), a high degree of correlation exists between the two for a wide range of soils. In particular, the relationship between $C/(1 + e)$ and e is shown to be approximately a straight line passing through the origin, and this latter assumption will be used herein. If the mean values for the void ratio and compression index are used, the slope, M , of the associated straight line may be determined from the expression :

$$M = \frac{C}{e(1+e)} \dots\dots\dots (4)$$

Then, from each value of the compression index generated in the Monte Carlo process, the void ratio is determined from :

$$e = -\frac{1}{2} + \frac{1}{2} \sqrt{1 + \frac{4C}{M}} \dots\dots\dots (5)$$

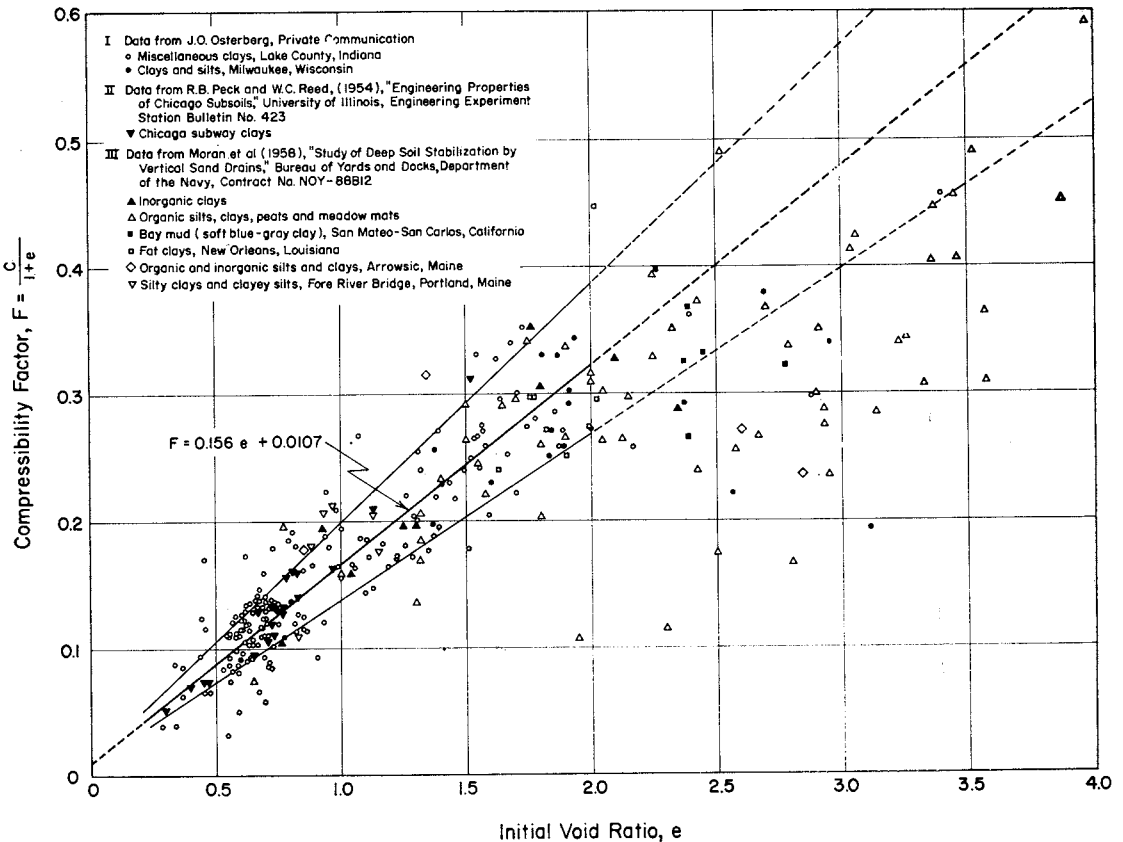


Fig. 3. Relationship between compressibility index and initial void ratio.

In addition to its dependence on C , the overall variation in void ratio, consistent with the data in Fig. 3, is achieved by introducing an independent degree of randomness into e . This is accomplished by generating a normally distributed random variable with a mean e determined from Eq. (5) and a standard deviation of $0.05 e$. This value of void ratio is then used with the associated compression index and the generated values for the other parameters in order to calculate the settlement-layer thickness ratio. The ability of the Monte Carlo technique to accommodate an interdependency such as this demonstrates its flexibility.

Although it is likely that the void ratio also depends to some extent on the initial stress and the preconsolidation stress, relatively little information is available on these relationships, and they are neglected in this work.

Final Vertical Stress

There is very little quantitative evidence on which to base a representation of the uncertainty associated with the estimation of stresses in the ground. Considerable uncertainty exists for the value assigned to the initial vertical stress, p_0 . In addition to the fact that the average dry density of the overlying soil can be only approximately determined, the elevation of the groundwater table may be variable, the degree of saturation may vary considerably, and seepage may result from ever-present transient conditions. Similarly, the added load, Δp , involves some degree of uncertainty. Owing to the lack of statistical evidence, it appears necessary to use personal judgment to define both the distribution type and spread. For the work presented herein, the combined influence of these uncertainties is subjectively assigned to be a normal distribution with a coefficient of variation of 0.05. For example, in terms of an average final vertical stress of 1000 lb/ft^2 , this latter choice of V implies that (a) there is a 32% chance that the average final vertical stress lies outside the range from 950 to 1050 lb/ft^2 , and (b) there is a 5% chance that the average final vertical stress lies outside the range from 900 to 1100 lb/ft^2 . These values indicate that this subjective representation of uncertainty is reasonable.

Preconsolidation Stress

Even more uncertainty must be associated with the evaluation of the preconsolidation stress. Again, a subjective estimate must be made, and a coefficient of variation of 0.1 is used in conjunction with a normal distribution. For an average preconsolidation stress of 1000 lb/ft^2 , this implies that (a) there is a 32% chance that the preconsolidation stress lies outside the

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range from 900 to 1100 lb/ft², and (b) there is a 5% chance that the preconsolidation stress lies outside the range from 800 to 1200 lb/ft². Again, these values appear reasonable.

UNCERTAINTY OF SETTLEMENT DETERMINATION

In order to evaluate the effect of the input parameters on the probability distribution function for the settlement-layer thickness ratio, it is desirable to minimize the number of variables involved. This is accomplished by holding constant the uncertainties associated with the stress parameters and establishing particular combinations of the remaining independent variables. The nature and ramifications of Eq. (2) indicate that the individual treatment of \bar{C} , \bar{e} , \bar{T} , \bar{P} , V and n is unnecessary; instead, only the combined terms $\bar{C}/(1 + \bar{e})$, \bar{T}/\bar{P} and V/\sqrt{n} need be considered. By varying these terms over the ranges indicated in Table 1, a large number of values (64 in all) were determined for L_R and H_R . Then, multiple regression curve-fitting techniques were used to establish the following empirical relationships :

$$L_R = 0.07 + \frac{0.90}{(\bar{T}/\bar{P})^2} + 1.07 \frac{V}{\sqrt{n}} \log \frac{\bar{T}}{\bar{P}} \dots\dots\dots (6)$$

$$H_R = \frac{1.21}{\bar{T}/\bar{P}} + 0.104 \frac{\bar{T}}{\bar{P}} + 4.84 \frac{V}{\sqrt{n}} - 0.88 \log \frac{V}{\sqrt{n}} - 0.047 \frac{\bar{C}}{1 + \bar{e}} \exp \left[10 \frac{V}{\sqrt{n}} \right] - 1.897 \dots\dots\dots (7)$$

Note that the low deviation ratio, L_R , is independent of the parameter $\bar{C}/(1 + \bar{e})$. The application of Eqs. (6) and (7) is facilitated by use of the nomographs given in Figs. 4 and 5.

Table 1. Ranges of parameters

Parameter	Range
$\bar{C}/(1 + \bar{e})$	0.070 to 0.185
\bar{T}/\bar{P}	1.5 to 4.0
V/\sqrt{n}	0.05 to 0.35

In order to illustrate the effects of the number of specimens tested, n , and the load ratio, \bar{T}/\bar{P} , on the deviation ratios, L_R and H_R , the relationships shown in Fig. 6 have been determined for constant values of the remaining parameters. Except for \bar{T}/\bar{P} , the ranges chosen for the other parameters are

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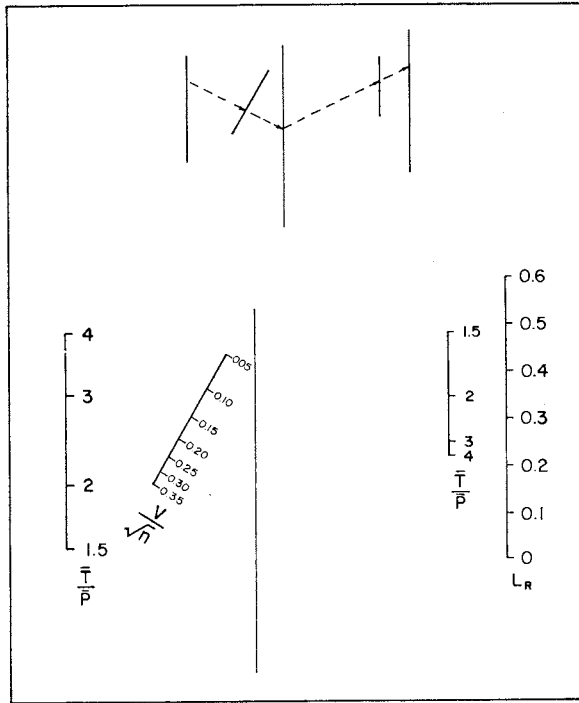


Fig. 4. Nomograph for low deviation ratio, L_R .

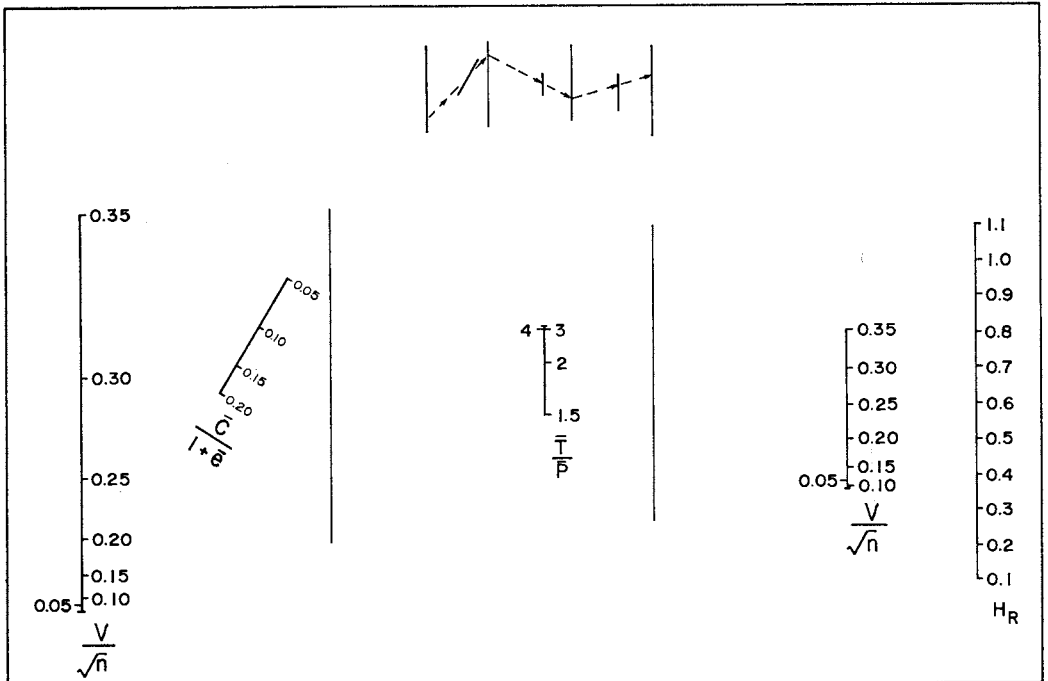


Fig. 5. Nomograph for high deviation ratio, H_R .

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consistent with those generally found in practice. Although \bar{T}/\bar{P} values from 1.0 to 1.5 are relatively common in field situations, certain programming complexities are encountered in this range, and the basic approach presented herein requires some modification. In particular, Fig. 6a indicates a tendency for L_R and H_R to increase rapidly for \bar{T}/\bar{P} values less than 1.5, and both would become infinite for a \bar{T}/\bar{P} value of unity. It should be recognized that the influence of the subjectively determined uncertainties associated with soil stresses plays an increasingly large role in the overall uncertainty as the \bar{T}/\bar{P} value approaches unity. Accordingly, the approach proposed herein then becomes increasingly inaccurate, and caution is recommended against its use in such cases.

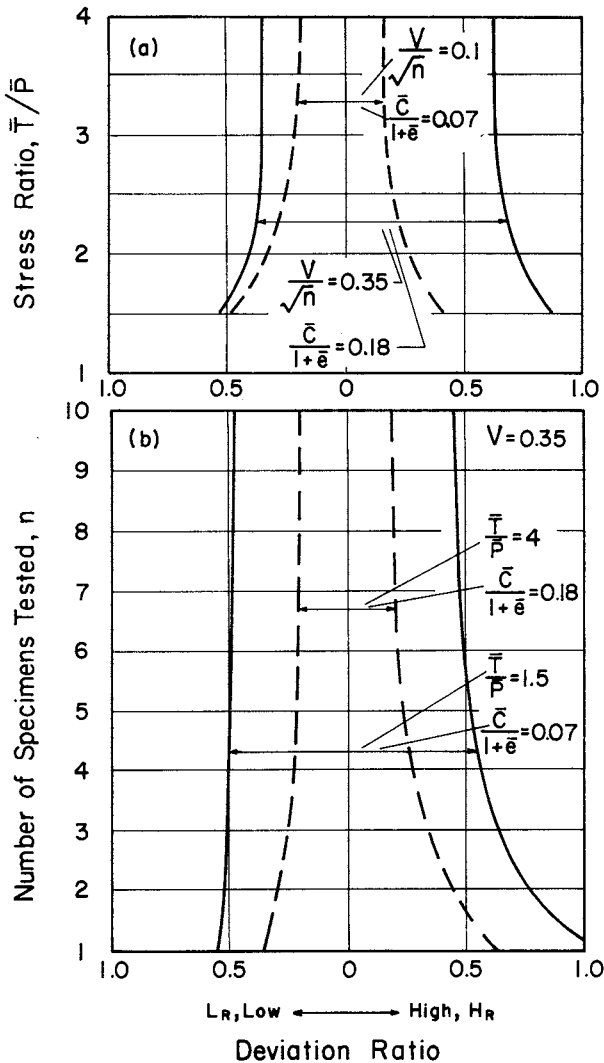


Fig. 6. Effect of stress ratio and number of specimens tested on deviation ratios.

ILLUSTRATIVE EXAMPLE

Consider the case where a compressible layer 15 ft thick is to be loaded one-dimensionally such that the estimated final stress at the mid-height of the layer is 4000 lb/ft². Four consolidation tests from samples taken in the layer indicate an average compression index of 0.2, an average void ratio of 0.75, and an average preconsolidation stress of 2000 lb/ft². Previous tests on similar soils in the area indicate that 0.3 is a conservative value for the coefficient of variation for the compression index. To determine the expected settlement due to primary consolidation, the solution would proceed as follows :

$$\begin{aligned} S &= H \frac{\bar{C}}{1 + \bar{e}} \log \frac{\bar{p}_o + \Delta\bar{p}}{\bar{p}_c} \\ &= (15 \times 12) 0.114 \times 0.302 \\ &= 6.2 \text{ in.} \end{aligned}$$

Since $\bar{T}/\bar{P} = 2$, $\bar{C}/(1 + \bar{e}) = 0.114$ and $V/\sqrt{n} = 0.15$, Figs. 3 and 4 indicate that $L_R = 0.33$ and $H_R = 0.35$, respectively. Hence, we may calculate :

$$\begin{aligned} (1 - L_R)S &= (1 - 0.33) 6.2 = 4.2 \text{ in.} \\ (1 + H_R)S &= (1 + 0.35) 6.2 = 8.4 \text{ in.} \end{aligned}$$

Therefore, there is a 90% probability that the settlement will lie within the range from 4.2 to 8.4 in.

Application of the foregoing approach presumes careful consideration of the geological aspects of the compressible layer and the relationship thereto of the sampling techniques. Although the selection of a coefficient of variation for the soil compression index should ideally be based on test data from a similar soil, the accuracy lost by use of a conservative value for the coefficient of variation may be regained by testing additional specimens. In general, specimens should be obtained from reasonably dispersed points in the layer. It is interesting to note that, if the preceding data were based on only one consolidation test, other values remaining the same, the range of the expected settlement would extend from 3.8 to 10.7 in.

CONCLUSION

A probabilistic analysis has been proposed to handle quantitatively the effects of uncertainties in the input parameters (compression index, void ratio, final stress and preconsolidation stress) on the settlement predicted

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from the primary phase of classical one-dimensional consolidation theory. The accuracy of the result has been considered in terms of its 90% confidence interval, and a simple quantitative means for its determination has been advanced. The use of this approach obviates the need to use conservative values for soil and stress parameters, as is often done in current design practice; rather, the most accurate average values available should be used, and the desired degree of conservatism should enter only into the selected coefficient of variation.

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A CONTRIBUTION TO THE STUDY OF THE PHYSICO-CHEMICAL IMPLICATIONS OF TROPICAL WEATHERING AND LATERISATION

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SYNOPSIS

Studies have shown that the structure and engineering properties of lateritic soils are influenced considerably by weathering and by genesis. In this paper, an attempt is made to assess the significance of tropical weathering on the physico-chemical properties of two genetic soil types in Ghana. The study has revealed that the physico-chemical properties are of more significance in the siliceous, tropical, clay soils of the coastal zone than in the sesquioxide-rich soils of the forest zone. Correlation of base-exchange capacity and hygroscopic moisture content with clay content shows that the influence of the clay content on these properties is more pronounced for the poorly laterised soils. The differences in the relationships between the clay contents and the physico-chemical properties is attributed to the degree of laterisation (free iron coating of the clay mineral) and to the type of clay mineral in the soil types.

INTRODUCTION

The development of concretionary structure in lateritic soils seems to involve three processes: (a) physical and chemical weathering of parent materials and release of small primary particles of iron or alumina gels with the leaching away of combined silica and bases; (b) coating and coagulation or flocculation of the particles by the iron or alumina gels; and (c) hardening due to the dehydration of the hydroxides of iron and aluminium. The free iron oxide is believed to play a major role in the development of this concretionary structure in lateritic soils. The gelatinous free iron oxide first coats the soil particles, exerting a cementing effect on the clay, silt and sand fractions, leading to the formation of larger particles and nodules upon dehydration. This binding of smaller particles into larger ones is considered to account partly for the low plasticity, activity and compressibility and for the high specific gravity, permeability and angle of shearing resistance of some lateritic clay soils. The high crushing strength of the concretionary aggregates also correlates well with the amount of iron enrichment. In spite of considerable advance in the engineering studies of lateritic soils, the physico-chemical implications of the iron coating of clay and silt particles and of the clay mineral type in the soil have not been evaluated sufficiently to date.

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In this paper an attempt is made to evaluate, in terms of physico-chemical properties, the differences between siliceous, tropical, clay soils and the sesquioxide-rich, fine-grained, lateritic soils of Ghana, and to advance possible reasons to explain these differences. It is hoped that this study will stimulate more studies on the physico-chemical and engineering implications of tropical weathering and laterisation.

ENGINEERING IMPLICATIONS OF LATERISATION

The available geological-pedological information indicates that the morphology, structure and general characteristics of a tropically weathered soil are influenced considerably by the particular weathering system, comprising the joint effects of the parent material (geological formation), climate, vegetation, topography and drainage conditions, and the time during which these factors have operated (e.g. HARRISON, 1910, 1933; JENNY, 1941; MOHR & VAN BAREN, 1954; NYE, 1955). In particular, it has been shown that climate, topography and vegetation influence the weathering, through their control of the character and direction of movement of water through the alteration zone, and determine whether the weathering system and drainage conditions are productive of a kaolinite, halloysite or montmorillonite type mineral or of some lesser known secondary mineral (MOHR & VAN BAREN, 1954; STEPHEN, 1953; HOUTEN, 1955; KELLER, 1957; DUMBLETON et al, 1966). The pedogenic processes (laterisation) involving the coating and coagulation of clay and silt particles by the alumino-ferruginous agents are considered to underlie the deviations in the engineering behaviour of lateritic soils from the expectations of conventional soil mechanics as developed in Europe and North America for temperate zone soils. D'HOORE & GEOGEART (1954) have described "pseudosilt" and "pseudosand" formations in Congolese (African) soils; oxide-coated clay particles are bound by alumino-ferruginous binding agents and/or organic complexes to form stable aggregates of silt size which, in turn, are compounded to form particles of sand size.

Engineering studies have also established that pretreatment, index test procedures and the interpretation of test data are influenced considerably by the unique concretionary structure of lateritic soils (ANDREWS, 1936; CLARE, 1948; REMILLON, 1955; DUMBLETON et al, 1966; QUINONES, 1963; MOH & MAZHAR, 1969; DE GRAFT-JOHNSON et al, 1969). The concretionary nature of the lateritic soils has led to difficulties in achieving particle dispersion during grading and sedimentation tests, and inconsistencies in results of particle size distribution and Atterberg limits because of different methods of pretreatment have been widely reported (e.g. BEALE, 1939; TERZAGHI, 1958; SHERWOOD, 1967). It was found, for example, that the use

TROPICAL WEATHERING AND LATERIZATION

of sodium oxalate on the typical halloysite clay from Kenya gave a clay fraction of between 20 and 30%, while sodium hexametaphosphate gave a value of between 40 and 50% with the same soil (TERZAGHI, 1958).

The liquid limit of a typical red clay from Kenya (SHERWOOD, 1967) has been found to vary with the mixing time up to the time necessary to break all the aggregations caused by free iron binding (Fig. 1). An increase in liquid limit from 77 to 93% for the same red clay soil upon the removal of the free iron was also reported. The effect of the iron oxide in binding smaller particles into larger ones is also considered to account partly for the successful performance under tropical conditions of road surfacings with silt and clay contents considerably higher than the accepted ASTM standard. TERZAGHI (1958) and NEWILL (1961) attributed the properties of low compressibility, high permeability and high angle of shearing resistance of the Sasumua clay (Kenya) to the iron oxide content, which binds into aggregates the clay minerals existing in the form of tube shaped crystals.

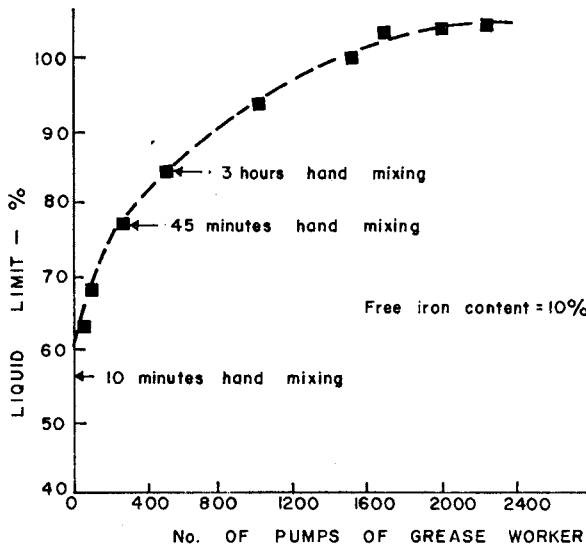


Fig. 1. Dependence of measured liquid limit on amount of mixing of an iron-cemented soil; Kabeke clay from Kenya (after SHERWOOD, 1967).

BHATIA & HAMMOND (1970) have shown that the strength (hardness) of concretionary aggregates correlates well with the amount of iron enrichment (see Fig. 2). It has also been shown that the higher the iron content, the higher the specific gravity (NOVAIS-FERREIRA & CORREIRA, 1965). The low colloidal activities in lateritic soils have been attributed to the existence of the considerable amount of iron in these soils (DUMBLETON & NEWILL, 1962). The existence in laterite materials of iron and aluminium oxides at different stages of hydration has been found to account for the very wide differences

Table 1. Summary of weathering conditions and nature of soil types

Pedological Soil Group	Parent Rocks	Average Annual Rainfall	Vegetation Conditions	Climatic Conditions	pH of Medium	Internal Drainage Condition	Degree of Leaching and Laterisation	Predominant Clay Mineral Type	Remarks
Tropical, black and brown, clay soils	Basic gneiss and acidic schists	Below 1000 mm	Short grass and shrubs	Arid to dry; evaporation exceeds precipitation	Basic to neutral	Poor to fair	Low to medium	Kaolinite with montmorillonite and vermiculite	Siliceous clay soils
Red to reddish brown, lateritic soils	Granite and phyllite	Above 1000 mm	Semi-deciduous to rainforest	Generally wet; precipitation almost equals evaporation	Neutral to acidic	Good to very good	High to very high	Kaolinite and gibbsite, with biotite	Ferruginous and aluminous soils

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in the compaction characteristics of some red, tropically weathered soils subjected to different pretest drying methods (NEWILL, 1961; QUINONES, 1963; GIDIGASU, 1971a).

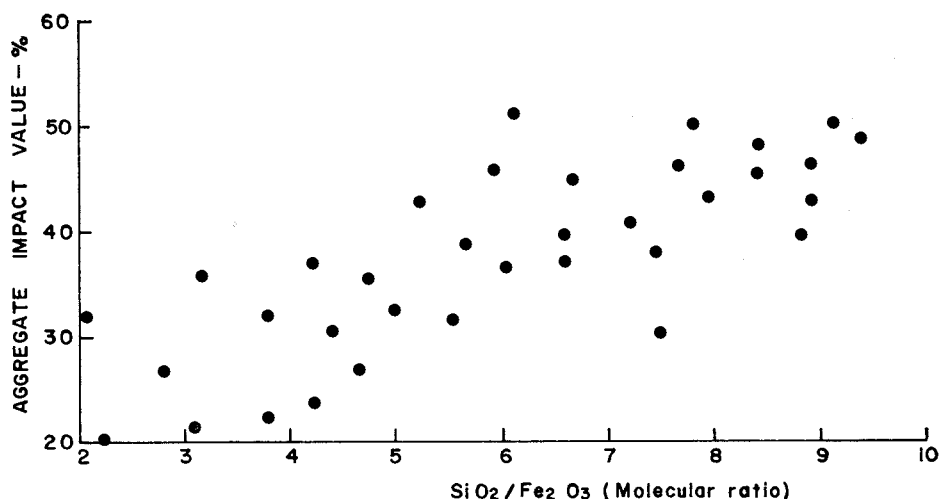


Fig. 2. Relationship between SiO₂/Fe₂O₃ and Aggregate Impact Value (after BHATIA & HAMMOND, 1970).

SCOPE OF THE STUDY AND SOILS USED

Two genetic soil types formed in two climatic-vegetational zones with different drainage conditions were selected for the study. These were :

- (i) The widely distributed red, residual, iron- or alumina-rich soils which form the most important road making materials in Ghana; these cover about 90% of the surface area of Ghana.
- (ii) The tropical, brown and black, clay soils which, though not widely distributed, are generally troublesome as engineering materials on account of their volume change characteristics.

THE FORMATION AND GENERAL CHARACTERISTICS OF THE SOILS

Formation of the Soils

The weathering conditions for the soils studied are summarised in Table 1. Detailed information on the weathering of these soils is given elsewhere (e.g. STEPHEN 1953; BRAMMER, 1962; HAMILTON, 1964). In the wet forest zone, the climate is characterised by alternate downward and upward movement of water through the alteration zone. With good internal drainage, the alkalis and alkaline earths present in the parent rock tend to be leached away. This intense leaching of the bases, coupled with partial or complete

oxidation of the surface organic material, creates a neutral or acid medium. This is followed by breakdown and leaching out of combined silica with a relative accumulation of oxides of iron and aluminium in the boundary between the highest water table and the surface zone of aeration. The thickness of this sesquioxide-rich horizon is a function of the local topography and the depth of the water table.

In the dry arid coastal savanna zone, the dominant movement of water is upward, through evaporation. With poor internal drainage, the leaching of bases is minimal, the alkalis and alkaline earths which remain in the profiles raising the soil to neutral-to-alkaline status. Under these conditions, silica-rich, tropical, black and brown, clay soil profiles are formed. These soils generally contain some montmorillonite in addition to kaolinite (STEPHEN, 1953; BRAMMER, 1962). A general feature of these profiles is that they are rich in calcium carbonate concretions at fairly shallow depths.

The Morphology and General Characteristics of the Soil Groups

Typical profiles of the two soil groups in Ghana are given in Figs. 3 and 4. In the residual profiles, the A-horizon is generally a humus-stained top soil,

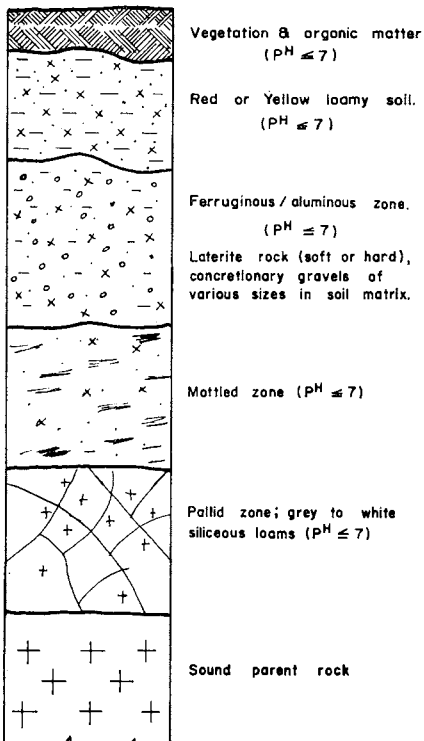


Fig. 3. Typical lateritic soil profile in the forest zone of Ghana.

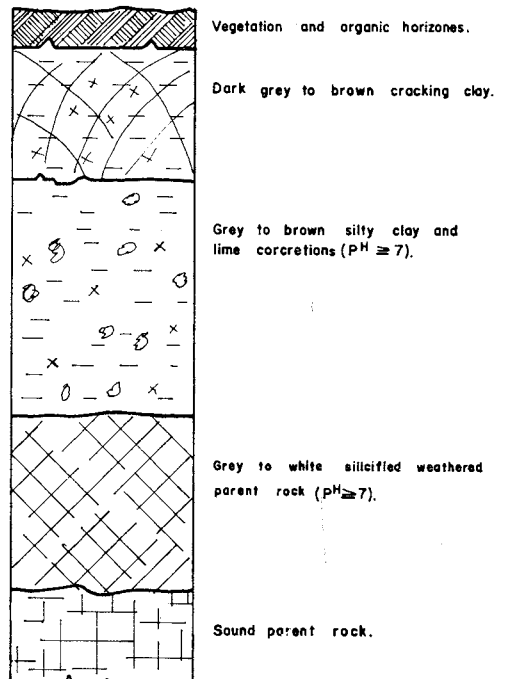


Fig. 4. Typical tropical clay soil profile in the coastal savanna zone of Ghana.

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which varies in thickness from 0 to about 2 ft depending on the climatic-vegetational zone and the local topography. *B* is a zone of accumulation from the *A*-horizon during the rainy season, and from the *C*-horizon in the dry season through evaporation. In the lateritic profiles, the *B*-horizon is very rich in the so-called *lateritic constituents*, mainly Al_2O_3 and Fe_2O_3 . The soil in this horizon is either *in situ* hardened or capable of hardening on exposure, and ranges texturally from very hard laterite concretionary crusts (craie), through laterite nodular gravels, to red clays.

In the non-laterised tropical clay profiles, the *B*-horizon is rich in calcium carbonate concretions. The *B*-horizon generally varies in thickness from about 5 ft in the middle slope of the local topography to over 30 ft as laterite mantles on peneplain remnants. The *C*-horizon is essentially a zone of weathered parent rock from which soluble substances have been moved up to the *B*-horizon. While the *C*-horizon barely attains 10 ft over basic gneiss in the coastal savanna zone, it goes down to over 50 ft on granites and phyllites in the forest zone. The *D*-horizon is the unweathered parent rock.

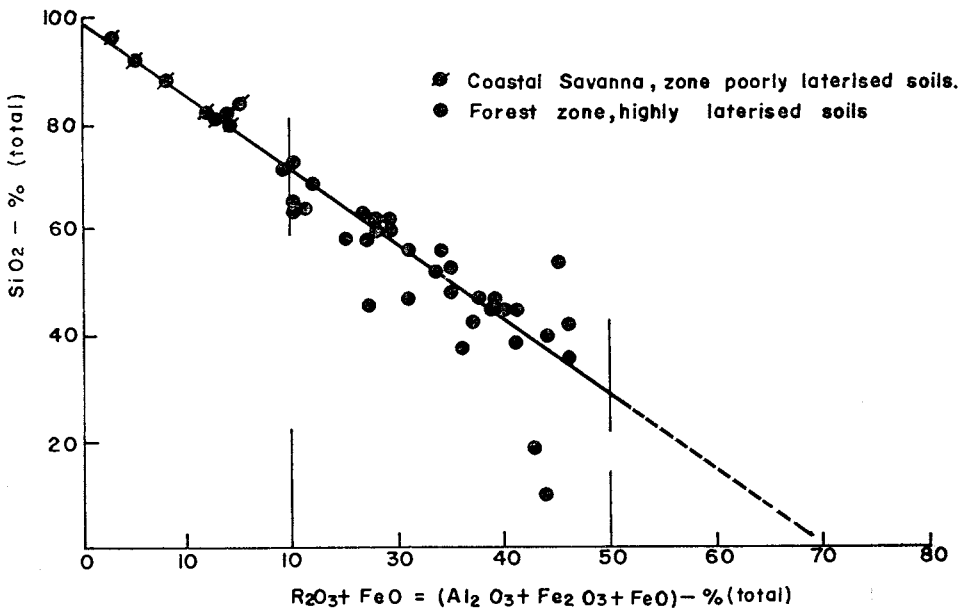


Fig. 5. Chemical characteristics of the two genetic soil types.

Figure 5 shows the chemical characteristics of the soil types. It should be noted that there is a fairly good inverse correlation between the contents of total silica and sesquioxides for the two soil groups. The total sesquioxide content of the poorly laterised soils ranges between 0 and 20%. For the highly laterised soils, the total sesquioxide content is between 20 and 50%.

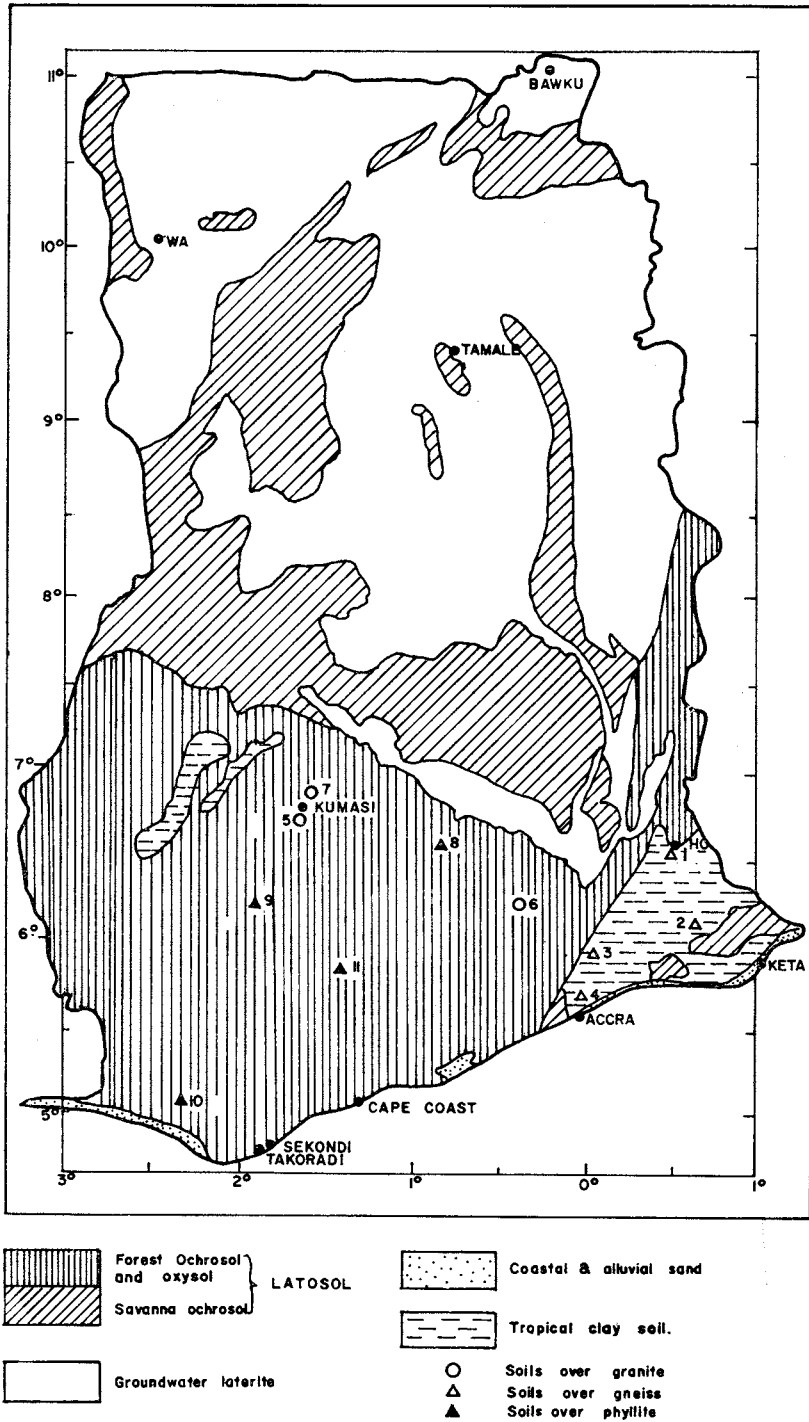


Fig. 6. Simplified pedological soil map of Ghana (after BRAMMER, 1962) showing locations of profiles studied.

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From the available data on the mineralogy of Ghana soils (e.g. STEPHEN, 1953; HAMILTON, 1955; RUDDOCK, 1967; BAMPO-ADDO et al, 1969), the main clay minerals in the siliceous tropical soils are kaolinite with some montmorillonite together with vermiculite and biotite. Kaolinite predominates together with gibbsite and mica (sericite and illite).

LABORATORY STUDIES

Figure 6 shows the distribution of the sample pits used in this study on the simplified pedological map of Ghana. The tropical clay soils were sampled from four profiles in the coastal savanna zone, *Cs-Gn-R*(1-4), from three profiles over granite, *F-Gt-R*(5-7), and four profiles over phyllite, *F-Ph-R* (8-11), in the forest zone.

Preparation and Testing of Soil Samples

The main object of the pretreatment used was to obtain the highest degree of dispersion of the soil types and to eliminate, as far as possible, the cementing effects of the clay fraction, silica, organic matter, calcium carbonate and free iron oxide. In order to ensure the removal of the cementing effects of the calcium carbonate and organic matter, the soils were treated with hydrochloric acid and hydrogen peroxide, according to the procedures described in the British Standard 1377:1961. Prolonged and intensive manipulation prior to the determination of the particle size distribution should have been sufficient to remove the cementing effects of the silica and of the clay fraction (SHERWOOD, 1967). The only possible source of additional aggregation or cementation of the particles was the free iron oxide content. Tests to determine the particle size distribution, pH value and organic matter content were also carried according to the procedures described in the British Standard 1377:1961. The calcium carbonate content, the base exchange capacity and the hygroscopic moisture content were determined according to modified specifications established for Ghana soils (DE ENDREDDY, 1954).

DISCUSSION OF TEST RESULTS

It is assumed that, among the three groups of cementing agents (organic matter, silica and free iron oxide), all except the iron oxides are removed by normal pretreatments with hydrochloric acid, hydrogen peroxide and during vigorous manipulation prior to the sedimentation test (SHERWOOD, 1967). Consequently, differences in the physico-chemical properties of the soils should be due mainly to the different degrees of laterisation and to the clay mineral types.

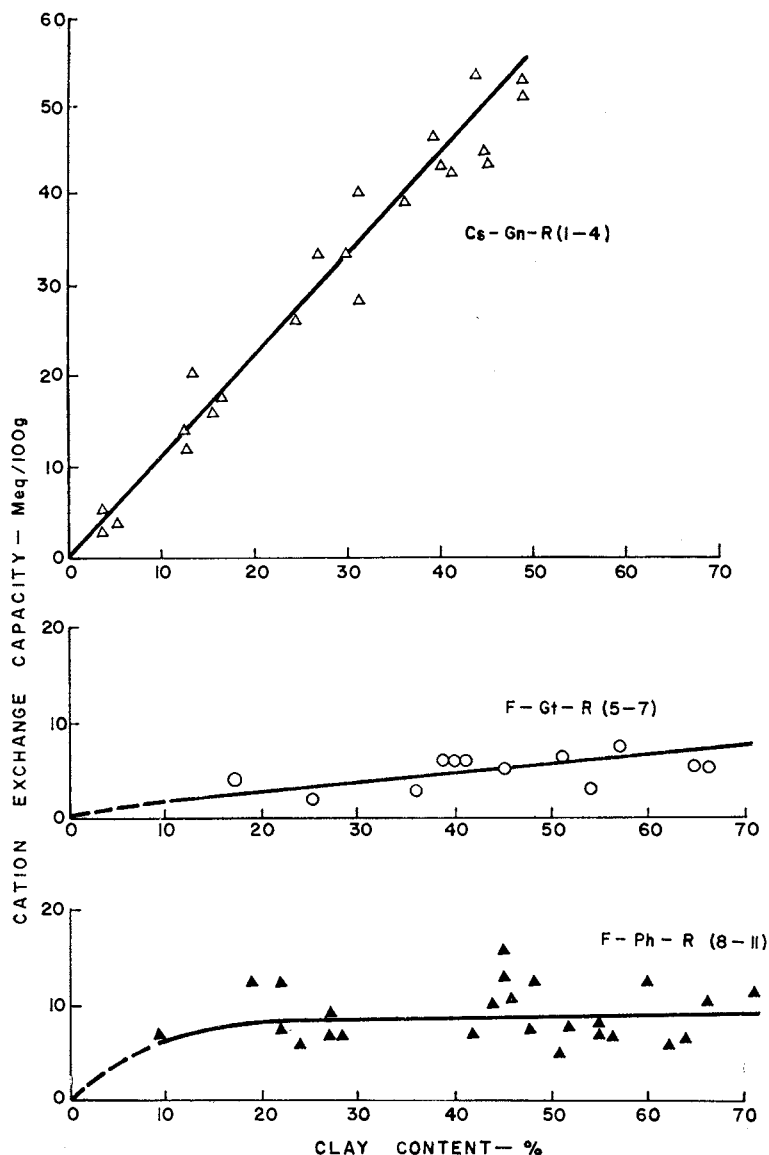


Fig. 7. Relationship between clay content and cation exchange capacity.

Organic Matter Content, pH and Calcium Carbonate Content

It has been shown in an earlier study (GIDIGASU, 1971) that there are appreciable differences in organic matter content between the coastal savanna and the forest zone soils in the 20 in. top soil layer, in which the organic matter content in all cases varies roughly between 0 and 5%. Below this depth, the organic matter content hardly exceeds 2% in either of the climatic-vegetational zones. The organic matter content has also been found to be highest in the clay soils and lowest in the gravelly soils (NEWILL, 1959). In

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terms of hydrogen exponent (pH), the soil medium in the coastal savanna zone is neutral-to-alkaline ($\text{pH} \geq 7$), while the soil medium in the forest zone is neutral-to-acidic ($\text{pH} \leq 7$). It is also found that, while the calcium carbonate content is very high in the coastal savanna zone profiles (up to 15%), it is either very low (about 1%) or zero in the forest zone profiles.

Base Exchange Capacity

The base exchange capacity is fairly high in the tropical clay profiles as compared to the lateritic soil profiles. Figure 7 shows the relation between the clay content and the base exchange capacity for the three soil groups. There is a fairly good correlation between the two parameters. Consideration of the average curves in Fig. 8 shows that the influence of the clay content on the base exchange capacity is more pronounced for the poorly laterised tropical clays. Since the base exchange capacity depends mainly on the specific surface of the clay fraction, the low capacities of the lateritic soils may be attributed mainly to the surface coating of the clay fraction by sesquioxide gels, which reduce the surface activity of the clay minerals, and to the clay minerals in the two soil types.

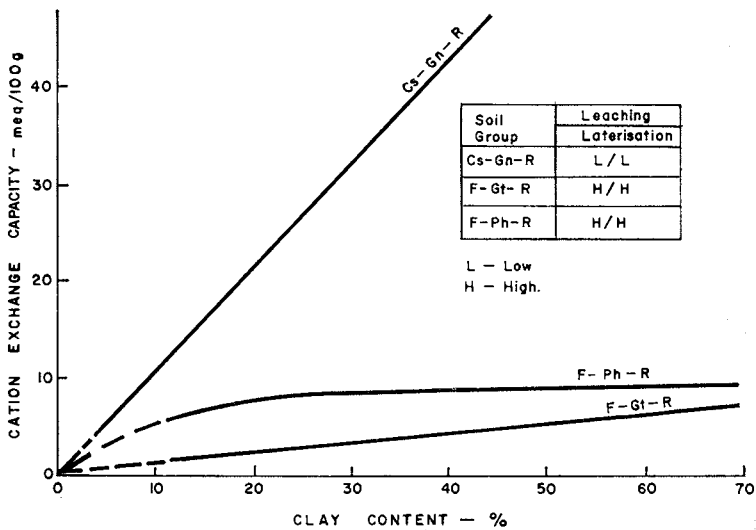


Fig. 8. Average curves for relationship between clay content and cation exchange capacity.

Hygroscopic Moisture Content

The hygroscopic moisture content is very high in the coastal zone tropical clays as compared with the highly laterised soils over granites and phyllites. Figure 9 shows the relation between the clay content and hygroscopic moisture content for the different soil groups. From the average curves in

Fig. 10, it can be seen that the influence of the clay content on the hygroscopic moisture content is more pronounced for the poorly laterised soils. While the increase in clay content consistently gives higher hygroscopic moisture content values for the tropical clays, the hygroscopic moisture content is almost constant with increase in clay content for the highly laterised soils. The higher the degree of leaching and laterisation, therefore, the less the influence of the clay content on the hygroscopic moisture content.

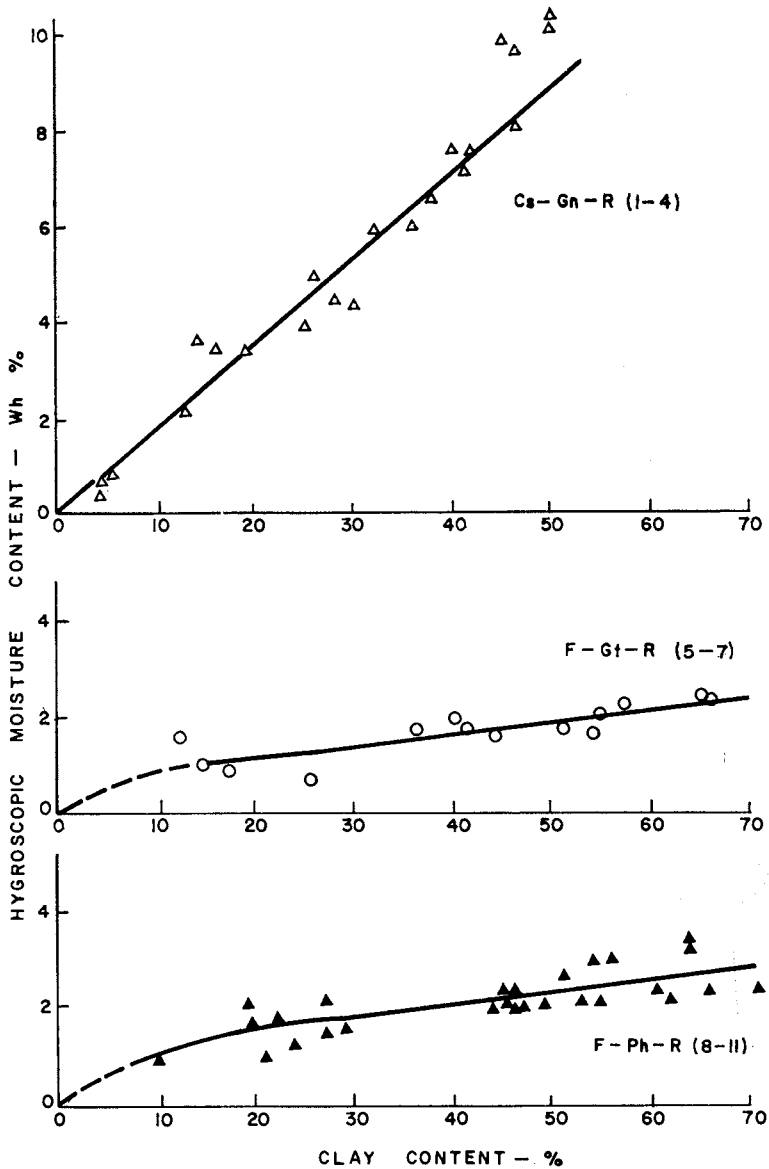


Fig. 9. Relationship between clay content and hygroscopic moisture content.

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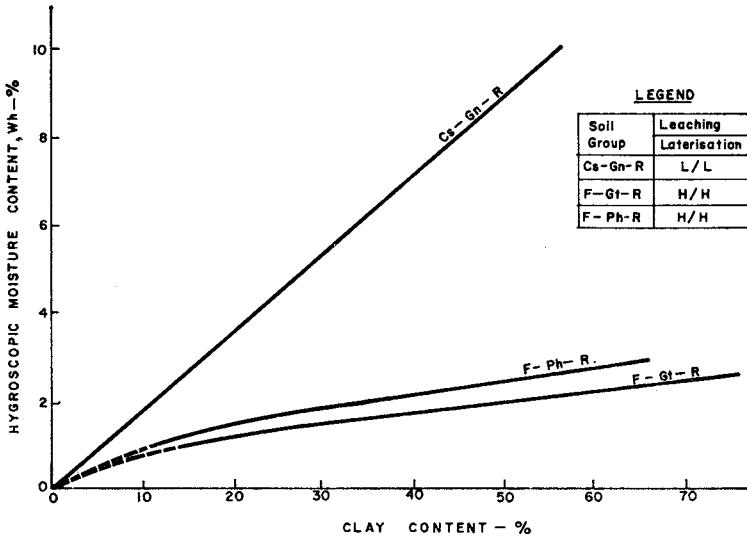


Fig. 10. Average curves for relationship between clay content and hygroscopic moisture content.

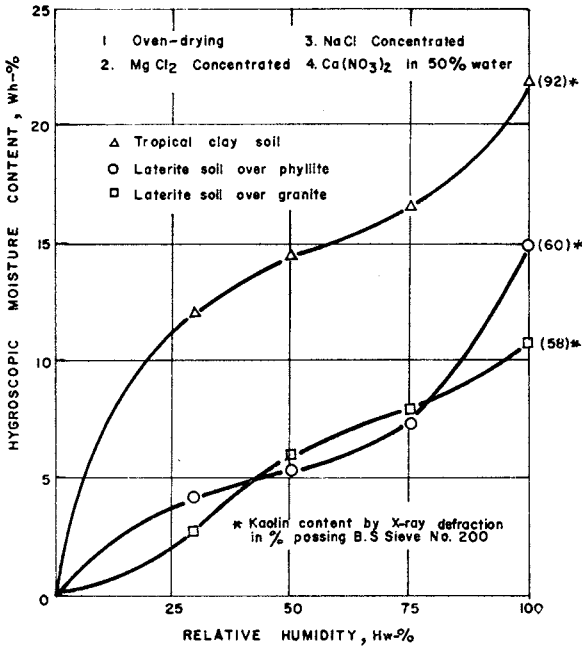


Fig. 11. Relationship between relative humidity and hygroscopic moisture content for typical clays in the soil groups.

Absorption and Loss on Ignition

Typical tropical and lateritic clay soils were subjected to absorption tests using different salts for creating media of different relative humidities. The

salts used and the absorption curves obtained for the clay soil types are given in Fig. 11. It is seen that, at the same relative humidity, the tropical clay absorbs more water than the lateritic clay. This may also be explained in terms of the laterisation process and the clay mineral type.

Figure 12 shows the thermal (gravimetric) curves of the two clay soils. The trend of each curve is similar, reflecting some common mineralogical composition, but the difference in the percentage loss in weight at the same temperature may be due to differences in the physico-chemical characteristics resulting from differences in the weathering conditions and degrees of leaching and laterisation.

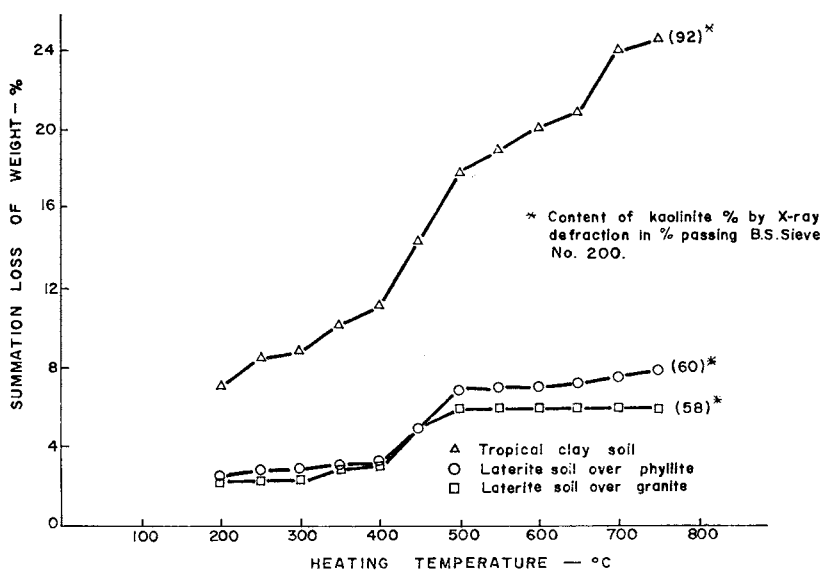


Fig. 12. Relationship between loss-on-ignition and heating temperature for typical clays in the soil groups.

CONCLUSIONS

The available literature shows that the laterisation processes, which mainly involve the free iron coating of soil particles and the binding together of smaller soil particles into larger ones, is considered to account for some engineering properties peculiar to lateritic soils.

The formation of both the lateritic and tropical clay soils studied herein is controlled by the particular weathering system. In the alternate wet and dry conditions of the forest zone, with a neutral-to-acidic subsurface environment and good internal drainage, lateritic profiles are formed. In the arid-to-dry coastal savanna zone, with a neutral-to-alkaline subsurface environment and poor internal drainage, tropical clay soils are formed.

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In terms of general characteristics, the tropical clay soil profiles are poor in laterite constituents (0-20%), very siliceous, rich in calcium carbonate (up to 15%), and have a high base exchange capacity and hygroscopic moisture content; the clay minerals found in these soils are kaolinite and montmorillonite with some vermiculite. Lateritic soil profiles, on the other hand, are very rich in laterite constituents (20-50%), have a low base exchange capacity and hygroscopic moisture content and are lacking in calcium carbonate content; the clay minerals in lateritic soils are kaolinite and gibbsite with some biotite.

Laboratory studies have shown that the differences between the physico-chemical properties of the lateritic and tropical clay soils may be attributed to the mode of formation (genesis) of the two soil types and to their clay mineralogies. The lower the degree of laterisation, the greater the influence of the clay content on the physico-chemical properties of the soil. In the highly laterised soils of the forest zone, which contain kaolinite and gibbsite, the physico-chemical properties are less significant than in the poorly laterised coastal savanna zone soils, which contain some montmorillonite in addition to kaolinite.

ACKNOWLEDGEMENTS

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INFLUENCE OF STRAIN RATE ON THE RESIDUAL STRENGTH OF A KAOLINITIC CLAY

B. K. RAMIAH* and P. PURUSHOTHAMARAJ⁺

INTRODUCTION

As a result of the rational explanations given by SKEMPTON (1964) as to the behaviour of slopes in stiff clays, sufficient research work has been directed to the study of the behaviour of such clays (and in general all clays) with particular reference to residual strength. Residual strengths are usually obtained from the reversal shear box technique, from continued shear displacement in the ring shear apparatus or from the triaxial test with a pre-cut slip plane in the sample. Differences in strength may arise from the various methods because of the limitations of each. However, tests with pre-cut planes appear to give reproducible results for given normal pressures.

The current literature reveals that the residual strength of natural clays or clay minerals may depend on factors like strain-rate (HVORSLEV, 1960; SKEMPTON, 1965; KENNEY, 1967; DE BEER, 1967), thixotropic effect (HVORSLEV, 1960), initial structure (SKEMPTON, 1964; KENNEY, 1967), ion concentration of pore fluid (KENNEY, 1967), clay fraction (BOROWICKA, 1965; SKEMPTON, 1964; KENNEY, 1967), etc.

A single test made at the WATERWAYS EXPERIMENT STATION (1951) (reported by HVORSLEV, 1960) has shown that, as the strain rate increases, the residual strength decreases at low strain rates, then appears to remain fairly constant over a wide range of deformations, and finally increases at very high strain rates. The results of SKEMPTON (1965) on cut samples of blue London Clay, of KENNEY (1967) on clay minerals and of RAMIAH et al (1970) on silty clay have revealed that the strain rate is insignificant. In the latter two cases, the effect of strain rate was studied by shearing the sample at an increased strain rate after the residual stage had been reached. On the other hand, the results of DE BEER (1967), both on intact and pre-cut samples, have shown high residual strengths at low rates compared to high strain rates.

In this investigation, the strain rate influence was studied by adopting a modified procedure of HUTCHINSON (1969).

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Discussion on this technical note is open until 1 November 1972.

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EXPERIMENTAL PROGRAMME

Samples and Sample Preparation

The soil samples used in this investigation were of a kaolinitic clay obtained from Bangalore, India. Identification test results of the soil are presented in Table 1.

Table 1. Identification test results

Atterberg Limits :	
Liquid Limit	66%
Plastic Limit	43%
Plasticity Index	23%
Grain Size Distribution :	
Sand	3%
Silt	86%
Clay (< 2 μ)	11%
Specific Gravity :	2.50
Activity :	2.05
Organic Content:	2.0%

The soil sample was air-dried and an amount of water equal to 80% of the dry weight of the soil was added and mixed thoroughly. The slurry was allowed to equilibrate for 24 hours. The sample was then placed in layers in a perforated inner mould 7.5 cm diameter surrounded by a solid outer mould of 10 cm diameter (see RAMIAH et al, 1970), and every attempt was made to remove air completely by shaking. The sample was then allowed to consolidate under its own weight for a day before being consolidated to pressures of 0.53, 1.0 and 1.5 kg/sq.cm by adopting double loading with a load duration of 24 hours. This procedure of sample preparation enabled relatively identical samples to be prepared.

Testing Procedure

The consolidated samples were mounted in a direct shear box (63 mm dia. \times 25 mm thick) and allowed to saturate further for 12 to 16 hours under the required normal load. All the samples were tested under normally consolidated conditions.

During the first forward strain, a low rate of strain of 0.02 mm/min. was applied for all tests and, after the peak, the samples were sheared further up to a deformation of 5 to 7 mm. Each sample was then cut with a thin wire

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and, for the subsequent reversals and forward displacements, the rate of strain was changed for each test. The ratio of strain rate for the first forward displacement to subsequent reversals and forwards was represented as a strain rate ratio; ratios of 1:1, 1:2, 1:4 and 1:8 were adopted for three values of normal pressure. After the first reversal, the sample was allowed to consolidate at the same pressure for 12 to 16 hours to allow for further moisture equilibration. The adoption of a slow rate for the first forward displacement allows for complete pore pressure dissipation and brings in the particles at the effective stress stage.

The structure at the residual stage was studied by the shrinkage ratio technique, for which a small core cutter (37 mm dia. and 14 mm thick) was pushed into the middle of the sample which had reached the residual strength stage, so as to have the shear zone at the middle of the cutter. The sample was air-dried and then oven-dried at 110°C for the concordant dry weight. The volume of the soil pat was determined by mercury displacement and the shrinkage ratio was represented in gm/cc. Tests were also run up to the peak for all normal pressures, and the shrinkage ratio at the peak stage was also determined.

DISCUSSION OF RESULTS

In Table 2, the results of peak strength for the three normal pressures at the 0.02 mm/min. strain rate and the residual strengths for the four strain rate ratios are presented. Figures 1 and 2 show typical stress-displacement curves for the two strain rate ratios of 1:1 and 1:8, for normal pressures of 1 kg/sq.cm. Stress-displacement curves for the other normal pressures showed similar trends for all strain rate ratios. It was found that the total displacement needed to bring a sample to the residual stage is greater at high strain rates than at low strain rates. At high strain rates, more disturbance occurs at the shear zone, more soil is squeezed out and there is a greater tendency for water to enter the shear zone; hence, more time elapses before the residual stage is reached than at low strain rates.

The average residual strength envelope (Fig. 3) is observed to be slightly curved at the initial stage, as found by KENNEY (1967) and SKEMPTON & PETLEY (1967). However, a straight line fitting yielded $c'_r = 0.05$ kg/sq.cm and $\phi' = 15.5^\circ$. It is observed that the residual strengths have a narrow range of variation of $\pm 1.5^\circ$ from the average for all the normal pressures and for all strain rate ratios (for $c'_r = 0$), except in one case where the variation was 2.6° at a high strain rate (Table 2). This behaviour is also presented graphically in Fig. 4.

The recent investigation by CULLEN & DONALD (1971) on speed effects on

Table 2. Residual strengths for different strain rates

Normal pressure (kg/sq.cm)	Shrinkage ratio at peak (gm/cu.cm)	Water content at peak(%)	Strain rate after first forward displacement (mm/min.)	Water content at residual (%)	Shrinkage ratio (gm/cu.cm)	Tan ϕ'_r (for $c'_r=0$) (degrees)	Deviation of ϕ'_r from average (for $c'_r=0$) (degrees)
0.53	1.28	50.4	0.02	43.60	1.39	0.400	21.1 + 0.7
			0.04	43.20	1.36	0.370	21.1 - 0.8
			0.08	43.10	1.35	0.365	21.1 - 1.1
			0.16	47.30	1.37	0.410	21.1 + 1.2
1.0	1.35	49.4	0.02	44.67	1.41	0.382	19.8 + 1.1
			0.04	46.87	1.35	0.382	19.8 + 1.1
			0.08	47.97	1.41	0.363	19.8 + 0.2
			0.16	47.60	1.38	0.310	19.8 - 2.6
1.5	1.36	45.8	0.02	43.20	1.42	0.275	16.9 - 1.5
			0.04	42.30	1.48	0.315	16.9 + 0.6
			0.08	41.80	1.38	0.325	16.9 + 1.1
			0.16	46.50	1.42	0.300	16.9 - 0.2

Note :- Strain rate for the first forward displacement in all cases was 0.02 mm/min.

TECHNICAL NOTE ON RESIDUAL STRENGTH

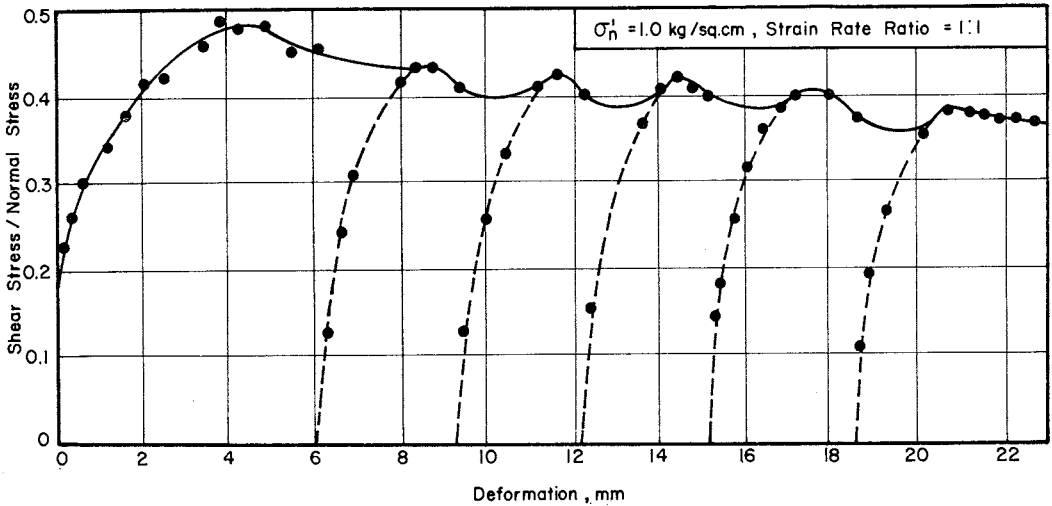


Fig. 1. Stress-displacement curve for a strain rate ratio of 1 : 1.

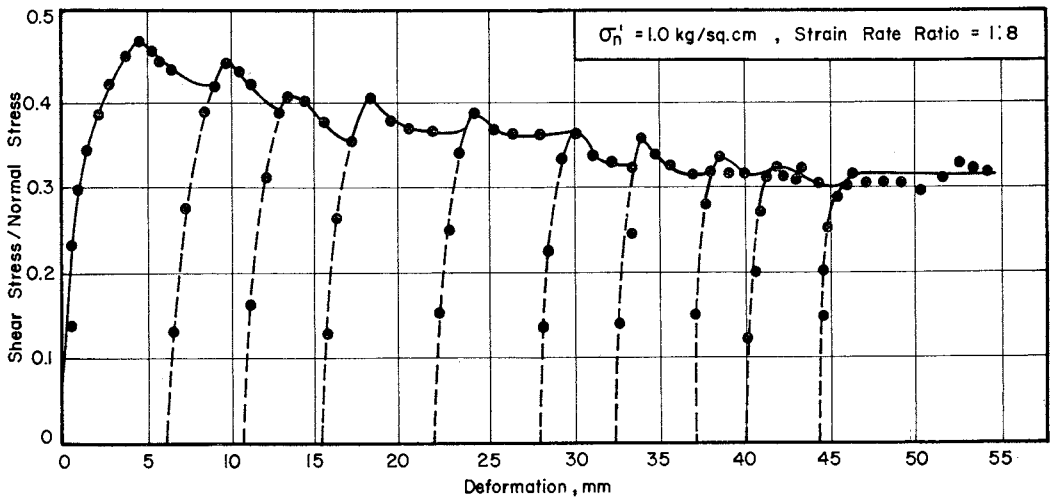


Fig. 2. Stress-displacement curve for a strain rate ratio of 1 : 8.

a decomposed Silurian clay (P.I. = 35%) showed that a rate of strain of 0.25 mm/min. (10×10^{-3} in./min.) was too fast to permit dissipation during shear; they reported that speeds of 0.07 to 0.125 mm/min. (0.67 to 5.0×10^{-3} in./min.), however, resulted in insignificant pore pressure build up. The soil used in the present investigation had a P.I. of 22.6%, which is much lower than that of the soil used by Cullen & Donald, and the adopted rate of strain for each strain rate ratio was relatively slow. At the end of intermediate cycles, the strength was probably not a fully drained strength,

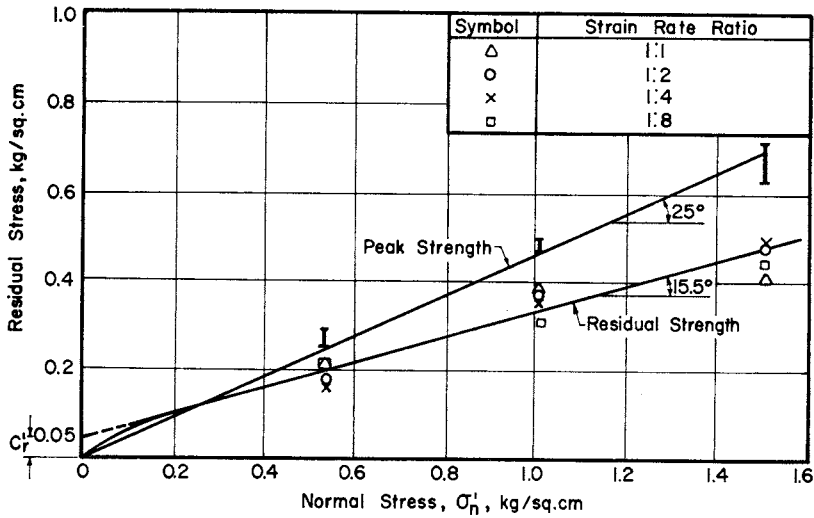


Fig. 3. Relationship between residual shear stress and normal stress.

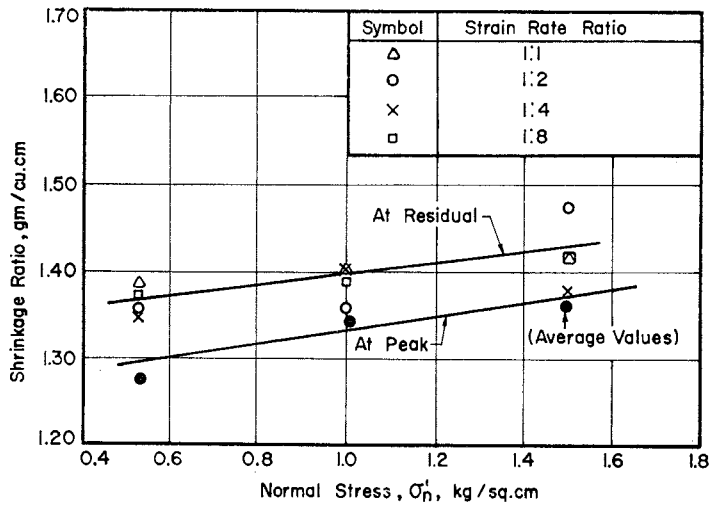


Fig. 4. Relationship between than ϕ'_r and normal stress.

because of the probability of the presence of some excess pore pressure. The strength at the residual stage of each test, however, is thought to have been the drained residual strength.

In Fig. 5, the residual stresses are plotted against strain rates for all the normal pressures. It can be seen that the strength is less at low strain rates and increases at high rates, which is in accordance with the findings of SKEMPTON (1965), KENNEY (1967) and RAMIAH et al (1970). At a very high rate, however, the strength reduces a little, probably due to a higher degree of

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disturbance, more room for water to enter and less time for complete pore pressure dissipation during each forward strain; this behaviour is clearly reflected in Fig. 2 by the scatter of the points at the last forward strain. The residual strength is practically constant for the strain rate ratios of 1:2 to 1:4 for all normal pressures.

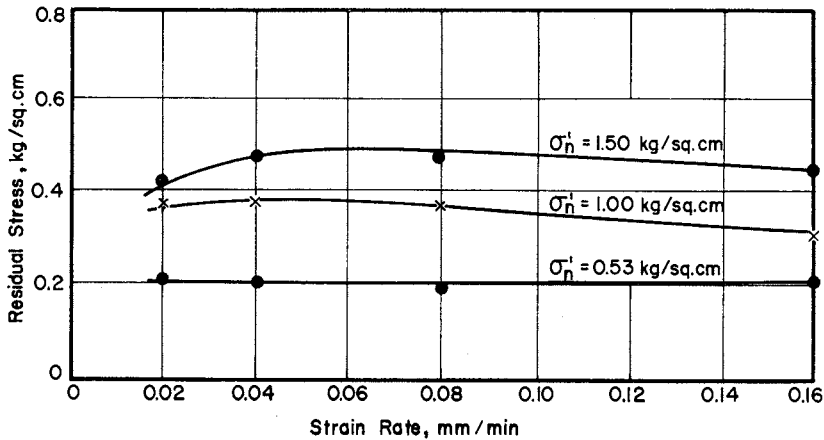


Fig. 5. Effect of strain on residual strength.

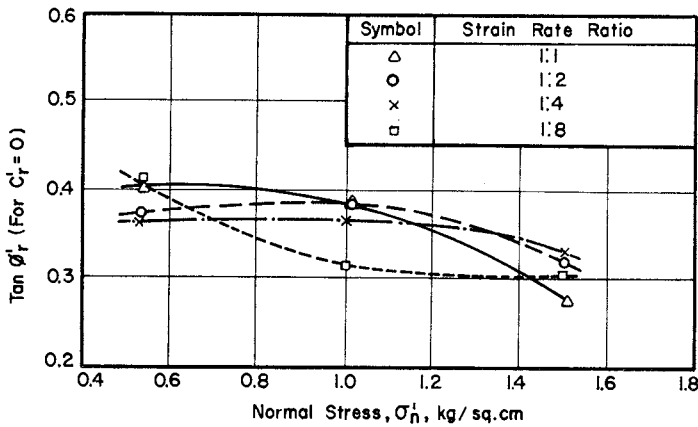


Fig. 6. Relationship between shrinkage ratio and normal stress.

According to LAMBE (1958), a clay with oriented particles will shrink more than a clay which has a random arrangement of particles and, consequently, the former will show a higher shrinkage ratio than the latter. Moreover, this amount of shrinkage has been observed to be a measure of both the force and orientation components (NARAIN & AYYAR, 1967). Here, the shrinkage ratio determined at the peak increases as the normal pressure increases (Fig. 6), quite evidently because of an increase in the degree of

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particle orientation as consolidation progresses. At the residual strain, the degree of orientation will be high and, consequently, a high shrinkage ratio is observed. The same trend for increase in shrinkage ratio with normal load increase is observed at the residual stage also. It is interesting to note that this increase is almost constant, as indicated by the two parallel lines.

CONCLUSIONS

The rate of strain does not seem to have a significant effect, provided a low strain rate is adopted for the first forward strain. Strain rate ratios of 1:2 to 1:4 may be employed for the sufficiently accurate determination of residual strength values. The shrinkage ratio may be used as a means to study the soil structure at the residual stage.

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INTERNATIONAL SOCIETY NEWS

The following is an abstract of a news letter received in October 1971 from the Secretary General of the International Society, Professor J.K.T.L. Nash

The International Society for Soil Mechanics and Foundation Engineering is best known for its International Conferences, seven of which have now taken place as follows: Harvard (1936), Rotterdam (1948), Zurich (1953), London (1957), Paris (1961), Montreal (1965) and Mexico City (1969). The next is due to take place in Moscow on 6 to 12 August, 1973. Bulletin No. 1 will shortly be sent to each National Society and it should be noted that summaries of papers to be included in the Proceedings will have to be received by the Organizing Committee not later than 1 March 1972, and the complete papers by 31 May 1972. The papers must be submitted through the members' National Societies, each of which has been given a quota, and who will clearly have to work to earlier dates. These conferences are intended for members of the International Society and their ladies only. Other persons wishing to attend must obtain the permission of the National Society in their country of residence, or of the Secretary General.

The International Society has an Executive Committee which has hitherto met at the time of an international conference, but this year they met in Sydney on August 4 and 5, mid-way in time between conferences. Their most important business was to discuss the plans for the Moscow conference, and the U.S.S.R. was represented by Professor N.A. Tsytoich and Mr. Yu G. Trofimenkov, who presented a draft of Bulletin No. 1. The Russian Organizing Committee has clearly been greatly helped by the response to the questionnaires circulated after the Mexico Conference, and by the report of the Conference Procedure Committee which was set up at that time with Dr. D.H. MacDonald (Canada) as its Chairman. The Executive Committee considered this report to be extremely valuable and believes that it will serve as a guide to future conferences.

Among other items of business, the following must be mentioned.

- (a) Chile was warmly welcomed as a new member country, so there are now 43 National Societies.
- (b) the German National Society reported that it would soon be in serious financial difficulties unless more subscribers to Geotechnical Abstracts could be found. [Each National Society has now been sent a list of the subscribers in its country and urged to try to double the number by the end of 1972. Unless the huge loss can be remedied the abstracts (which are extremely good) will have to come to an end.]

The *Geodex Retrieval System* was also commended by the Executive Committee. Full details about both can be obtained from National Societies.

Another, though less well recognized, activity of the International Society takes place at regional level. National Societies are divided among six regions as follows: Africa, Asia, Australasia, Europe, North America and South America. Each of these holds a regional conference at some time between international conferences, though North and South America hold a combined, Panamerican, conference. The fourth of these was held at San Juan, Puerto Rico, from 14 to 18 June, 1971. There were six formal technical sessions on the performance of earth structures and foundations, and the keynote address was given by Professor T.W. Lambe. The President, Professor Ralph B. Peck, acted as discussor-at-large to summarize and evaluate the formal technical sessions.

The Fourth Asian Regional Conference took place in Bangkok, Thailand on 26 to 30 July 1971 and was attended by more than 300 delegates and wives representing 21 countries. There were five formal technical sessions on various aspects of soil mechanics and foundation engineering, and special guest lectures were given by Professor J.K.T.L. Nash and Professor T.W. Lambe. In addition, a special symposium on Quality in Soil Sampling took place at the time of the conference. This was organized by the International Group on Soil Sampling and was a follow up to the Specialty Session on Soil Sampling held during the Mexico City conference.

Many of those who attended the Sydney Executive Committee also attended first the Bangkok conference, and subsequently the Australasian Regional Conference, termed the First Australia-New Zealand Conference on Geomechanics, which was held at Melbourne on 9 to 13 August, 1971. Here there were nine formal technical sessions and Professor T.W. Lambe gave the keynote address.

And finally, the Fifth Regional Conference for Africa took place at Luanda, Angola, from 23 to 28 August, 1971, with 83 delegates present. The relatively small numbers present made it possible to have discussion in depth.

That accounts for all the regional conferences for the 1969-73 period, with the exception of the Fifth European Regional Conference, which is to take place in Madrid, Spain, from 10 to 13 April 1972, on the theme *Structures Subjected to Lateral Forces*.

The Spanish National Society has recently suffered a severe loss in the premature death on 27 June of their President, Professor J.L. Escario, and the Executive Committee sent deepest sympathy in its loss to the Spanish Society.

J.K.T.L.N.

SOUTHEAST ASIAN SOCIETY NEWS

Report on the Fourth Asian Regional Conference

The Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering was held from 26 to 30 July, 1971 at the Dusit Thani Hotel in Bangkok. It was hosted by the Southeast Asian Society of Soil Engineering and sponsored by the Asian Institute of Technology. The Conference was formally opened by the Deputy Prime Minister of Thailand, H.E. Mr. Pote Sarasin, and was attended by more than 300 delegates and wives representing 21 countries. Over half of the delegates were from outside Thailand, the largest foreign delegation, of 58 persons, being from Japan.

Five technical sessions were held under the following titles:

- (1) Strength and stability problems.
- (2) Stress-strain characteristics and time effects.
- (3) Soil dynamics.
- (4) Structures on soft ground and pile foundations.
- (5) Soil technology and stabilization.

Each of the sessions was presided over by a Chairman, and featured a General Reporter and a panel of experts. The Chairmen of the sessions were Prof. H.B. Poorooshasb, Royal Iranian Geotechnical Institute; Prof. E.E. De Beer, Universities of Ghent and Louvain; Prof. J.G. Zeitlen, Israel Institute of Technology; Prof. Takeo Mogami (Vice-President for Asia), University of Tokyo; Dr. Sirilak Chandransu, Ministry of Communications, Thailand. The five General Reporters were Mr. Peter Lumb, University of Hong Kong; Prof. Laing Barden, University of Strathclyde; Dr. Kenji Ishihara, University of Tokyo; Prof. Chin Fung Kee, University of Malaya; Prof. Raymond Yong, McGill University.

A special *Symposium on Quality in Soil Sampling* was held in two sessions during the period of the Conference. This Symposium was organized by the International Grouping on Soil Sampling (IGOSS) and was a follow-up of the specialty session held during the Seventh International Conference in Mexico. It was chaired by Mr. John Dover from the Department of Applied Geomechanics, C.S.I.R.O., Melbourne.

An Open Forum on *The Role of Soil Engineering in the Economic Development of Asia* was also a special feature. This was thought to be most timely, since many countries in Asia are currently undergoing rapid development. The Forum was chaired by Dr. Edward W. Brand and was led by a group of distinguished civil engineers from several countries who represented government agencies, consulting engineers, contractors and academics.



From left to right : Dr E.W. Brand, Dr. Z.C. Moh, Mr. J. Dover, Prof. J.K.T.L. Nash, Dr. J. D. Nelson, Dr. Sirilak Chandrangsou, Prof. J.G. Zeitlen, Mr. P. Lumb, Prof. F.K. Chin, Prof. R. N. Yong, Prof. T. W. Lambe, Dr. K. Ishihara, Prof. T. Mogami, Prof. H.B. Poorooshab, Prof. L. Barden, Prof. E.E. De Beer.

The highlights of the conference were two guest lectures. The first, entitled "The Stability of Waste Tips" was presented by Prof. J.K.T.L. Nash of King's College, London, who is Secretary-General of the International Society for Soil Mechanics and Foundation Engineering. Professor T. William Lambe of Massachusetts Institute of Technology later lectured on "Accurate Predictions-Essential to Civil Engineering".

Two technical visits to current construction projects in the Bangkok area were organized for the participants, and a large exhibition of drilling, testing and construction materials and equipment was staged by manufacturers from six countries. The conference was successfully concluded with several tours outside Bangkok.

A total of 74 papers, contributed by authors from 16 countries, were accepted and published as Volume 1 of the Proceedings. The second volume of the Proceedings will contain the guest lectures, general reports, written discussions and details relevant to the other conference activities. The conference organizing committee was chaired by Dr. Za-Chieh Moh; Dr. John D. Nelson served as Conference Secretary and Dr. Edward W. Brand was chairman for the technical programme.

CONFERENCE NEWS

Eighth International Conference

The Eighth International Conference on Soil Mechanics and Foundation Engineering will be held in Moscow from 6 to 11 August, 1973. The Conference will feature four main technical sessions on the following themes :

- (1) Up-to-date methods for investigating the strength and deformability of soils (laboratory and field testing of soils for their strength, deformative and rheological properties).
- (2) Interaction of soil bases and structures (prediction of settlement, design of massive foundations based on the limiting state, design of flexible foundation beams and slabs).
- (3) Deep foundations, including pile foundations (design and new methods of construction).
- (4) Problems of soil mechanics and construction on structurally unstable and weak soils (collapsible, expansive, loess, saline soils, etc.).

In addition, eight specialty sessions will be held on the subjects of :

- (1) Equipment for the observation of settlements and reactions of bases.
- (2) Problems of nonlinear soil mechanics.
- (3) Statical design of earth and rockfill dams.
- (4) Soft soil bases of hydrotechnical structures.
- (5) Lateral pressure of clayey soils on structures.
- (6) Stability of slopes of deep excavations, and of structures on slopes.
- (7) Methods of soil stabilization (chemical, slurry trench construction, freezing, etc.).
- (8) Soil dynamics and seismic effects on foundations.

Each main session will have a General Reporter and a Co-Reporter, and will be preceded by a lecture given by an eminent specialist on the latest advancements in that particular field.

Intending authors must submit summaries of papers through their National Societies in time to reach the Organizing Committee in the U.S.S.R. by 1 March, 1972. Each National Society will be responsible for the selection of papers from those submitted by its members.

The registration fee for the Conference is U.S.\$ 60; those accompanying delegates will be charged \$ 25. Full information on registration procedure is given in Bulletin No. 1, which is now available from Secretary General, VIII ISSMFE, GOSSTROY USSR, Marx Prospect 12, Moscow K-9, U.S.S.R.

Fifth European Conference

The Spanish Society of Soil Mechanics and Foundations will stage the Fifth European Conference, entitled *Structures Subjected to Lateral Forces*, to be held in Madrid from 10 to 13 April, 1972. The previous European Conferences were held in Stockholm (1954), Brussels (1958), Weisbaden (1963) and Oslo (1967). Participation will be limited to members of European National Societies, although the Organizing Committee may invite or authorize the participation of other persons. The official languages of the Conference will be English, French and Spanish, and simultaneous translation will be provided in all three languages.

The Conference will be divided into four sessions, the topics for which will be :

- (1) General earth pressure theories, including the influence of displacements and time effects.
- (2) Stability of rigid structures.
- (3) Stability of flexible structures.
- (4) Construction problems and case stories.

All correspondence about the Conference should be addressed to Mr. José M.^a Rodríguez Ortiz, Secretario CEMS-72, Sociedad Espanola de Mecánica del Suelo y Cimentaciones, Laboratorio del Transporte y Mecánica del Suelo, Alfonso XII, n.º. 3, Madrid, Spain.

Third Southeast Asian Conference

The Third Southeast Asian Conference on Soil Engineering will be held in Hong Kong from 6 to 10 November, 1972. This follows those held in Bangkok in 1967 and in Singapore in 1970. The Conference is being sponsored by the Southeast Asian Society of Soil Engineering, the Hong Kong Engineering Society, The Institution of Civil Engineers (London), and the Asian Institute of Technology. The primary objectives of the Conference are :

- (1) To promote the advancement of soil engineering in the region.
- (2) To provide an opportunity for engineers to exchange ideas and experiences.
- (3) To review and discuss research in soil mechanics of particular relevance to Southeast Asia.

There will be no special theme for the Conference. Papers will be accepted on general topics of soil mechanics and engineering geology dealing with testing and site investigation, foundations, earth dams, slope stability, and roads and runways. Papers of a practical engineering nature may be given preference over purely theoretical studies. A provisional title and a brief

summary (less than 500 words) is required from each prospective author by 31 December, 1971.

A Symposium on Deep Pier and Caisson Foundations, dealing with the design, constructions, economics, and performance of such foundations will be the main feature of the Conference. The Symposium will be introduced by papers from invited authors.

The guest lecturer will be Professor Victor F.B. de Mello from Brazil.

Further information and Bulletin No. 1 are available from The Secretary III SEACSE, P.O. Box 13987, Hong Kong.

Conference on Earth and Earth Supported Structures

A Conference on the Performance of Earth and Earth Supported Structures will be held at Purdue University, Lafayette, Indiana, from 12 to 14 June, 1972 under the auspices of the American Society of Civil Engineers.

The technical sessions will be :

- (1) *Embankments on Soft Ground*—total, neutral, and effective stress distributions, and corresponding strain distributions, initially and as a function time, in the embankment and its foundation; stability and stabilization measures.
- (2) *Earth and Earth-Rockfill Dams*—stability and integrity of embankments; settlements and their effects; cracking of impermeable cores and membranes; measures to control and monitor seepage, including rapid drawdown; stability of reservoir slopes.
- (3) *Shallow Foundations*—isolated and combined footings and rafts in shallow or deep excavations; stabilizing measures, including preloading, compacted bearing strata, etc.; tolerable displacements of structures.
- (4) *Deep Foundations*—driven and bored piles, and caissons; construction problems; bearing capacity and settlement of pile foundations.
- (5) *Soil-Structure Interaction*—tunnels and conduits; locks and cantilever walls; braced and tie-back walls, slurry trenches, anchored bulkheads, cofferdams.

Each technical session has been assigned a Session Leader, a Reporter and Panel Members consisting of outstanding men in the field of soil engineering. Session leaders will prepare a historical review of the session topic, critically analyze papers submitted to the Conference, and lead Panel discussions. Panel members will discuss the Session Leader's report, establish areas of agreement, and debate conflicting viewpoints. The Reporter is assigned to synthesize Panel deliberations and discussions from the floor.

All correspondence should be addressed to Prof. G.A. Leonards, School of Civil Engineering, Purdue University, Lafayette, Ind. 47907, U.S.A.

Symposium on Strength and Deformation of Soils

A Symposium on the Strength and Deformation Behaviour of Soils will be held in Bangalore, India from 25 to 27 February 1972, under the auspices of the Indian Geotechnical Society (Mysore Centre) and Bangalore University. The primary objectives of the Symposium will be to provide an opportunity to collect and collate the experiences of researchers and field engineers in the field of strength and deformation behaviour of soils. It will also provide an opportunity for engineers, researchers and educators in the profession to exchange ideas and experiences and to discuss particular problems of the region.

Papers have been accepted on the following topics :

- (1) Physico-chemical aspects.
- (2) Shear strength.
- (3) Pore pressure.
- (4) Volume changes.
- (5) Properties of regional soils.
- (6) Structure-soil interaction studies.
- (7) Performance studies.

Information can be obtained from Dr. B.K. Ramiah, Chairman, Mysore Centre, Indian Geotec. Society, Visweswaraya College of Engineering, Bangalore 1, India.

Symposium on Relative Density

A Symposium on Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils will be held at the 75th Annual A.S.T.M. Meeting during the week of 25 to 30 June, 1972 in Los Angeles, California. The aim of the Symposium is (i) to provide a comprehensive review and evaluation of the maximum, minimum, and *in situ* density tests for determining relative density, and (ii) to establish the usefulness of relative density in geotechnical projects involving cohesionless soils.

The programme will consist of invited discussions on :

- (1) Determination of relative density considering means and reliability for measuring maximum, minimum and *in situ* or sample densities.
- (2) Correlation between direct and indirect determinations of relative density and measured performance of any kind; this may include examples from both field and laboratory studies.
- (3) Use and usefulness of relative density in geotechnical projects involving cohesionless soils.

Papers may be submitted which deal with any aspect of the Symposium topic; both short case studies and the results of extensive research are of

interest. Several of these will be selected for presentation, and others will be abstracted by the invited speakers. All suitable papers will be published in a special Proceedings volume together with the prepared presentations and discussions.

Potential contributors to this Symposium should contact, as soon as possible, either the Chairman, Ernest T. Selig, Dept. of Civil Engineering, State University of New York, Buffalo, N.Y. 14214, or Co-Chairman, Richard S. Ladd, Woodward-Moorhouse & Assocs. Inc., 1425 Broad Street, Clifton, N.J. 07012, U.S.A.

Earth Penetration Conference

A Conference on Rapid Penetration of Terrestrial Materials will be held at Texas A & M University from 1 to 3 February, 1972. Twenty-one papers have been accepted for the five sessions to be entitled :

- (1) Structural design and instrumentation of penetrometers.
- (2) Theory of penetration.
- (3) Laboratory testing.
- (3) Applications of penetrometers.
- (5) Field penetration tests and observations.

Mr. Robert L. McNeil will initiate the Conference with an invited paper on the State-of-the Art on Earth Penetration.

Registration fee for the Conference, including a set of the Proceedings, will be \$ 20. Forms for registration at the Conference and for reservation of accommodation are available upon request from Penetration Program Committee, Texas Engineering Experiment Station, Texas A & M University, College Station, Texas 77843, U.S.A.

European Symposium on Earthquake Engineering

The Fourth European Symposium on Earthquake Engineering will be held at Imperial College, London from 5 to 7 September, 1972. Three review papers will be given by invited speakers on the topics of (i) Modern trends in engineering seismology, (ii) Design methods, and (iii) Modern construction techniques.

Papers on the following topics will be included in the Symposium :

- (1) Codes of practice and building regulations in earthquake areas.
- (2) Earthquake resistant design of low-rise buildings with emphasis on new materials and new techniques.
- (3) Strong ground movements and foundation problems including liquefaction.

- (4) Case histories, such as analysis of failures of slopes, dams, buildings and structures.

The registration fee, which will include attendance at the Symposium, one copy of the Proceedings, and coffee, lunch and tea during the Symposium, will be approximately £ 40.

All correspondence should be addressed to The Secretary (Earthquake Engineering Symposium), Institution of Civil Engineers, Great George Street, London SW1, England.

Conference on Settlement of Structures

A Conference on the Settlement of Structures will be held at Cambridge University from 2 to 4 April, 1974. Early notice of this conference is given in the hope that it will encourage potential contributors to put in hand settlement measurements of structures should the opportunity now occur. There should be time for at least preliminary results of such measurements to be presented.

The subject matter of the Conference will be dealt with under the following headings :

- (1) Granular materials.
- (2) Normally consolidated and lightly over-consolidated cohesive materials.
- (3) Heavily over-consolidated cohesive materials.
- (4) Rocks.
- (5) Allowable and differential settlements, including damage to structures and soil-structure interaction.

It is intended that there should be full-length papers (not more than 5000 words) and short papers—'technical notes'—of not more than 1500 words. The latter will permit the presentation of isolated or limited case histories without full discussion. Prospective authors are requested to send synopses (200/250 words) in respect of full-length papers, and brief notification of subject regarding technical notes, to the Conference Secretary by 1 February, 1973. Final manuscripts of accepted papers will be required by 1 July, 1973.

For each session, there will be a General Reporter who will prepare a state-of-the-art paper covering existing knowledge as well as the data presented in papers and technical notes. Review papers, papers and technical notes will be issued to participants about one month before the Conference.

Correspondence about the Conference should be addressed to The Secretary, Settlement of Structures Conference, The Institution of Civil Engineers, Great George Street, London SW1, England.

International Clay Conference

The 1972 International Clay Conference will be held in Madrid, Spain from 25 to 30 June. It will be organized by the Spanish Clay Society (S.E.A.), under the auspices of Association Internationale Pour l'Etude des Argiles (A.I.P.E.A.), in cooperation with the University of Madrid, the National Research Council of Spain (C.S.I.C.) and the Geological and Mining Institute of Spain (I.G.M.E.).

The main purpose of the Conference is to promote international cooperation in the study of clays through scientific sessions, field trips and publications. Participants are invited from any country who are interested in clay science, soil science, ceramics and clay technology.

The titles of the planned sections of the Conference are :

- (1) Crystalchemistry of clay minerals (structures included).
- (2) Clay mineral genesis and synthesis.
- (3) Colloidal properties of clays.
- (4) Surface chemistry of clays (including catalytic properties).
- (5) Volume absorption phenomena (organic compounds included).
- (6) Technical properties and applications of clays and clay minerals.
- (7) General papers.

During the Conference, a joint meeting of the *Working Group of the Kaolin Correlation Program* is planned. This will include :

- (1) Discussion and approval of the ammended project proposal.
- (2) Conferences on the genesis of kaolin deposits.
- (3) Field trips to Spanish kaolin deposits.

Scientific papers relevant to this program will be included in the most appropriate Conference sections.

The Second Circular, and any information desired, can be obtained by writing to the Organizing Committee, 1972 International Clay Conference, c/o Departamento de Cristalografia y Mineralogia, Facultad de Ciencias, Seccion de Geologia, Ciudad Universitaria, Madrid-3, Spain.

Roorkee Earthquake Symposium

A Symposium on the Behaviour of Earth and Earth Structures Subjected to Earthquakes and Other Dynamic Loads is to be held at Roorkee, India, from 10 to 12 February, 1973. This Symposium is sponsored by the Indian Geotechnical Society, the Indian Society of Earthquake Technology, the Institution of Engineers (India) and the University of Roorkee.

Papers relevant to the theme of the Symposium, and of not more than 5,000 word-equivalents in length, are invited on the following topics :

- (1) Soil properties.
- (2) Retaining walls.
- (3) Earth dams.
- (4) Shallow foundations.
- (5) Deep foundations.
- (6) Theoretical studies on soil behaviour.

Bulletin No. 1 is now available from Dr. Shamsheer Prakash, Room 212, Civil Engineering Department, University of Roorkee, Roorkee (U.P.), India.

Conference on Rapid Excavation and Tunneling

The American Society of Civil Engineers and the American Institute of Mining, Metallurgical, and Petroleum Engineers have announced plans for a Conference on Rapid Excavation and Tunneling, to be held in Chicago from 2 to 7 June, 1972. The purpose of the conference is to provide a forum for the dissemination of new knowledge on the technology of underground, rapid machine excavation and tunneling.

The technical program for the Conference will include over 60 papers to be presented by industrial leaders and authorities in the field. These papers will be selected by a special Technical Program Committee representing both societies. It is also expected that some 50 commercial companies, educational institutions and government agencies concerned with the technology of underground rapid excavation and tunneling will present exhibits at the conference.

For further information, contact Mr. Alexander R. Scott, Conference Manager, AIME, 345 East 47th St., New York, N.Y. 10017, U.S.A.

International Geological Congress

The 24th International Geological Congress is to be held in Montreal, Canada from 21 to 30 August, 1972. The titles of the technical sessions will be :

- (1) Precambrian geology.
- (2) Petrology.
- (3) Tectonics.
- (4) Mineral deposits.
- (5) Mineral fuels.
- (6) Stratigraphy and sedimentology.
- (7) Paleontology.
- (8) Marine geology and geophysics.
- (9) Exploration geophysics.

- (10) Geochemistry.
- (11) Hydrogeology.
- (12) Quarternary geology.
- (13) Engineering geology.
- (14) Mineralogy.
- (15) Selection, storage, retrieval and processing of geological data.
- (16) Geological education.

All communications about the congress should be addressed to The Secretary General, 24th International Geological Congress, 601 Booth Street, Ottawa 4, Ontario, Canada.

Conference on Earthquake Engineering

The Earthquake Engineering Research Institute of the University of California at Los Angeles is sponsoring The National Conference on Earthquake Engineering to be held at U.C.L.A. during the week of 7 February, 1972. The conference will emphasize engineering aspects of the San Fernando Earthquake of 9 February, 1971.

The following topics in earthquake engineering will be covered :

- (1) Geotechnical aspects.
- (2) Structural aspects.
- (3) Utility systems.
- (4) Socio-economic aspects.
- (5) New concepts or new directions in earthquake engineering.

All correspondence should be addressed to Prof. R.B. Mathiesen, School of Engineering and Applied Science, 3173 Engineering I, University of California, Los Angeles, California 90024, U.S.A.

International Congress on Rheology

The Sixth International Congress on Rheology will be held in Lyon, France from 4 to 8 September, 1972. The scientific programme of the congress will be composed of general and specialized lectures as well as contributed papers. Papers on the following aspects of rheology will be presented during the congress :

- (1) Theory.
- (2) Measuring techniques.
- (3) Test methods.
- (4) Fractures.
- (5) Rheologic properties of materials (*including soils and rocks*).
- (6) Rheo-optic.
- (7) Bio-rheology.

(8) Miscellaneous.

Complete details about the programme and participation are obtainable from Dr. C. Smadja, B.P. No. 1, 69 - Lyon-Mouche, France.

Conference in Finite Element Methods

The Second Vanderbilt University Conference on the Application of Finite Element Methods in Civil Engineering will be held from 16 to 17 November, 1972. Papers are sought concerning the application of finite element methods in the following areas :

- (1) Static and dynamic structural analysis and design.
- (2) Soil mechanics and foundations.
- (3) Design and analysis of dams.
- (4) Ground water and seepage.
- (5) Computer methods.
- (6) Composite materials.
- (7) Design projects.

An award of \$ 100 will be given to the author of the best paper submitted and presented at the conference.

Those interested in submitting a paper should send a complete abstract (minimum of 500 words) for review by the Conference Publication Committee before 1 April, 1972 to Dr. Fred W. Beaufait, Vanderbilt University, Box 1533, Station B, Nashville, Tenn. 37203, U.S.A.

International Symposium on River Mechanics

An International Symposium on River Mechanics will be held in Bangkok from 9 to 12 January, 1973 under the auspices of the International Association for Hydraulic Research. The Symposium will be co-sponsored by the International Association of Hydrological Sciences, UNESCO and the Asian Institute of Technology. The official language of the Symposium will be English but papers may be submitted in French.

The Symposium will be divided into three technical sessions :

- (1) Flood investigation.
- (2) Erosion and sedimentation.
- (3) River and estuary model analysis.

Summaries of proposed papers must be submitted by 1 March, 1972; these should be in English and not more than 500 words in length. Final manuscripts will be required by 1 July, 1972.

Detailed information about the Symposium is given in Bulletins Nos. 1 and 2 which are now available from Dr. Subin Pinkayan, International

Symposium on River Mechanics, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

Congress on Tunnel Construction

It has been announced that the Swiss Society for Soil Mechanics and Foundation Engineering will organize a European Congress for Tunnel Construction on the theme *Problems of Yielding Rock Conditions in Tunnel Construction*. This will take place in Lucerne from 11 to 14 September, 1972. Further details will be available in due course.

NEWS OF PUBLICATIONS

Proceedings of the First International Conference

The *Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering*, held at Harvard University in June 1939, will be reprinted, provided that 100 orders are received in advance. The anticipated total price is U.S. \$ 40.00 for the three volumes (in full size and bound with hard covers), including postage. Orders should be placed with Dr. Steve J. Poulos, Geotechnical Engineering Inc., 934 Main Street, Winchester, Mass. 01890, U.S.A.

Proceedings of the Seventh International Conference

The three volumes of the *Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering*, held in Mexico City in August 1969, can be purchased for U.S.\$ 40. A 'state-of-the-art' volume is also available for an additional \$ 5.50. These are available from Sociedad Mexicana de Mecanica de Suelos, A.C./Apartado Postal 8200/Mexico 1, D.F.

Proceedings of the Southeast Asian Conferences

The volume of *Proceedings of the Second Southeast Asian Conference on Soil Engineering*, held in Singapore in June 1970, is now available at a price of U.S.\$ 18 with hard covers. There are still a few copies left of the *Proceedings of the First Southeast Asian Conference*, held in Bangkok in 1967 (U.S.\$ 20). Orders for these volumes should be sent to the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand. Cheques should be made payable to "Asian Institute of Technology".

Proceedings of the Fourth Asian Regional Conference

Orders are now being accepted for the *Proceedings of the Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Bangkok, July 1971 (2 volumes). Volume 1 is available now and volume 2 is expected to be printed early in 1972. Orders, together with payment of U.S.\$ 30, should be sent to the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

Proceedings of the First Australia-New Zealand Geomechanics Conference

Copies of the *Proceedings of the First Australia-New Zealand Conference on Geomechanics*, which was held in Melbourne in August 1971, are available

from the Institution of Engineers Australia, 157 Gloucester Street, Sydney, N.S.W. 200, Australia. The price is \$ 40 (Aust.) plus postage.

Proceedings of the Fourth Panamerican Conference

The *Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering*, held in Puerto Rico in June 1971, have been printed in three volumes. Volume I contains the six state-of-the-art papers, Volume II contains the conference papers, and Volume III contains discussion and the proceedings of the session on the Business and Practice of Foundation Engineering. These volumes are available separately at U.S.\$ 8 each, or as a set for \$ 20, from American Society of Civil Engineers, 345 East 47th Street, New York, N.Y. 10017, U.S.A.

Proceedings of the Fifth African Conference

The two volumes of *Proceedings of the Fifth Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, which took place in Luanda, Angola in August 1971, are available at U.S.\$ 40 from Laboratorio de Engenharia de Angola, Caixa Postal 6500, Luanda, Angola (Portugese West Africa).

Proceedings of the Symposium on Sampling of Soil and Rock

The *Proceedings of the Symposium on Sampling of Soil and Rock*, held in Toronto in June 1970, is now available as A.S.T.M. Special Technical Publication No. 483, for U.S.\$ 8, from American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pa. 19103, U.S.A.

Proceedings of the Symposium on Highway Geology

The *Proceedings of the 20th Annual Highway Geology Symposium*, held in Urbana-Champaign in April, 1970, is available at a price of U.S.\$ 3 from University of Illinois, Engineering Publication Office, 112 Engineering Hall, Urbana, Ill. 61801, U.S.A.

Proceedings of the Conference on Dynamic Waves

The *Proceedings of the Conference on Dynamic Waves in Civil Engineering*, held at the University College of Swansea in July 1970 (edited by D.A. Howells, I.P. Haigh and C. Taylor), can be purchased for £ 9.50 from John Wiley & Sons Ltd., Baffins Lane, Chichester, Sussex, England.

Proceedings of the Conference on Pile Foundations and Cellular Structures

The volume of *Proceedings of the Conference on Design and Installation of Pile Foundations and Cellular Structures*, held at Lehigh University in April 1970 (edited by H.T. Fang and T.D. Dismuke), is available at U.S.\$ 26.95 from Envo Publishing Co. Inc., P.O. Box 2326, Lehigh Valley, Pa. 18001, U.S.A.

Proceedings of the Fourth Roorkee Earthquake Symposium

The two volumes of *Proceedings of the Fourth Symposium on Earthquake Engineering*, held at Roorkee, India in November 1970, are available at a cost of U.S.\$ 20 (including postage) from Messrs. Sarita Prakashan, Nanahandi Grounds, Meerut (U.P.), India.

Proceedings of the IGOSS Symposium

The two volumes of *Proceedings of the IGOSS Symposium on Quality in Soil Sampling*, held in conjunction with the Fourth Asian Regional Conference in Bangkok in July 1971, are available at a price of U.S.\$ 8 from the Convener, IGOSS, c/o Division of Applied Geomechanics, CSIRO, P.O. Box 54, Mount Waverley, Victoria, Australia.

Proceedings of the Conference on Piles

The volume of *Proceedings of the Conference on the Behaviour of Piles* held in London in September 1970 is available, price £ 6, from Publications Sales Office, The Institution of Civil Engineers, Great George Street, London SW1, England.

Proceedings of the Vanderbilt Conference on Finite Element Methods

There are still a few copies left of the *Proceedings of the First Vanderbilt University Symposium on the Application of Finite Element Methods in Civil Engineering*, at a price of U.S.\$ 10.50, from Dr. Fred W. Beaufait, Box 1433, Station B, Vanderbilt University, Nashville, Tenn. 37203, U.S.A.

Latin American Geotechnical Journal

The first number of the new journal *Revista Latino Americana de Geotecnica* has recently been produced by the Venezuelan Society for Soil Mechanics and Foundation Engineering. Further details can be obtained from the Secretary of the Society, Apartado de Correos 4074, Caracas 101, Venezuela.

Geotechnical Abstracts

Under the sponsorship of the International Society, the German National Society of Soil Mechanics and Foundation Engineering have begun publication of their *Geotechnical Abstracts*. These abstracts provide a regular worldwide literature information service in the fields of soil mechanics, foundation engineering, rock mechanics and engineering geology. The abstracts are published monthly, at an annual subscription rate of U.S.\$ 32 plus postage, by Deutsche Gesellschaft für Erd-und Grundbau, 35a Kronprinzenstrasse, 43 Essen, Germany.

BOOK REVIEWS

Soil : Mechanics and Engineering by R.E. Scott and J.J. Schoustra, McGraw-Hill, 1968. U.S. \$ 8.95.

As stated in the *Preface*, this book is intended to be a first course in the mechanics and engineering of soils, and, to this end, it is divided into a 'theoretical' and a 'practical' part.

The basic principles of the mechanics of soils are treated in a very straight forward, simple manner which is reminiscent of Scott's *Principles of Soil Mechanics*. The coverage is good. The fundamental ideas of soil strength and compressibility are clearly and concisely dealt with in a very readable manner, but the reviewer was disappointed in the chapter on *Failure in Soils*, which seems all too brief and which greatly over-simplifies the factors involved in such a complex issue. The brevity of the treatment, moreover, has led to a rather inadequate treatment of some topics; in places, Scott resorts to the mere quotation of equations without indication as to their derivation. It is also to be regretted that, in the first half of the text, not a single reference is made to published sources other than books.

We are told in the *Preface* that "The entire text is aimed at demonstrating . . . that theoretical and practical soil mechanics are well matched to solve everyday soil engineering problems". It is difficult to believe that this aim has been achieved in the section of the book dealing with the practical aspects of soil mechanics. The subject matter in this half of the book is dealt with in a very superficial way and, contrary to the claim in the *Preface*, it does not tie in very well with the first section. Neither can this part of the book be called comprehensive. The treatment of piles and pile foundation is rather trivial, and earth structures are not dealt with at all. The worked examples in the section, however, are certainly a useful feature.

The whole book is extremely well produced, the general layout, and the figures and photographs, being of a particularly high standard. The theoretical half of the book can be recommended as an introductory text to undergraduate students, but there are many books already on the market which would probably be of more use to students and practicing engineers in their handling of everyday practical problems.

E.W. Brand

Foundation Analysis and Design by J.E. Bowles, McGraw-Hill, 1968. U.S. \$ 17

In his *Foundation Analysis and Design*, Bowles has produced an extremely useful and comprehensive reference book. Very full coverage is given to

both the soil engineering and structural engineering aspects of foundations. It contains a wealth of worked examples and there are very large numbers of invaluable figures and tables. The treatment is detailed and modern, the chapter on piles being particularly excellent and up-to-date. The appendix of computer programmes is a valuable addition to the book.

It is not felt that this book is particularly suitable for student use, since the whole aspect of the book is one of *reference* and not *text*. Unfortunately, the layout is not as attractive as it might have been; the reviewer found the tables in the form of computer outputs particularly difficult and unattractive to read.

To the practicing engineer, this book should prove to be an invaluable source of reference on all aspects of the design of foundations. It might well be the most modern, comprehensive treatment of the subject in existence. For this reason, it is to be strongly recommended.

E.W. Brand

Ground Water Resources Evaluation by W.C. Walton, McGraw-Hill, 1970, U.S. \$ 17.50

Professor Walton has an international reputation in the field of hydrology, and it is to be applauded that he has produced *Ground Water Resources Evaluation* at a time when a modern, comprehensive treatment of this subject is much in demand. It would be difficult to be disappointed with this book, since it covers very aspect of ground water that is of interest to engineers and geologists in a way that only a leading authority could. The vast volume of literature that exists on the subject has been so selectively abstracted and arranged for the reader that, in many cases, it would be found totally unnecessary to refer to the original sources. This is particularly true of the large chapter entitled *Aquifer Test and Flow Net Analysis*, but this will come as no surprise to those who are familiar with Walton's work. This single volume also probably contains the most useful collection of ground water case histories available in print.

It is a pity that consideration is given only to conditions in the United States, from where all the case histories are drawn; a slightly more international flavour would have been preferable. Apart from this small criticism, however, the book can be said to be excellent in every way. There can be little doubt that it will be immediately accepted as one of the standard references on the subject of ground water resources.

E.W. Brand

NOTES ON CONTRIBUTIONS TO THIS JOURNAL

Contributions to **Geotechnical Engineering** are invited from anyone. Items submitted to the Editor will be published under one of the following headings.

Original Papers

Original papers should be submitted in accordance with the *Notes for the Guidance of Authors* given inside the back cover of this journal. The Editor undertakes to acknowledge all manuscripts immediately they are received and to arrange for early review of each paper by *two* reviewers. The earliest possible publication date of contributions will be aimed for. Each Author will receive 25 free copies of his paper.

Technical Notes

Technical notes will be accepted for publication. These contributions should be presentations of technical information which might be useful to the practicing or research engineer but which are not sufficient in themselves to warrant a full paper. The format to be followed for technical notes is the same as that for papers but only *two* copies need be submitted and no *Synopsis* is required. The Author will receive 25 free copies of his technical note.

Reprints

Consideration will be given to reprinting papers which have been published previously but which are unlikely to have come to the attention of Society members. Only papers of a high standard which would be of particular interest to S.E.A.S.S.E. members will be considered.

Discussions

Discussion is invited on any of the papers published in this journal. The closing date for discussion is indicated at the foot of the first page of each paper. Discussions sent to the Editor may be in any form, but figures and references should comply with the general requirements for publications in this journal. *Two* copies are required.

News Items

As the official organ of the Southeast Asian Society of Soil Engineering, this journal will publish any news item of interest to the Society members. Items to be included in the next issue (June, 1972) should be sent so as to reach the editor not later than 15 April, 1972.