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LOAD DISTRIBUTION IN RECTANGULAR FOOTINGS ON PILES

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SYNOPSIS

The interaction with the subgrade of rectangular footings resting on piles is studied in order to determine the distribution of column loads on the piles and the subgrade. The piles are treated as independent elastic springs while the subgrade is treated as an elastic half-space. The finite difference technique is employed in the solution of the problem. A parametric study is carried out by varying the parameters which characterize the properties of the piles and the subgrade. The results obtained indicate that the resistance of the subgrade has considerable effect on the settlement of footings and should be taken into consideration in the analysis in order to predict the elastic settlement more realistically.

INTRODUCTION

In the design of a footing on piles, it is normally assumed that all the column load is carried by the piles and the resistance of the ground under the footing is completely neglected. In normal construction practice, however, a footing on piles rests on both the piles and the ground. A part of the column load is transmitted to the subgrade directly under the footing, and the remaining part of the load is carried by the piles. Therefore, to determine the distribution of the column load on the piles and the subgrade, and to predict the elastic settlement of the footing more realistically, the resistance of the subgrade must be included in the analysis in which the subgrade may be treated as an elastic medium.

The problem of slabs on elastic foundations has received considerable attention over the years. CHEUNG & ZIENKIEWICZ (1965) and CHEUNG & NAG (1968) studied the problems of slabs and tanks on isotropic elastic half-space type foundations and on Winkler foundations by using the finite element method. The separation of the contact surface in the interaction of slabs on grade with the foundation was investigated by VALANTAGUL et al (1971) who employed the finite difference technique. The behaviour of single piles and pile groups embedded in a homogeneous isotropic elastic medium was studied by POULOS & DAVIS (1968) and POULOS (1968) who used

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Mindlin's solutions. A similar study was made by BUTTERFIELD & BANERJEE (1971a) who considered the compressibility of piles in the analysis. The problem of a pile group with a rigid pile cap and its interaction with the subgrade was also studied by BUTTERFIELD & BANERJEE (1971b).

The purpose of this study is to investigate the distribution of a column load on the piles and the subgrade. For the pile footing shown in Fig. 1, the interaction of the pile cap with the subgrade will be studied by treating the pile cap as a flexible plate, and the finite difference method will be used to obtain solutions. The foundation will be treated as an isotropic elastic half-space, and the well-known Boussinesq solution will be used to determine the flexibility influence coefficient of the foundation. In this study, the piles will be treated as independent linear springs; by this simplifying assumption, the presence of the piles has no effect on the subgrade flexibility influence coefficient thus determined. The distribution of the column load between the piles and the subgrade will be studied for different types of clay ranging from soft clay to stiff sandy clay.

Although the method of analysis is applicable in general to any pattern of loading, in order to reduce the numerical work, only doubly symmetrical cases will be discussed.

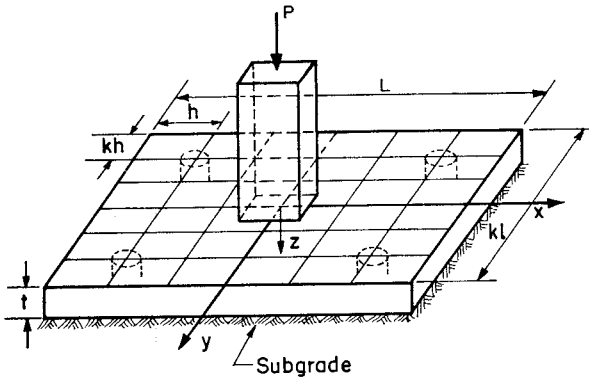


Fig. 1. Rectangular footing on piles and subgrade.

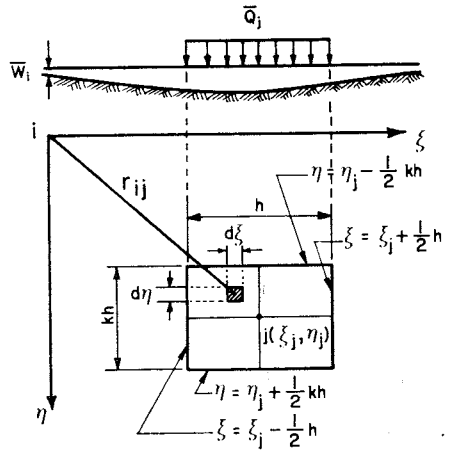


Fig. 2. Deflection at point *i* due to uniform load on elastic half-space.

METHOD OF ANALYSIS

Stiffness of Subgrade Surface

In the following discussion, only the vertical pressure on the footing is considered and the horizontal displacement due to vertical load is neglected. The vertical deflection at any point *i* (Fig. 2) due to vertical point load \bar{P}_j at point

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j on the surface of an isotropic elastic half-space is given by Boussinesq in the form :

$$W_i = \frac{(1-\nu_o) \bar{P}_j}{\pi E_o r_{ij}} \dots \dots \dots (1)$$

in which ν_o and E_o denote the Poisson's ratio and the Young's modulus respectively of the subgrade material and r_{ij} is the distance between points i and j .

The vertical deflection at any point i due to a uniform vertical pressure of magnitude \bar{Q}_j over the rectangular area shown in Fig. 2, with node j as its centre, can be obtained by integrating Eq. 1 over the rectangular area. This vertical deflection in the nondimensional form is :

$$w_i = \frac{\bar{f}_{ij}}{\gamma} \bar{q}_j \dots \dots \dots (2)$$

in which :

$$w = \frac{D}{Pkh^2} W \dots \dots \dots (3)$$

$$\bar{q} = \frac{\bar{Q}}{P/kh^2} \dots \dots \dots (4)$$

$$\gamma = \frac{12 \pi (1 - \nu^2) E_o}{(1 - \nu_o^2) E} \left(\frac{h}{t} \right)^3 \dots \dots \dots (5)$$

$$D = \frac{Et^3}{12 (1 - \nu^2)} \dots \dots \dots (6)$$

In these equations, D is the flexural rigidity of the footing, h is the spacing of the finite difference grid along the x -axis, k is the ratio of the spacing along the y -axis to that along the x -axis, P is the column load, w is the nondimensional vertical deflection of the subgrade surface, \bar{q} is the nondimensional contact pressure between the footing and the subgrade and ν , E and t denote the Poisson's ratio, Young's modulus and thickness of the footing.

The flexibility influence coefficient, \bar{f}_{ij} , can be determined from the following expressions. For $\xi_i \neq 0$ and $\eta_j \neq 0$:

$$\bar{f}_{ij} = \frac{1}{k^2} \left[\frac{c_1}{h} \sinh^{-1} \frac{d_1}{c_1} + \frac{c_2}{h} \sinh^{-1} \frac{d_2}{c_2} + \frac{d_1}{h} \sinh^{-1} \frac{c_1}{d_1} + \frac{d_2}{h} \sinh^{-1} \frac{c_2}{d_2} - \frac{c_1}{h} \sinh^{-1} \frac{d_2}{c_1} - \frac{c_2}{h} \sinh^{-1} \frac{d_1}{c_2} - \frac{d_1}{h} \sinh^{-1} \frac{c_2}{d_1} - \frac{d_2}{h} \sinh^{-1} \frac{c_1}{d_2} \right] \dots \dots (7)$$

For $\xi_j \neq 0$ and $\eta_j = 0$:

$$\bar{f}_{ij} = \frac{2}{k_2} \left[\frac{c_1}{h} \sinh^{-1} \frac{d_1}{c_1} + \frac{d_1}{h} \sinh^{-1} \frac{c_1}{d_1} - \frac{c_1}{h} \sinh^{-1} \frac{d_2}{c_1} - \frac{d_2}{h} \sinh^{-1} \frac{c_1}{d_2} \right] \quad (8)$$

Note that when $\xi_j = 0$ and $\eta_j \neq 0$, Eq. 8 can be applied after interchanging the coordinates ξ and η . Finally, for $\xi_j = 0$ and $\eta_j = 0$:

$$\bar{f}_{ij} = \frac{4}{k^2} \left[\frac{c_1}{h} \sinh^{-1} \frac{d_1}{c_1} + \frac{d_1}{h} \sinh^{-1} \frac{c_1}{d_1} \right] \quad \dots \dots \dots (9)$$

In Eqs. 7 to 9 :

$$c_1 = \eta_j + \frac{1}{2} kh \quad \dots \dots \dots (10)$$

$$c_2 = \eta_j - \frac{1}{2} kh \quad \dots \dots \dots (11)$$

$$d_1 = \xi_j + \frac{1}{2} h \quad \dots \dots \dots (12)$$

$$d_2 = \xi_j - \frac{1}{2} h \quad \dots \dots \dots (13)$$

For a finite difference grid system, the deflections of the subgrade surface can be written in matrix form as :

$$\{ w \} = \frac{1}{\gamma} [\bar{f}] \{ \bar{q} \} \quad \dots \dots \dots (14)$$

Inverting Eq. (14) yields :

$$\{ \bar{q} \} = \gamma [\bar{K}] \{ w \} \quad \dots \dots \dots (15)$$

in which :

$$[\bar{K}] = [\bar{f}]^{-1} \quad \dots \dots \dots (16)$$

is the nondimensional stiffness matrix of the subgrade surface.

In order to take advantage of double symmetry, the flexibility coefficient, \bar{f}_{ij} , is determined by superimposing the subgrade deflections at point i due to uniform pressure on the loaded areas over nodal point j and its symmetrical nodal points.

Load-Settlement Relationship of Piles

BARKAN (1962) suggested that the elastic deformation of piles can be linearly related to the loads. The data compiled by VONGTHIERES (1966) from pile loading tests conducted in Bangkok area show that, for a relatively low level of loading, the short time load-deformation relation of the piles is linear. This linear relationship is expressed in the form :

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$$\{P^*\} = [C] \{W^*\} \dots \dots \dots (17)$$

in which $\{P^*\}$ is the load vector on the piles, $[C]$ is the diagonal stiffness matrix of the piles and $\{W^*\}$ is the settlement vector of the top of the piles. Assuming that all the piles have the same stiffness, Eq. 17 becomes, in nondimensional form :

$$\{q^*\} = \beta [K_p] \{w^*\} \dots \dots \dots (18)$$

in which :

$$q^* = \frac{Q^*}{P/kh^2} \dots \dots \dots (19)$$

$$Q^* = \frac{P^*}{kh^2} \dots \dots \dots (20)$$

$$w^* = \frac{D}{Pkh^2} W^* \dots \dots \dots (21)$$

$$\beta = \frac{kh^2}{D} C \dots \dots \dots (22)$$

where q^* and Q^* are nondimensional and dimensional equivalent contact pressures on the nodal area contributed by the pile, w^* is the nondimensional settlement of the pile, C and β are the dimensional and nondimensional stiffness coefficient of the pile and $[K_p]$ is the matrix indicating the location of the pile.

Governing Equation

In view of Eq. 4 and the foregoing discussion, the governing equation for a plate resting on piles and subgrade can be written in the form :

$$k^2 h^4 \nabla^4 w + q' = q \dots \dots \dots (23)$$

in which :

$$q = \frac{Q}{P/kh^2} \dots \dots \dots (24)$$

where w is the nondimensional transverse deflection of the plate, Q and q are the dimensional and nondimensional applied load intensity on the plate, q' is the nondimensional combined contact pressure of subgrade and piles and ∇^4 is the biharmonic operator.

For a finite difference grid system, the equilibrium equation (Eq. 23) with appropriate boundary conditions can be written in matrix form :

$$[K] \{ w \} + \{ q' \} = \{ q \} \dots \dots \dots (25)$$

where $[K]$ is the nondimensional stiffness matrix of the plate. The nondimensional combined contact pressure between the footing and the subgrade is :

$$\{ q' \} = \{ \bar{q} \} + \{ q^* \} \dots \dots \dots (26)$$

In view of Eqs. 15 and 18 and the fact that $w_i^* = w_i$, substitution of Eq. 26 into Eq. 25 yields :

$$[K^*] \{ w \} = \{ q \} \dots \dots \dots (27)$$

where :

$$[K^*] = [K] + \gamma[\bar{K}] + \beta[K_p] \dots \dots \dots (28)$$

Solving Eq. (27) leads to :

$$\{ w \} = [K^*]^{-1} \{ q \} \dots \dots \dots (29)$$

The contact pressure between the plate and the subgrade and the load distribution on the subgrade and the piles can be readily obtained from Eqs. 15 and 18 once w is known.

Finite Difference Equations and Boundary Conditions

The elements of the stiffness matrix, $[K]$, are derived as follows. Adopting the grid system shown in Fig. 3, Eq. 23 can be written in finite difference form :

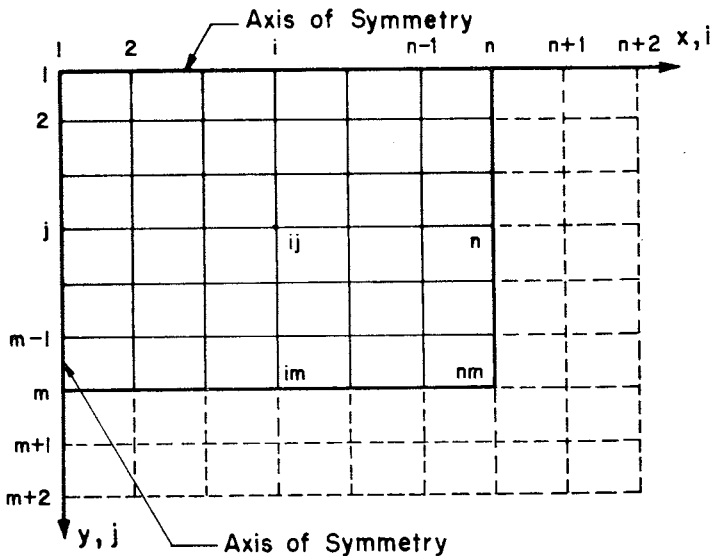


Fig. 3. Finite difference grid system.

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$$\begin{aligned}
 & k^2[w_{(i-2)j} + w_{(i+2)j}] + \frac{1}{k^2} [w_{i(j-2)} + w_{i(j+2)}] \\
 & + 2[w_{(i-1)(j-1)} + w_{(i+1)(j-1)} + w_{(i-1)(j+1)} + w_{(i+1)(j+1)}] \\
 & - 4(k^2 + 1) [w_{(i-1)j} + w_{(i+1)j}] - 4\left(\frac{1}{k^2} + 1\right) [w_{i(j-1)} + w_{i(j+1)}] \\
 & + \left(6k^2 + 8 + \frac{6}{k^2}\right)w_{ij} + q'_{ij} = q_{ij} \quad \dots \dots \dots (30)
 \end{aligned}$$

The boundary conditions of the plate for the edges perpendicular to the *x*-axis are :

$$M_x = -D \left[\frac{\partial^2 W}{\partial x^2} + \nu \frac{\partial^2 W}{\partial y^2} \right] = 0 \quad \dots \dots \dots (31)$$

$$V_x = Q_x - \frac{\partial M_{xy}}{\partial y} = -D \left[\frac{\partial^3 W}{\partial x^3} + (2 - \nu) \frac{\partial^3 W}{\partial x \partial y^2} \right] = 0 \quad \dots \dots (32)$$

and, for the edges perpendicular to the *y*-axis :

$$M_y = -D \left[\frac{\partial^2 W}{\partial y^2} + \nu \frac{\partial^2 W}{\partial x^2} \right] = 0 \quad \dots \dots \dots (33)$$

$$V_y = Q_y - \frac{\partial M_{xy}}{\partial x} = -D \left[\frac{\partial^3 W}{\partial y^3} + (2 - \nu) \frac{\partial^3 W}{\partial x^2 \partial y} \right] = 0 \quad \dots \dots (34)$$

At the four corners, the corner forces must vanish, i.e. :

$$R = 2M_{xy} = 2D(1 - \nu) \frac{\partial^2 W}{\partial x \partial y} = 0 \quad \dots \dots \dots (35)$$

In these equations, M_x , M_y , M_{xy} , Q_x and Q_y are the stress resultants in the plate, V_x , and V_y are the supplemented shearing forces and R is the corner force on the plate.

In view of Eq. 3, Eqs. 31 to 35 can be represented by the following difference equations which relate the nondimensional deflections of the nodal point of the plate in the vicinity of the boundary. At a generic point nj along the edge $i = n$ (Fig. 3), the difference equations corresponding to Eqs. 31 and 32 take the respective forms :

$$w_{(n+1)j} = -w_{(n-1)j} - \frac{\nu}{k^2} [w_{n(j-1)} + w_{n(j+1)}] + 2 \left(1 + \frac{\nu}{k^2}\right) w_{nj} \quad \dots \dots (36)$$

$$\begin{aligned}
 w_{(n+2)j} = & w_{(n-2)j} + \frac{2(2-\nu)}{k^2} [w_{(n-1)(j-1)} + w_{(n-1)(j+1)}] \\
 & - \frac{4(k^2 + 2 - \nu)}{k^2} w_{(n-1)j} + \frac{\nu(2-\nu)}{k^4} [w_{n(j-2)} + w_{n(j+2)}]
 \end{aligned}$$

$$\begin{aligned}
 & - \frac{4(2\nu - \nu^2 + k^2)}{k^4} [w_{n(j-1)} + w_{n(j+1)}] \\
 & + \frac{4k^4 + 8k^2 + 12\nu - 6\nu^2}{k^4} w_{nj} \dots \dots \dots (37)
 \end{aligned}$$

Similarly, along the edge $j = m$ at a generic point im , Eqs. 33 and 34 lead to :

$$\begin{aligned}
 w_{i(m+1)} = & - w_{i(m+1)} - \nu k^2 [w_{(i-1)m} + w_{(i+1)m}] \\
 & + 2(1 + \nu k^2)w_{im} \dots \dots \dots (38)
 \end{aligned}$$

$$\begin{aligned}
 w_{i(m+2)} = & w_{i(m-2)} + 2k^2(2 - \nu) [w_{(i-1)(m-1)} + w_{(i+1)(m-1)}] \\
 & - 4(1 + 2k^2 - \nu k^2)w_{i(m-1)} + \nu k^4(2 - \nu) [w_{(i-2)m} + w_{(i+2)m}] \\
 & + 4k^2(\nu^2 k^2 - \nu k^2 - 1) [w_{(i-1)m} + w_{(i+1)m}] \\
 & + (4 + 8k^2 + 12\nu k^4 - 6\nu^2 k^4)w_{im} \dots \dots \dots (39)
 \end{aligned}$$

At the corner point nm , Eq. 35 yields :

$$\begin{aligned}
 w_{(n+1)(m+1)} = & - 3w_{(n-1)(m-1)} - \frac{\nu}{k^2} w_{n(m-2)} - \nu k^2 w_{(n-2)m} \\
 & + 2\left(1 + \frac{\nu}{k^2}\right) w_{n(m-1)} + 2(1 + \nu k^2) w_{(n-1)m} \\
 & - \left(\frac{\nu}{k^2} + \nu k^2\right) w_{nm} \dots \dots \dots (40)
 \end{aligned}$$

In view of Eqs. 36 to 40, application of the boundary conditions gives the deflections at points $(n + 1)m$, $n(m + 1)$, $(n + 2)(m - 1)$, $(n + 2)m$, $(n - 1)(m + 2)$ and $n(m + 2)$ in terms of the deflections at the nodal points inside the plate as :

$$w_{(n+1)m} = 2w_{nm} - w_{(n-1)m} \dots \dots \dots (41)$$

$$w_{n(m+1)} = 2w_{nm} - w_{n(m-1)} \dots \dots \dots (42)$$

$$\begin{aligned}
 w_{(n+2)(m-1)} = & \frac{\nu(2 - \nu)}{k^4} w_{n(m-3)} + \frac{4(\nu^2 - k^2 - 2\nu)}{k^4} w_{n(m-2)} \\
 & + \frac{4k^4 + 8k^2 - 5\nu^2 + 10\nu}{k^4} w_{n(m-1)} + \frac{2\nu^2 - 4k^2 - 4\nu}{k^4} w_{nm} \\
 & + \frac{2(2 - \nu)}{k^2} [w_{(n-1)(m-2)} + w_{(n-1)m}] - \frac{4(k^2 + 2 - \nu)}{k^2} w_{(n-1)(m-1)} \\
 & + w_{(n-2)(m-1)} \dots \dots \dots (43)
 \end{aligned}$$

$$\begin{aligned}
 w_{(n+2)m} = & \frac{2\nu(2 - \nu)}{k^4} w_{n(m-2)} - \frac{4(2k^2 - \nu k^2 + 2\nu - \nu^2)}{k^2} w_{n(m-1)} \\
 & + \frac{4k^4 - 4\nu k^2 + 8k^2 - 2\nu^2 + 4\nu}{k^4} w_{nm} + \frac{4(2 - \nu)}{k^2} w_{(n-1)(m-1)} \\
 & - \frac{4(k^2 + 2 - \nu)}{k^2} w_{(n-1)m} + w_{(n-2)m} \dots \dots \dots (44)
 \end{aligned}$$

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$$\begin{aligned}
 w_{(n+1)(m+2)} = & (2 - \nu) \nu k^4 w_{(n-3)m} + 4k^2 (\nu^2 k^2 - 2\nu k^2 - 1) w_{(n-2)m} \\
 & + (4 - 5\nu^2 k^4 + 10\nu k^4 + 8k^2) w_{(n-1)m} + 2k^2 (\nu^2 k^2 - 2\nu k^2 - 2) w_{nm} \\
 & + 2k^2 (2 - \nu) [w_{(n-2)(m-1)} + w_{n(m-1)}] \\
 & - 4(1 + 2k^2 - \nu k^2) w_{(n-1)(m-1)} + w_{(n-1)(m-2)} \dots \dots (45)
 \end{aligned}$$

$$\begin{aligned}
 w_{n(m+2)} = & 2\nu k^4 (2 - \nu) w_{(n-2)m} - 4k^2 (2 - \nu + 2\nu k^2 - \nu^2 k^2) w_{(n-1)m} \\
 & + 2(2 - \nu^2 k^4 + 2\nu k^4 - 2\nu k^2 + 4k^2) w_{nm} \\
 & + 4k^2 (2 - \nu) w_{(n-1)(m-1)} + 4(\nu k^2 - 2k^2 - 1) w_{n(m-1)} \\
 & + w_{n(m-2)} \dots \dots \dots (46)
 \end{aligned}$$

The boundary conditions along the two axes of symmetry are :

$$w_{ij} = w_{i(-j)} \text{ along the } x\text{-axis} \dots \dots \dots (47)$$

$$w_{ij} = w_{(-i)j} \text{ along the } y\text{-axis} \dots \dots \dots (48)$$

The equilibrium equation (Eq. 30) together with Eqs. 36 to 48 can be used to set up the finite difference equations in terms of the deflections at the nodal points inside the plate.

NUMERICAL EXAMPLES AND PARAMETRIC STUDY

The centrally loaded square footing supported on four piles shown in Fig. 4 was selected for the parametric study. A preliminary study indicated

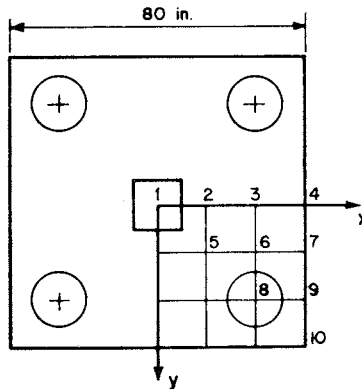


Fig. 4. Centrally-loaded square footing on piles and subgrade.

that the thickness of this type of footing within a practical range had insignificant effect on the distribution of load on the piles and subgrade; hence, thickness was kept constant in the parametric study. A 6 × 6 finite difference mesh was used in the calculation for which the pile spacing shown in Fig. 4 was adopted for convenience. The Young's modulus, *E*, and the Poisson's ratio *ν*, of the footing were assumed to be 3 × 10⁶ lb/in² and 0.15 respectively, and the thickness is taken as 20 in.

The Young's modulus for the subgrade, E_o , was varied from 600 to 6000 lb/in², which covers the range from soft clay to stiff clay, as shown in Table 1 (BOWLES, 1968). The corresponding Poisson's ratio was varied from 0.3 to 0.4. The adopted values of pile stiffness, C , were 0, 20, 40, 80, 160 and 240 ton/in., which were taken from VONGTHIERES (1966) and cover the cases of footings without piles and footings with short, medium and long piles. With this data, the parameter γ varies from 0.00247 to 0.02276, and the nondimensional stiffness, β , varies from 0 to 0.0452.

Table 1. Young's moduli for clay (BOWLES, 1968)

Type of Clay	Young's Modulus, E_o , lb/in ²
Very soft clay	50-400
Soft clay	250-600
Medium clay	600-1,200
Hard clay	1,000-2,500
Sandy clay	4,000-6,000

The column load, P , acting at node 1 (Fig. 4) is converted to a uniformly distributed load of intensity Q over the tributary area of node 1. Thus, the corresponding nondimensional applied load intensity on the plate is unity. The percentage of column load carried by the piles and the subgrade, the settlement at the centre of the footing for different values of the subgrade parameter, γ , and the total nondimensional pile stiffness $\sum\beta = 4\beta$ are plotted in Figs. 5 and 6. The contact pressures along the x -axis for the values of the parameters studied are plotted on a semi-log scale in Fig. 7. The settlements, bending moments and shear forces along the x -axis of the footing on medium piles are shown in Fig. 8.

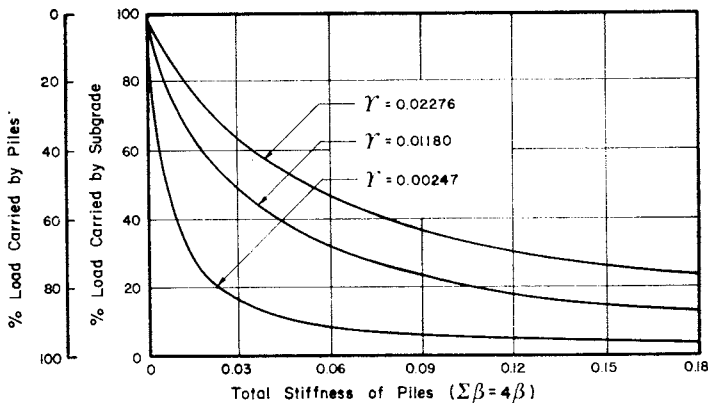


Fig. 5. Distribution of column load between piles and subgrade.

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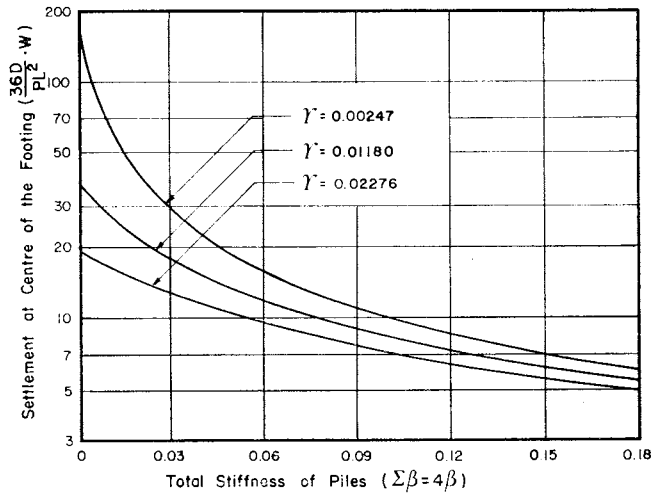


Fig. 6. Settlement at centre of footing.

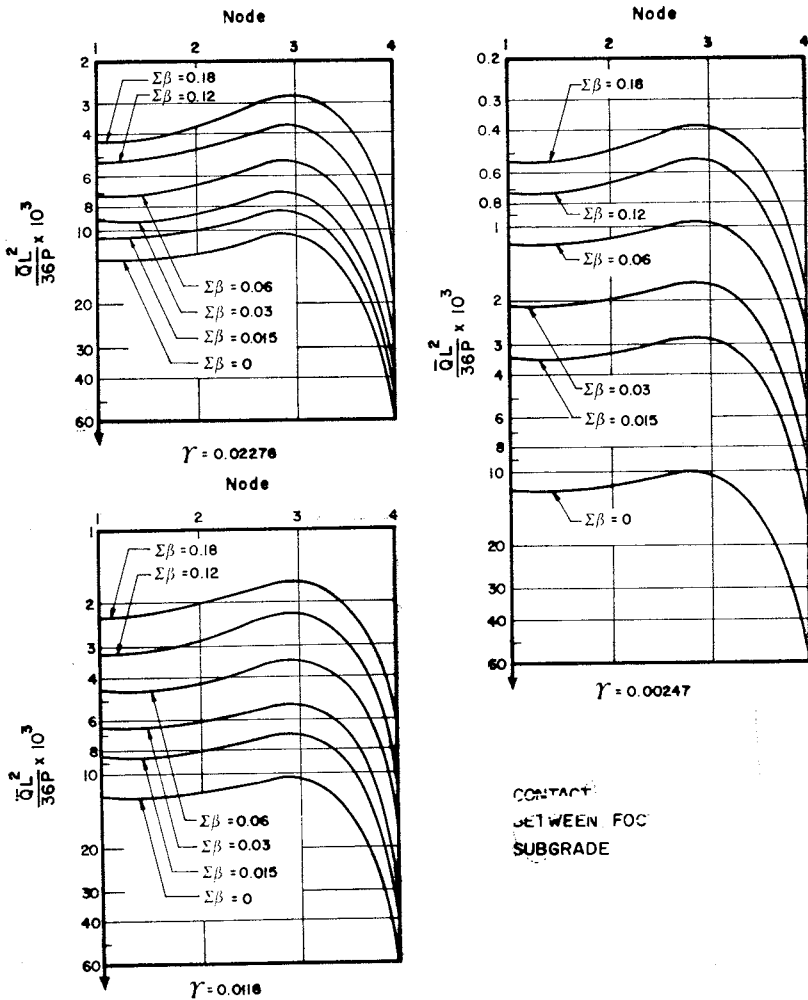


Fig. 7. Contact pressure along x-axis between footing and subgrade.

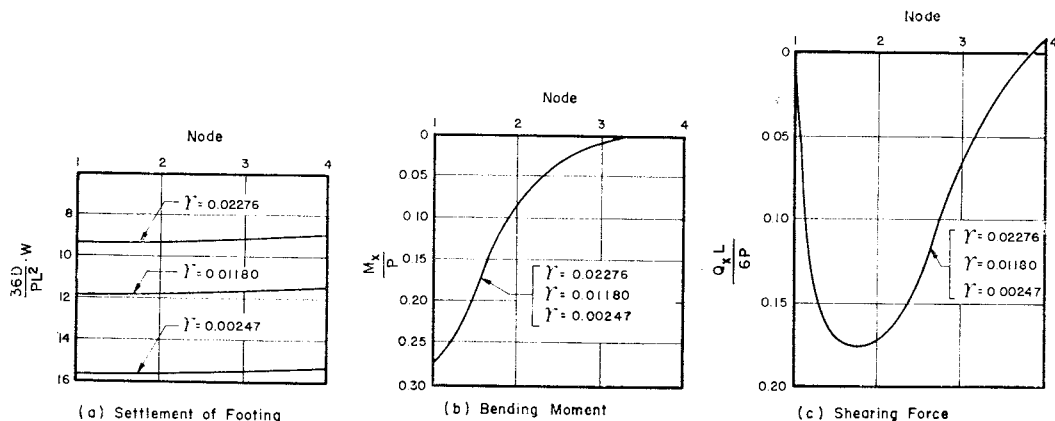


Fig. 8. Settlement and stress resultants of footing along x-axis ($\Sigma\beta = 0.06$).

DISCUSSION AND CONCLUSIONS

The results shown in Fig. 5 indicate that, for a pile footing on soft ground, most of the column load is carried by the piles, as expected, and the subgrade carries more and more load as it becomes stiffer. It is interesting to note that, for a very soft subgrade where $E_o = 600 \text{ lb/in}^2$, the medium stiffness piles ($\Sigma\beta = 0.06$) carry nearly the same load as the high stiffness piles ($\Sigma\beta = 0.18$). However, the settlement at the centre of the footing in the former case is approximately 2.7 times that in the latter, as indicated in Fig. 6.

For a stiff subgrade, a considerable amount of the column load is carried by the subgrade even when relatively stiff piles are used. Figure 5 shows that, for a footing with $E_o = 6000 \text{ lb/in}^2$ and $\Sigma\beta = 0.06$, the subgrade carries as much as 47% of the column load, and the corresponding nondimensional settlement of the footing indicated in Fig. 6 is approximately 9.3, which is about half of that for the footing resting on the subgrade only. In this particular case, if the resistance of the subgrade is completely neglected, the piles will carry all the column load and the settlement predicted will be approximately twice as large as the calculated value.

It should be emphasized that the settlement curves shown in Fig. 6 are plotted on a semi-log scale. The settlement increases rapidly as the stiffness of the piles decreases. If the resistance of the subgrade were not taken into account in the analysis, the settlement of the footing would be proportional to the stiffness of the piles. This indicates clearly the influence of the resistance of the subgrade upon the settlement of the footing.

The numerical results obtained in this study show that the footing studied was rather stiff, as indicated by the deflection curves shown in Fig. 8; this is normal in practice. Although the study was made for a footing with four

LOAD DISTRIBUTION IN FOOTINGS

piles, the results shown in Figs. 5 and 6 may be used as an estimate for square footings with, say, eight piles, as long as the footing is reasonably stiff.

It should be noted that the model used in this study may not be applicable to a rigid pile cap for the prediction of the distribution of the column load on each pile for a five-pile or nine-pile group. The use of this model leads to an equal distribution of the column load throughout the pile group, which has been shown to be unrealistic by WHITAKER (1957), SOWERS et al (1961), POULOS (1968) and BUTTERFIELD & BANERJEE (1971a, 1971b). These investigators have shown that the load in the outer piles of a group is larger than the load in the inner piles. In spite of this fact, the results obtained in this study may be used to predict the distribution of the column load on the piles and subgrade for a four-pile group with a rigid pile cap. The method of analysis, however, may be employed in the study of a pile group with a non-rigid pile cap, especially where the piles are considerably apart.

From this study it is clear that the resistance of the subgrade has considerably effect on the distribution of the column load and the settlement of a footing on piles. It should be taken into consideration in the analysis in order to obtain economical design and to predict the elastic settlements more realistically.

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APPENDIX : NOTATION

c_1, c_2, d_1, d_2	=	limits of integration.
C	=	elastic stiffness coefficient of pile.
D	=	flexural rigidity of plate.
E, E_o	=	Young's modulus of footing, subgrade material.
\bar{f}_{ij}	=	nondimensional flexibility coefficient of subgrade.
h	=	spacing of finite difference grid system.
k	=	ratio of finite difference grid spacing.
$[K], [\bar{K}]$	=	nondimensional stiffness matrices of plate, subgrade surface respectively.
$[K_p]$	=	matrix indicating location of the piles.
$[K^*]$	=	nondimensional combined stiffness matrix of system.
L	=	width of footing.
m	=	number of nodal points along y -axis.
M_x, M_y	=	bending moments of plate.
M_{xy}	=	twisting moment of plate.
n	=	number of nodal points along x -axis.
P	=	column load.
\bar{P}	=	point load on subgrade.
P^*	=	load acting on pile.
q, Q	=	nondimensional, dimensional applied load intensities on footing.
\bar{q}, \bar{Q}	=	nondimensional, dimensional contact pressure.
q^*, Q^*	=	nondimensional, dimensional contact pressure on piles.
q'	=	nondimensional combined contact pressure between subgrade surface and piles.
Q_x, Q_y	=	shearing forces of footing.
r_{ij}	=	distance between points i and j .
R	=	corner force on plate.
t	=	thickness of plate.
V_x, V_y	=	supplemented shearing forces in plate.
w, W	=	nondimensional, dimensional transverse deflections of footing.
$\{w^*\}, \{W^*\}$	=	nondimensional, dimensional settlement vectors of piles.
x, y, z, ξ, η	=	coordinate axes.
β	=	nondimensional stiffness coefficient of pile.
γ	=	$\frac{12\pi(1-\nu^2)E_o}{(1-\nu_o^2)E} \left(\frac{h}{t}\right)^3$.
ν, ν_o	=	Poisson's ratio of footing material, subgrade

STRESS HISTORY EFFECTS ON STRESS-STRAIN BEHAVIOUR OF A SATURATED CLAY

A.S. BALASUBRAMANIAM*

SYNOPSIS

This paper summarizes data illustrating the effects of stress history on the stress-strain behaviour of remoulded specimens of kaolin tested in the conventional triaxial apparatus under stress controlled conditions. The effects considered are initial one-dimensional consolidation stress used in sample preparation, load increment, magnitude of isotropic consolidation stress prior to shear, and type of applied stress path. It is shown that these factors influence the deformation characteristics of a saturated clay. Methods which take account of these effects are proposed to correlate the test results. The results are useful in investigating the possibility of including these effects in the existing stress-strain theories.

INTRODUCTION

The successful development of a theory for the adequate description of the stress-strain relationship for clays requires a detailed experimental investigation of the material behaviour. Often, the stress history experienced by the sample prior to shear and the stress increments used during testing influence the stress-strain behaviour. A knowledge of the magnitude of these effects is needed to establish approximate limits for the variability of the test results.

The major factors that influence the stress-strain behaviour of remoulded specimens during shear are:

- (i) initial one-dimensional consolidation stress used in sample preparation,
- (ii) load increment,
- (iii) magnitude of isotropic consolidation stress prior to shear,
- (iv) type of applied stress path during testing.

The effects of these factors on the stress-strain behaviour are discussed in detail and methods are proposed to correlate the test results.

STRESS AND STRAIN PARAMETERS

The stress parameters p and q are defined by :

$$p = (\bar{\sigma}_1 + 2\bar{\sigma}_3)/3$$
$$q = (\bar{\sigma}_1 - \bar{\sigma}_3),$$

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where $\bar{\sigma}_1$, $\bar{\sigma}_2$ and $\bar{\sigma}_3$ are the principal effective compressive stresses, and $\bar{\sigma}_2 = \bar{\sigma}_3$ under the triaxial stress system. Similarly, the strain parameters v and ϵ are given by

$$v = \epsilon_1 + 2\epsilon_3$$

$$\epsilon = 2(\epsilon_1 - \epsilon_3)/3$$

where ϵ_1 , ϵ_2 and ϵ_3 are the principal compressive strains, and $\epsilon_2 = \epsilon_3$ under the triaxial stress system.

MATERIAL TESTED, SAMPLE PREPARATION AND TESTING PROCEDURE

All specimens were prepared from air-dried kaolin (liquid limit = 72%, plastic limit = 42% and specific gravity = 2.61) mixed with water to a slurry. Unless otherwise stated, the initial moisture content of the slurry was 160% and the slurry was one-dimensionally consolidated in a special former to a maximum pressure of 22.6 lb/in². Subsequently, the former was removed and the sample was isotropically consolidated to the required cell pressure. For a detailed description of the sample preparation and testing procedure see BALASUBRAMANIAM (1969).

EFFECT OF INITIAL ONE-DIMENSIONAL CONSOLIDATION STRESS

Figure 1 illustrates the stress paths followed by three specimens which were subjected to different one-dimensional consolidation pressures and subsequently brought to the same isotropic stress. In this figure, the points A, B and C correspond to specimens AU, AB and AR which were subjected to one-dimensional consolidation stresses of 11, 22 and 55 lb/in² respectively.

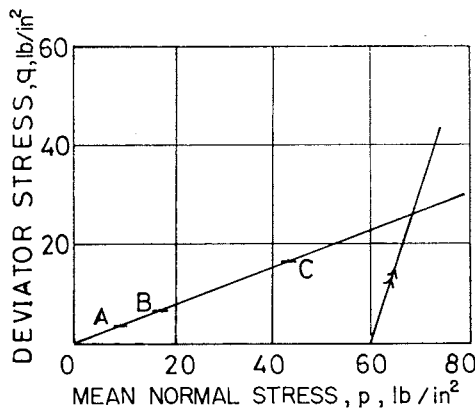


Fig. 1. States of three specimens in the (q, p) stress plane at the end of one-dimensional consolidation and isotropic consolidation.

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Experimental Observation on Fully Drained Tests

Since all the specimens were subjected to the same applied stress paths of slope 3 commencing from the same isotropic stress of 60 lb/in², any stress point (q, p) can be uniquely represented by the parameter q/p . Figures 2 and 3 illustrate the variations of the volumetric and shear strains with respect to this parameter. It is clear that (i) at any particular stress ratio, specimens with high initial one-dimensional consolidation pressure have low volumetric and shear strains, and (ii) except for the specimen with the one-dimensional consolidation stress of 55 lb/in², the ($q/p, v$) characteristics are the same. Hence, any further reduction of the one-dimensional consolidation stress below

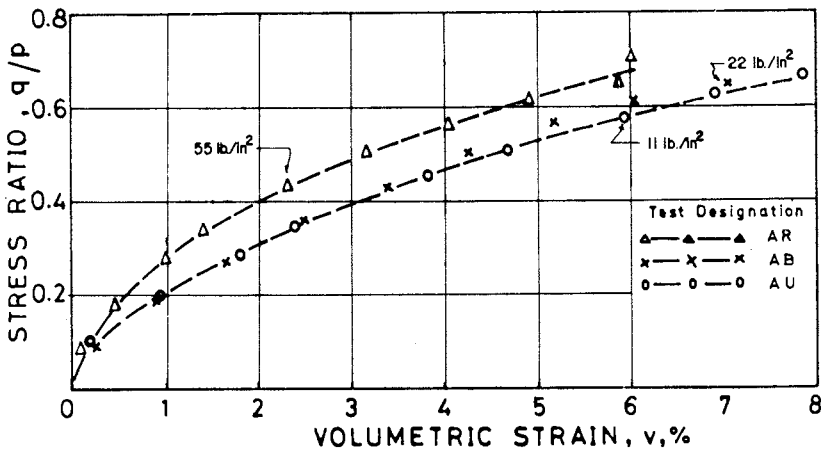


Fig. 2. The ($q/p, v$) characteristics of specimens prepared with different initial one-dimensional stress and sheared under fully drained conditions from an isotropic stress of 60 lb/in² with constant cell pressure.

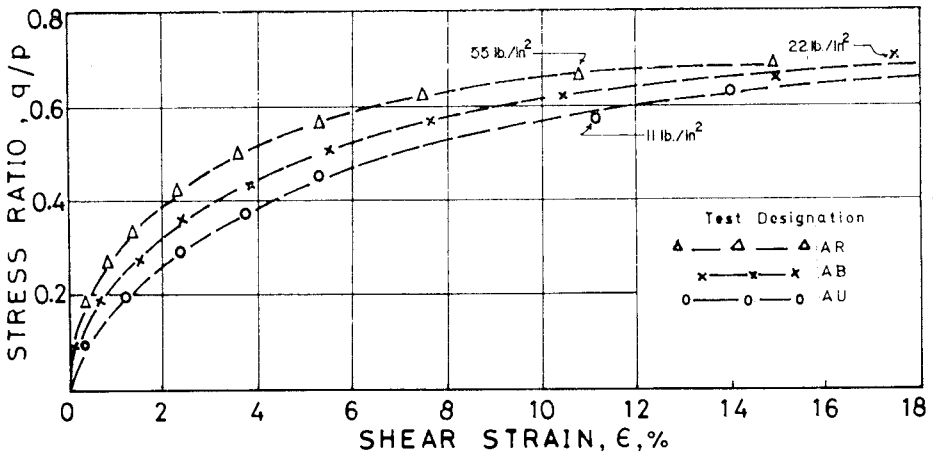


Fig. 3. The ($q/p, \epsilon$) characteristics of specimens prepared with different initial one-dimensional stresses and sheared under fully drained conditions from an isotropic stress of 60 lb/in² with constant cell pressure.

11 lb/in² will not have an appreciable effect on the volumetric strain-stress ratio relationship.

Correlation of Drained Test Results with Initial Consolidation Stress

The assumption made in the calculation of the initial shear stress experienced by a specimen during one-dimensional consolidation is that the ratio of the minor principal stress to the major principal stress is a constant during the initial one-dimensional consolidation. This ratio is taken to be 0.7 (K_o) and is in agreement with the lateral pressure measurements carried out by THOMPSON (1962) and BURLAND (1967), during one-dimensional consolidation of cylindrical specimens. The corresponding value of q/p is 0.375.

Two methods (designated as Method I and Method II) will be studied for the correlation of the observations on the specimens prepared with different initial one-dimensional stresses. These methods are based on two different additional assumptions.

Method I—In this method, it is assumed that there is a unique relationship between q/p and v , and q/p and ϵ during shear under a given applied stress path of all specimens if they are initially prepared under truly isotropic stress conditions (i.e. the specimens are assumed to be initially isotropically consolidated from the slurry, instead of being subjected to a one-dimensional consolidation). Since all specimens prepared by the author were initially sheared to a stress ratio of 0.375 during one-dimensional consolidation, one would expect them to exhibit strains of small magnitude during the initial

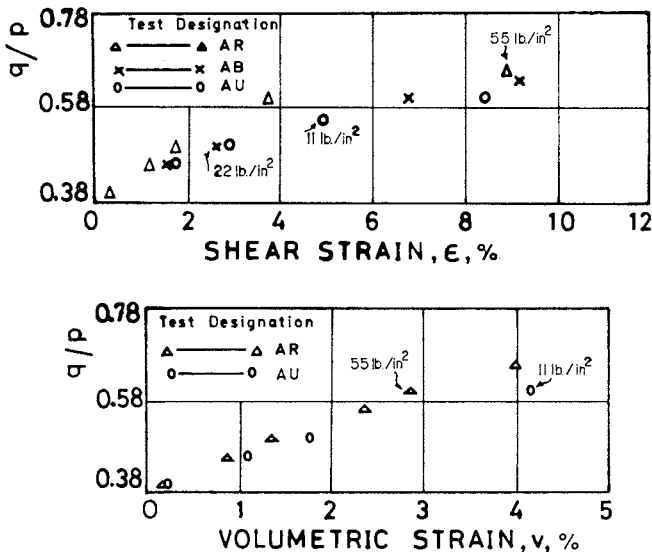


Fig. 4. The $(q/p, \epsilon)$ and $(q/p, v)$ characteristics beyond a stress ratio of 0.375.

STRESS-STRAIN OF CLAY

phase of the subsequent shear in the triaxial apparatus, until the stress ratio of 0.375 (experienced by the specimen during one-dimensional consolidation) was exceeded. Therefore, the behaviour of all these specimens beyond a stress ratio of 0.375 should be identical in the $(q/p, v)$ and $(q/p, \epsilon)$ spaces, irrespective of their initial one-dimensional consolidation stress. Figure 4 illustrates the $(q/p, \epsilon)$ and the $(q/p, v)$ relationships of the specimens for stress ratios higher than 0.375. It is observed that, even after allowing for the initial shear effects based on the K_0 condition, the stress-strain behaviour in the $(q/p, v)$ and the $(q/p, \epsilon)$ spaces is not unique.

Method II—In this method, only one different assumption is made to those used in Method I. In the previous method, it was assumed that the stress-strain behaviour during shear of all the specimens might be unique once the stress ratio, q/p , had exceeded the value corresponding to the K_0 condition. In Method II, it is assumed instead that the behaviour of all specimens might be unique once the deviator stress, q , has exceeded the value attained during the initial one-dimensional consolidation. The initial shear stresses experienced by the three specimens prepared under the initial one-dimensional stresses of 11, 22 and 55 lb/in² were 3.3, 6.6 and 16.5 lb/in² respectively. In this method of correlation, it is suggested that the behaviour of the three specimens before each reaches its initial shear stress of 3.3, 6.6 and 16.5 lb/in² respectively may be different. However, thereafter the behaviour of the specimens should be identical. The modified stress-strain relationship for the specimens AU, AB and AR are shown in Figs. 5 and 6. It is observed that the behaviour of the three specimens in Figs. 5 and 6 are still not unique. The correlation is somewhat better, however, than that produced by Method I.

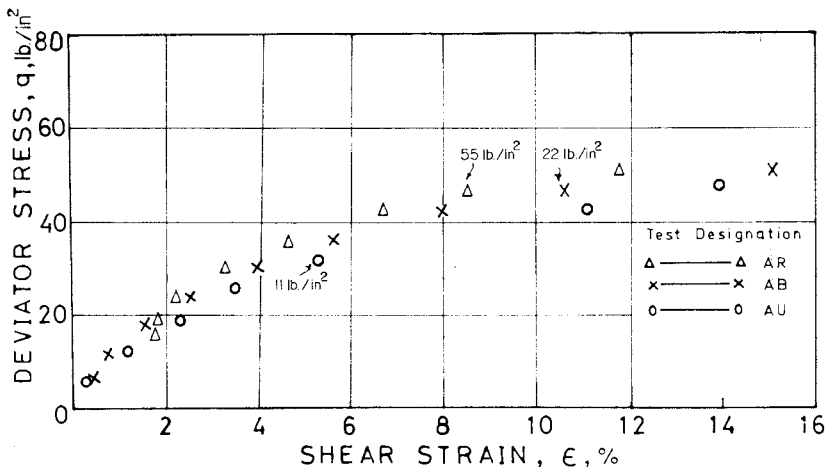


Fig. 5. The (q, ϵ) characteristic of Specimen AU, and the modified (q, ϵ) characteristics of Specimens AR and AB.

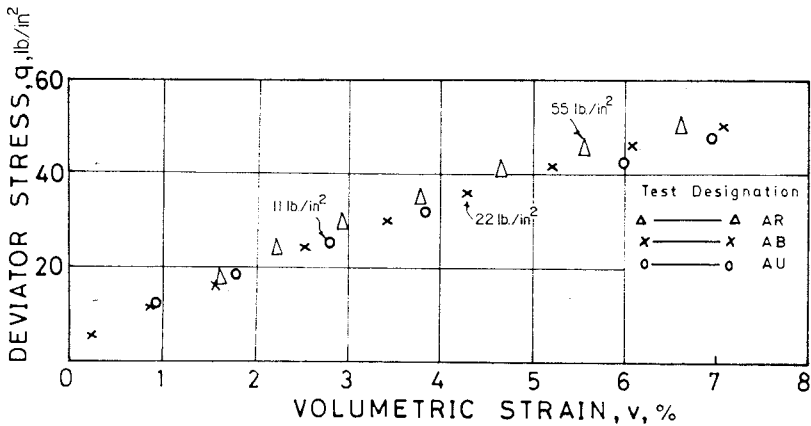


Fig. 6. The (q, v) characteristics of Specimen AU, and the modified (q, v) characteristics of Specimens AR and AB.

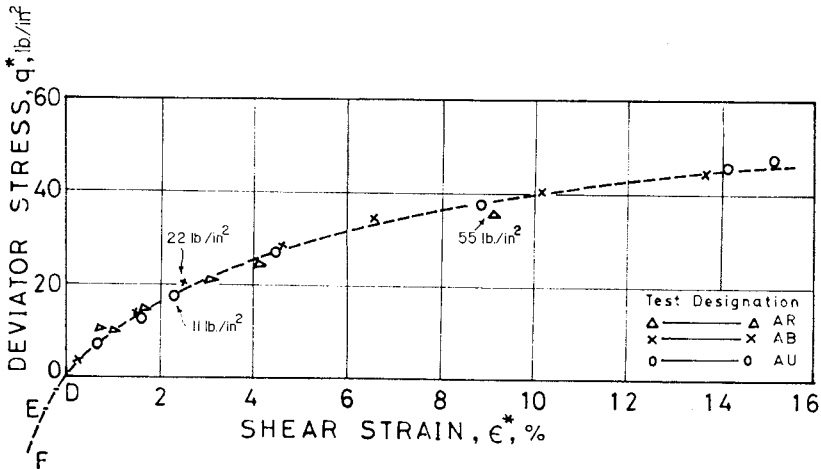


Fig. 7. The (q^*, ϵ^*) characteristics of Specimens AR, AB and AU prepared under different initial one-dimensional consolidation stresses and sheared under fully drained conditions from an isotropic stress of 60 lb/in^2 with constant cell pressure.

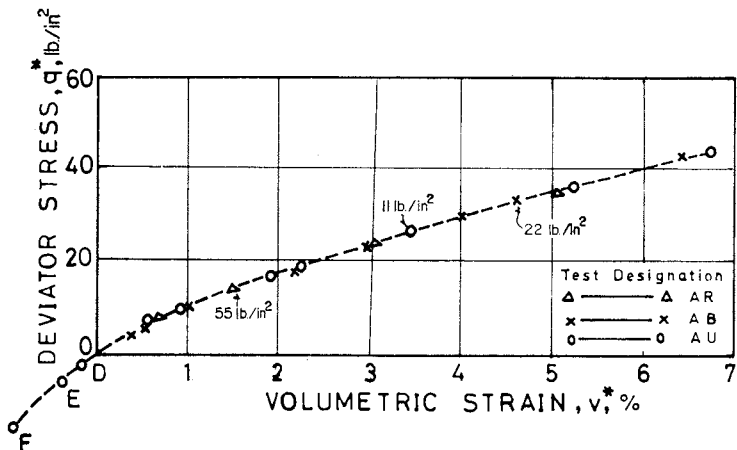


Fig. 8. The (q^*, v^*) characteristics of Specimens AR, AB and AU prepared under different initial one-dimensional consolidation stresses and sheared under fully drained conditions from an isotropic stress of 60 lb/in^2 with constant cell pressure.

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Shift of Origin of Stress-Strain Curve with Consolidation Stress

An alternative method by which the author was able to correlate the stress-strain behaviour of the three specimens satisfactorily will now be discussed. In this method, it is suggested that the behaviour of the specimens in the plots (q^*, ε^*) and (q^*, v^*) are unique, where :

$$q^* = q - q_{K_0} \dots \dots \dots (1)$$

$$v^* = v - v_{K_0} \dots \dots \dots (2)$$

$$\varepsilon^* = \varepsilon - \varepsilon_{K_0} \dots \dots \dots (3)$$

In these expressions, q_{K_0} is the magnitude of the initial shear stress due to one-dimensional consolidation, and v_{K_0} and ε_{K_0} correspond to volumetric and shear strain experienced by the specimens during the subsequent shear in the triaxial apparatus up to a deviator stress of q_{K_0} . The results are plotted as the modified plots (q^*, ε^*) and (q^*, v^*) in Figs. 7 and 8. The unique relationships revealed by these plots indicate that the origin of the stress-strain curve, denoted by D, E and F in the (q, ε) and (q, v) plots, shift by amounts $(-q_{K_0}, -\varepsilon_{K_0})$ and $(-q_{K_0}, -v_{K_0})$ respectively. It is of interest to note that the stress-strain behaviour for a deviator stress less than q_{K_0} , i.e. for negative values of q^* , ε^* and v^* , also appears to be unique and occurs as a continuation of the curve describing the behaviour in the positive range of q .

The method suggested in this section for the correlation of the stresses and strains of specimens subjected to varying one-dimensional consolidation stresses and subsequently sheared from an isotropic stress condition, has an important field application. Specimens which are in the normally consolidated state under a K_0 condition in the field when tested in the triaxial apparatus at an isotropic stress (greater than the initial one-dimensional consolidation stress) will only give unique behaviour in the (q^*, ε^*) plot and not in the conventional (q, ε) or $(q/p, \varepsilon)$ plots. The initial one-dimensional consolidation stress experienced in the field by a specimen could be determined by performing a single one-dimensional consolidation test in the laboratory on the specimen. Then, by doing a single drained triaxial test after isotropic consolidation at a cell pressure higher than the initial one-dimensional consolidation stress, it would be possible to established the (q^*, ε^*) and (q^*, v^*) behaviour of the field specimen.

Effect of Initial Consolidation Stress on Undrained Test Results

The observations during undrained tests on two specimens designated as T_7 and T_4 will now be presented, and an attempt will then be made to correlate the data. Specimen T_7 was prepared from a slurry by initial one-dimensional consolidation under a vertical stress of 3 lb/in² which corresponds to

an applied deviatoric stress of 1 lb/in². This specimen was then isotropically consolidated under a pressure of 60 lb/in² and was subsequently sheared in a conventional triaxial apparatus under undrained conditions. Sample T₄ was subjected to a similar treatment but was subjected to a one-dimensional consolidation stress of 22 lb/in², corresponding to an initial shear stress of 6.6 lb/in².

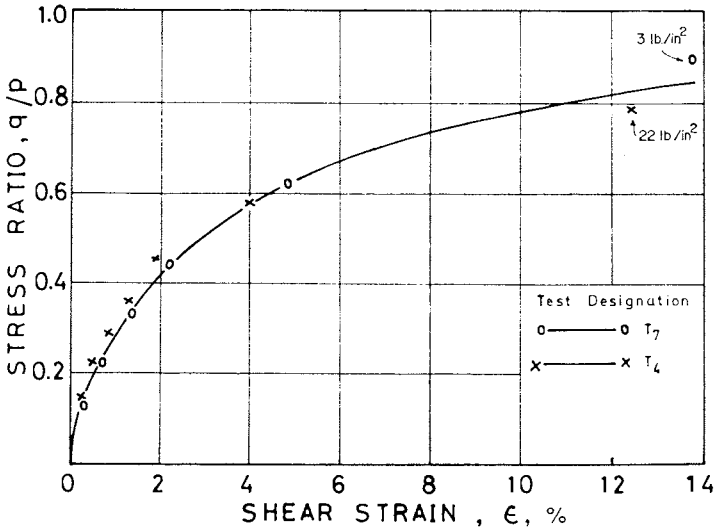


Fig. 9. The $(q/p, \epsilon)$ characteristics of specimens prepared under different initial one-dimensional consolidation stresses and sheared under undrained condition from an isotropic stress of 60 lb/in² with constant cell pressure.

Experimental data from undrained tests—In Fig. 9 the $(q/p, \epsilon)$ relationships observed during triaxial shear tests T₇ and T₄ are plotted and these can be seen to be virtually identical. The magnitudes of the parameters ϵ and q were both taken to be zero at the commencement of the triaxial shear phase of the tests (i.e. after the isotropic consolidation to 60 lb/in²). This unique $(q/p, \epsilon)$ relationship is independent, therefore, of the previous shear stress (or strain) history which had been imposed on the samples during the initial one-dimensional consolidation. The fact that no such uniqueness was observed in drained tests will be discussed later.

Two methods which attempt to correlate the results of undrained tests T₇ and T₄ will now be considered. The first is the same as the (q^*, v^*, ϵ^*) method used in correlating drained test results.

Correlation of undrained test results : Method A — If, during the initial one-dimensional consolidation, a sample is subjected to a shear stress q_{K_0} and undergoes a shear strain ϵ_{K_0} , then, during subsequent shear in the triaxial shear phase of a test, $q^* = q - q_{K_0}$ and $\epsilon^* = \epsilon - \epsilon_{K_0}$. The (q^*, ϵ^*) relations for specimens T₇ and T₄ can be seen to be completely different in Fig. 10. The

STRESS-STRAIN OF CLAY

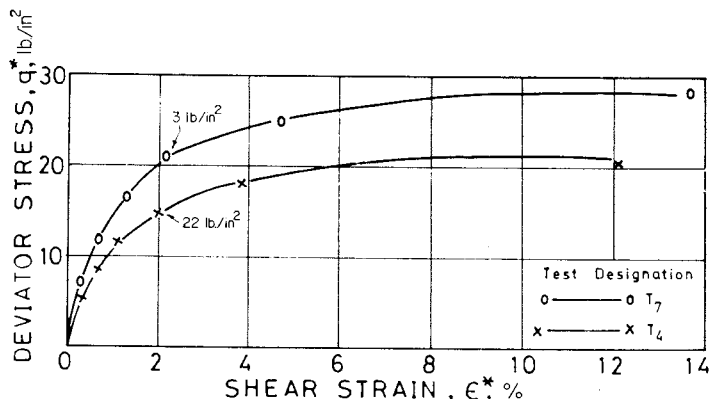


Fig. 10. The (q^*, ϵ^*) characteristics of specimens prepared under different initial one-dimensional consolidation stresses and sheared under undrained condition from an isotropic stress of 60 lb/in² with constant cell pressure.

$(q^*/p^*, \epsilon^*)$ relations were also found to be different. In deriving p^* it was assumed that :

$$p^* = p_o + \frac{1}{3} q^* - u \dots \dots \dots (4)$$

A possible explanation for the fact that both the (q^*, ϵ^*) and the $(q^*/p^*, \epsilon^*)$ relationships are not unique for undrained tests on samples with different shear stress (or strain) histories, whereas they were for drained tests, will be discussed later.

Correlation of undrained test results : Method B — In this method, it is assumed that (i) a unique (q, u) relationship exists during triaxial shear of any specimen that has been prepared from a slurry under isotropic conditions only (i.e. without any initial one-dimensional consolidation), and (ii) the changes in q and u actually observed during triaxial shear of a sample, which has in its history previously experienced a shear stress of q , would only be the same as those for an isotropic sample for values of $q > q_1$. In Fig. 11, the values of q_1 for samples T₇ and T₄ correspond to points A and B respectively. If the two assumptions stated above are correct, then curve BY when B is displaced to C (where C is at the same value of q as B) should coincide with curve CX. This can be seen to be the case in Fig. 12. Hence, the (q, u) relations for the two specimens are identical for all values of q larger than the highest value of q_1 imposed on either specimen during the preliminary one-dimensional consolidation. If, however, the values of u shown in Fig. 12 are used to calculate the revised values of p during the shear tests on specimens T₇ and T₄, it is found that the $(q/p, \epsilon)$ relationship is not unique at higher values of q/p , as shown in Fig. 13. It will be recalled in Fig. 9 that, when the experimentally observed values of u were used to calculate p , the $(q/p, \epsilon)$ curve was unique for all values of q/p . In the light of the evidence provided so far, it is concluded that the

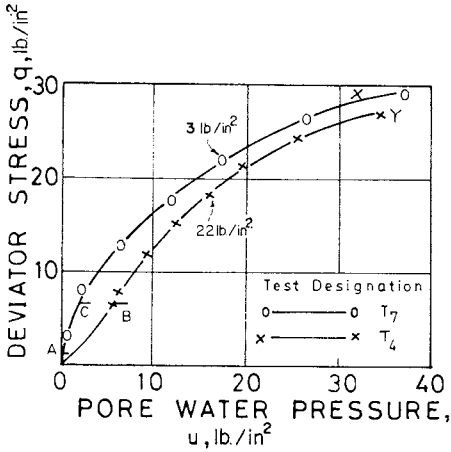


Fig. 11. The (q, u) characteristics for Specimens T7 and T4.

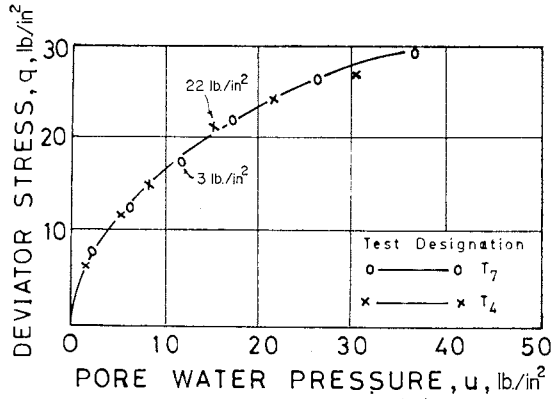


Fig. 12. Modified (q, u) relations for Specimens T7 and T4.

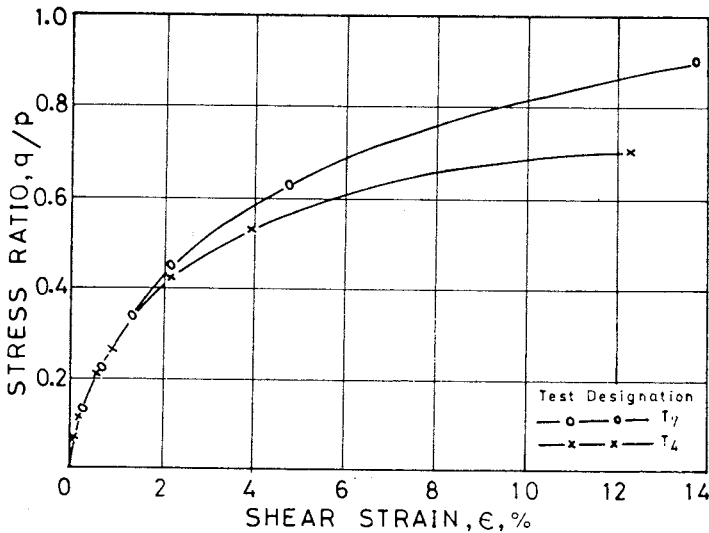


Fig. 13. Modified $(q/p, \epsilon)$ relationships for Specimens T7 and T4.

$(q/p, \epsilon)$ relation actually observed in undrained tests is unique and is not affected by the previous shear stress, q_1 , imposed during one-dimensional consolidation. Furthermore, the pore pressures observed during triaxial shear are dependent on q_1 , but deviations from the values observed with isotropically prepared samples occur in the range $0 < q < q_1$.

It will be recalled that the $(q/p, \epsilon)$ relationship observed in drained tests was affected by the initial shear stress. The apparent contradiction between this and the conclusion made above for undrained tests may be explained by appealing to the hypothesis of ROSCOE & POOROOSHASB (1963) or its modification by ROSCOE & BURLAND (1968). The basic equation used by Roscoe &

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Poorooshasb has been discussed in detail by BALASUBRAMANIAM (1969). This equation can be expressed in terms of shear strain, ϵ , as :

$$d\epsilon = \left(\frac{d\epsilon}{d\eta}\right)_v d\eta + \left(\frac{d\epsilon}{dv}\right)_\eta dv \dots \dots \dots (5)$$

where $(d\epsilon/d\eta)_v$ corresponds to the slope of the (ϵ, η) characteristic in an undrained test, and $(dv/d\epsilon)_\eta$ refers to the slope of the anisotropic consolidation path in a (v, ϵ) space. Consequently, an increment of shear strain during any test in which there is a reduction of volume can be considered to be made up of two components, namely :

- (i) an undrained component as represented by the first term on the right-hand side of Eq. 5,
- and (ii) an anisotropic consolidation component as represented by the second term in Eq. 5.

Hence, it is possible for the undrained component of the shear strain to be independent of the initial shear stress history while the anisotropic consolidation component *does* depend on the initial shear stress history for the range $0 < q < q_1$. It can be seen that the anisotropic consolidation component in Eq. 5 is itself the product of two components, namely:

- (i) a component $(d\epsilon/dv)_\eta$ which is obtained from the results of anisotropic consolidation tests,
- and (ii) the volume change, dv , which is predicted by projecting the imposed stress increment onto the state boundary surface (see ROSCOE & POOROOSHASB, 1963).

This latter surface is assumed to be unique and to be best represented by data from undrained tests. The author believes, from his own data and from that of POOROOSHASB (1961) and THURAIRAJAH (1961), that $(d\epsilon/dv)_\eta$ is not dependent on the magnitude of the shear stress, q . This implies that the volumetric strains experienced during triaxial shear tests with geometrically similar stress paths are not uniquely related to q/p until $q > q_1$.

EFFECT OF LOAD INCREMENT SIZE ON THE STRESS-STRAIN BEHAVIOUR

In studying the effect of stress paths on the stress-strain behaviour it is essential to keep the magnitude of pore pressure associated with a stress increment as low as possible. Fully drained compression tests at constant cell pressures with large increments can cause high pore pressures to develop. Hence, the specimen will be virtually subjected initially to an undrained stress path and, subsequently, to a q -constant stress path. Though this type of stress path is more likely to occur in engineering practice, it would give a totally

misleading picture in the laboratory if it was there assumed that this path referred to the stress-strain behaviour under fully drained conditions.

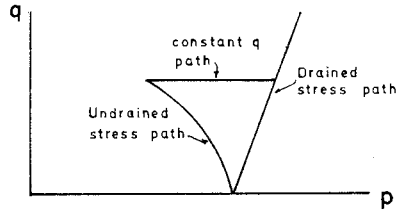


Fig. 14. Stress paths followed by a specimen subjected to a large load increment.

The stress path actually followed by a 'drained' test specimen subjected to a large increment is indicated in Fig. 14, where it is evident that the specimen has been subjected to a stress path which is totally different from the one intended. Also, the stress ratio at the end of the undrained phase is higher than the stress ratio at the end of the q -constant phase. Hence, the specimen will have been subjected to a higher stress ratio than the recorded value at the end of its pore pressure dissipation. In this section, the effects of the load increment size on the stress-strain behaviour of the specimens (sheared under fully drained conditions from a cell pressure of 60 lb/in²) will be investigated. The stress history of each specimen was identical prior to shear, namely a one-dimensional stress of 22 lb/in² and an isotropic stress of 60 lb/in².

Experimental Observations Illustrating the Effects of Load Increment Size

Experimental observations on four fully drained tests (Tests No. Z, R, X and Y) are presented in this section. In Test Z, stress increments of 0.7 lb/in² were applied at 3 hour intervals. Five stress increments were applied during a day, and the equilibrium readings were taken before applying the next increment in the following morning. In Test R, stress increments of 7 lb/in² were applied at two days intervals. Stress increments of 14 lb/in² were applied in Test X at two day intervals, and in Test Y a single stress increment of 26 lb/in² was used.

The (q, ϵ) characteristics of the specimens are as shown in Fig. 15. It is seen that the shear strain corresponding to any particular deviator stress is different for each of the tests, indicating that the magnitude of the applied stress increment has an important effect on the results obtained in drained tests. The results indicate that ϵ is path dependent. The (q, ϵ) characteristics of the specimen in Test Z before and after application of the load increments is illustrated in Fig. 16. It is evident that the difference in strains caused by each stress increment is so small that the deviation of shear strains before

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and immediately after the application of the stress increment are within the limits of the experimental accuracy.

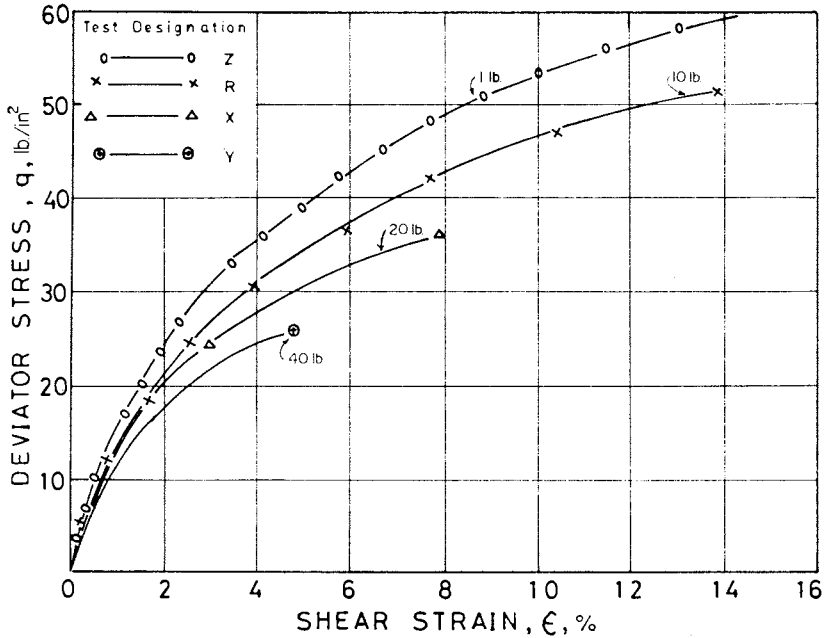


Fig. 15. The (q, ϵ) relationships of specimens sheared under fully drained condition from an isotropic stress of 60 lb/in^2 with different load increment sizes.

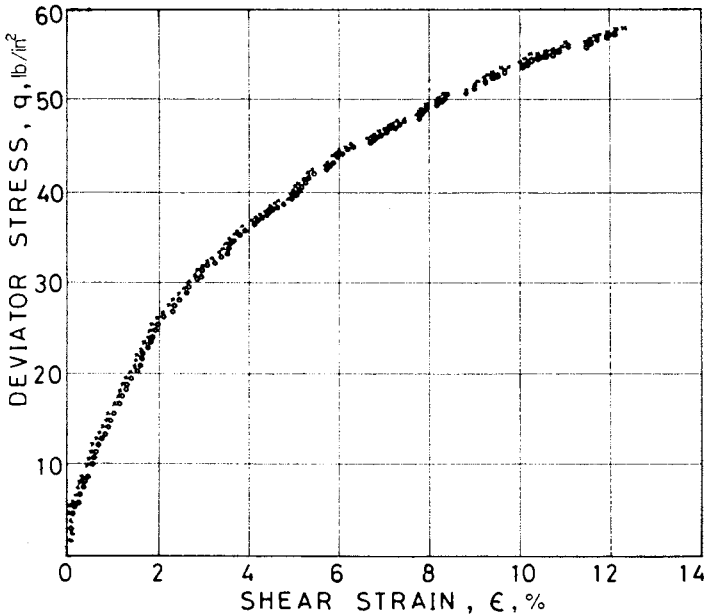


Fig. 16. The (q, ϵ) relationship for Specimen Z sheared under small load increments.

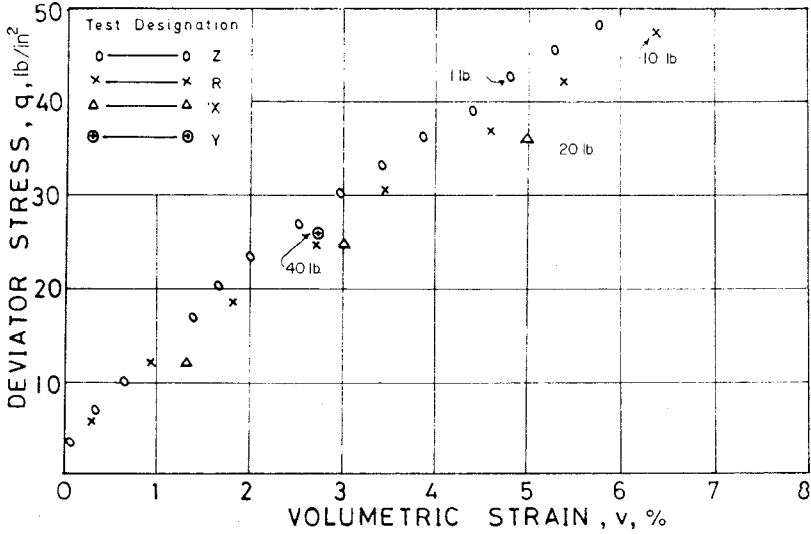


Fig. 17. The (q, v) relationships of specimens sheared under fully drained conditions from an isotropic stress of 60 lb/in² with different load increment sizes.

Figure 17 illustrates the (q, v) relationships for all the specimens. The (q, v) relationships of the specimens are seen to be different. There is no orderly behaviour in the (q, v) relationships of these samples. One would expect this discrepancy since it has already been established that the pore pressures

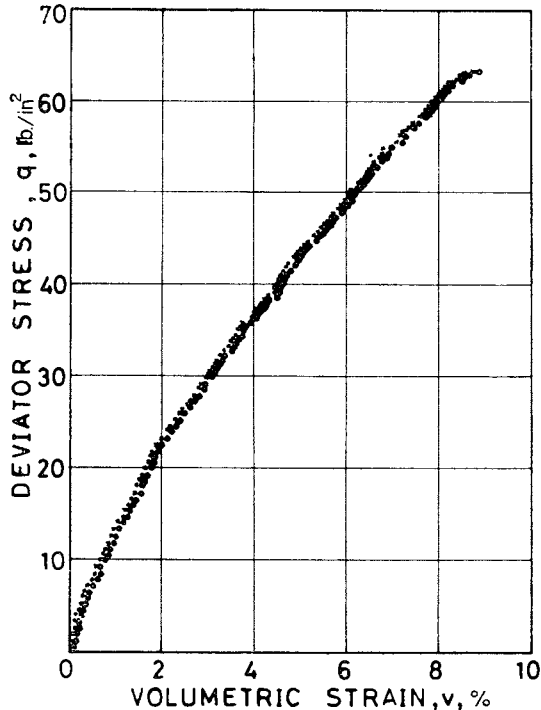


Fig. 18. The (q, v) relationship for Specimen Z sheared under small load increments.

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developed in undrained tests and the volumetric strains experienced in drained tests are affected by the initial shear stress effects caused by the preliminary one-dimensional consolidation. It is suggested that, if the effect of the initial shear stress history had not existed, then there would have been a unique (q, v) relationship independent of the load increment size.

Figure 18 illustrates the (q, v) relationship for Specimen Z; it should be noted that the stress increment is so reduced that the stress-strain behaviour before and after the application of the load lies within a narrow band.

STRESS-STRAIN CURVES OF SPECIMENS SHEARED AT DIFFERENT LEVELS OF ISOTROPIC STRESS WITH SIMILAR APPLIED STRESS PATHS

In this section, the author will discuss the stress-strain behaviour of specimens sheared from isotropic stress levels of 30, 60 and 90 lb/in² when subjected to three different types of imposed stress path. The imposed stress paths were those of (i) an undrained test, (ii) a p -constant test, and (iii) a fully drained test (applied stress path of slope $dq/dp = 3$). All specimens were initially prepared under a one-dimensional consolidation stress of 22 lb/in² and were subsequently isotropically consolidated to the relevant level of isotropic stress. The experimental observations on each type of test are first presented and subsequently discussed.

Experimental Observations on Undrained Tests

Figure 19 illustrates the variation of shear strain, ϵ , with stress ratio, q/p

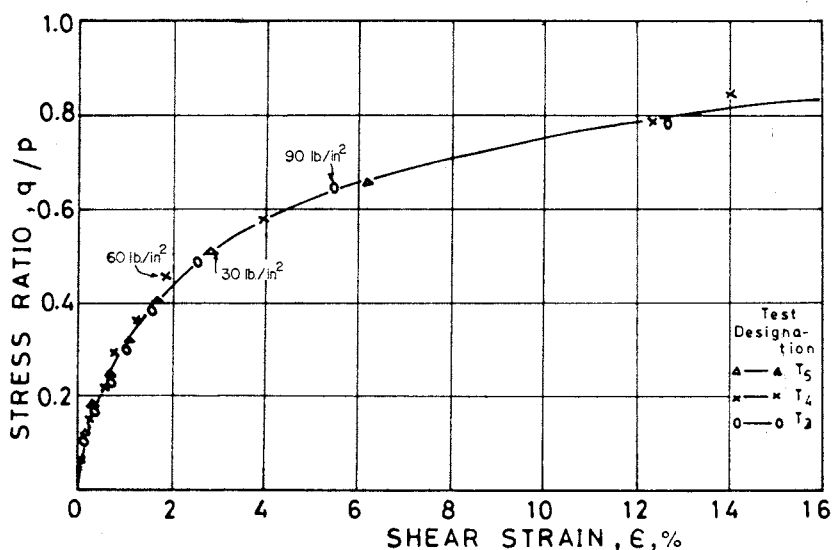


Fig. 19. The $(q/p, \epsilon)$ characteristics of Specimens T₃, T₄ and T₅ prepared under an initial one-dimensional stress of 22 lb/in² and sheared under undrained conditions from isotropic stresses of 30, 60 and 90 lb/in² respectively.

for three undrained compression tests. It is seen that the variation of q/p with ϵ is unique for all three tests.

Experimental Observations on p-Constant Tests

Figure 20 illustrates the variation of shear strain, ϵ , with stress ratio q/p for three specimens AJ, AQ and AO sheared from isotropic stresses of 30, 60 and 90 lb/in²

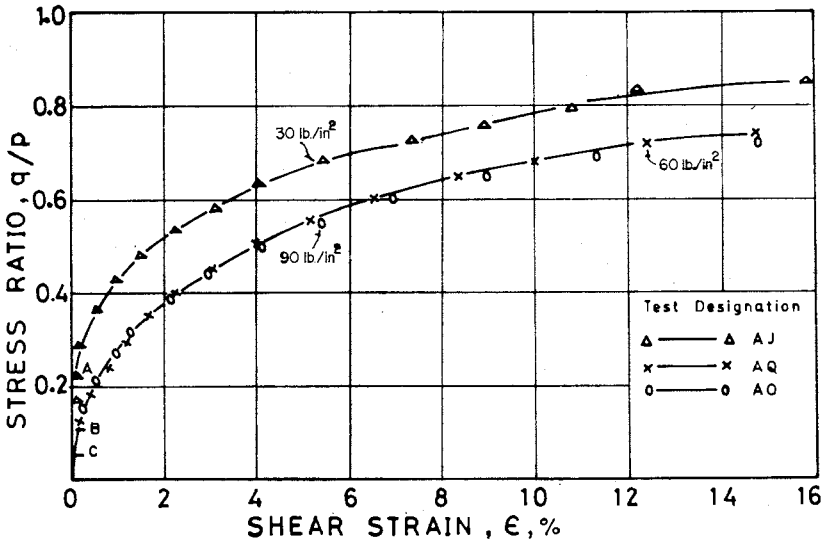


Fig. 20. The $(q/p, \epsilon)$ characteristics of Specimens AJ, AQ and AO prepared under an initial one-dimensional stress of 22 lb/in² and sheared under p-constant conditions from isotropic stresses of 30, 60 and 90 lb/in² respectively.

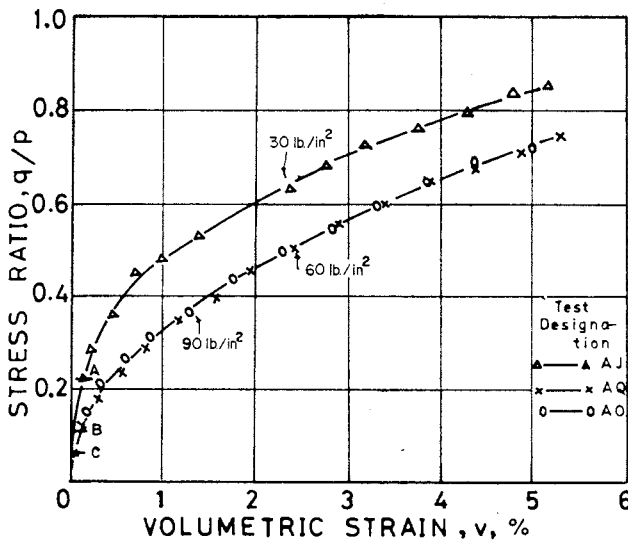


Fig. 21. The $(q/p, v)$ characteristics of Specimens AJ, AQ and AO prepared under an initial one-dimensional consolidation stress of 22 lb/in² and sheared under p-constant condition from isotropic stresses of 30, 60 and 90 lb/in² respectively.

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and 90 lb/in² respectively. It is seen that the $(q/p, \varepsilon)$ characteristics for the 30 lb/in² specimen (Test AJ) are quite different from those of the other two. Similar differences are observed in the $(q/p, v)$ relationships of the three specimens in Fig. 21. However, the (v, ε) characteristics for the three specimens AJ, AQ and AO were found to be approximately unique. During a p -constant test, the volumetric strain is assumed to be only a function of the deviator stress. If elastic volumetric strain were assumed to be dependent on the mean normal stress, p , and if elastic shear strain were neglected, then the (v, ε) relationship would refer to plastic strains. Consequently, the slope of this curve would represent that of the plastic strain rate vector which has been extensively used in some of the Cambridge stress-strain theories (see ROSCOE et al, 1963; ROSCOE & BURLAND, 1968; SCHOFIELD & WROTH, 1968).

Correlation of p -constant test data— For the correlation of the experimental observations of p -constant tests, two assumptions will be made namely :

- (i) unique relationships exist between (a) q^* and ε^* and between q^* and v^* , where q^* , ε^* and v^* are as defined before,
- (ii) these unique relationships are similar for all specimens sheared from isotropic stress states; this entails the use of the parameter q^*/p^* for the comparison of strains in specimens sheared after consolidation to different isotropic stress levels.

The results are presented in the $(q^*/p^*, \varepsilon^*)$ and $(q^*/p^*, v^*)$ plots in Figs. 22 and 23 respectively. The unique relationships in both these plots indicate that the assumptions (i) and (ii) made above are valid for the correlation of the test

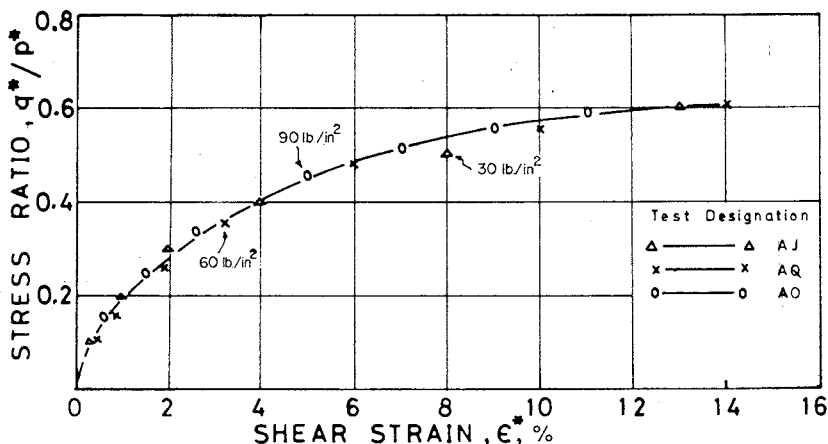


Fig. 22. The $(q^*/p^*, \varepsilon^*)$ characteristics of Specimens AJ, AQ and AO prepared under an initial one-dimensional consolidation stress of 22 lb/in² and sheared under p -constant conditions from isotropic stresses of 30, 60 and 90 lb/in² respectively.

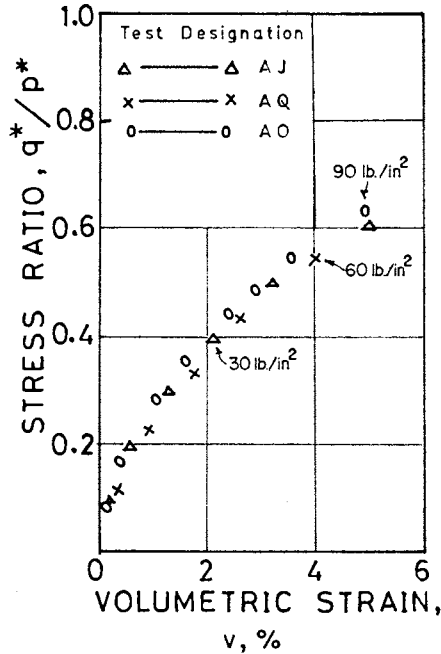


Fig. 23. The $(q^*/p^*, v^*)$ characteristics of Specimens AJ, AQ and AO prepared under an initial one-dimensional consolidation stress of 22 lb/in² and sheared under p-constant condition from isotropic stresses of 30, 60 and 90 lb/in² respectively.

results. Furthermore, since q^* and ϵ^* , and q^* and v^* , are uniquely related, the relationship between v^* and ϵ^* is also unique.

Experimental Observations on Fully Drained Tests

Figure 24 illustrates the variation of ϵ with q/p for the fully drained tests

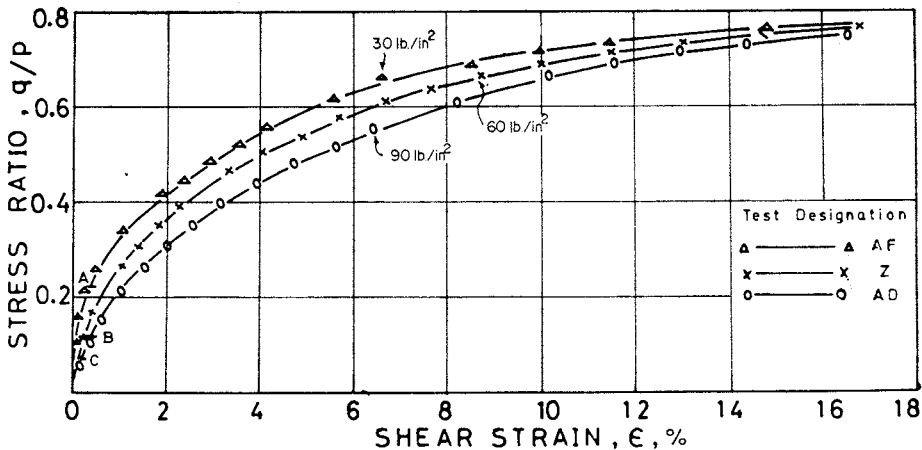


Fig. 24. The $(q/p, \epsilon)$ characteristics of Specimens AF, Z and AD prepared under an initial one-dimensional stress of 22 lb/in² and sheared under fully drained conditions from isotropic stresses of 30, 60 and 90 lb/in² respectively.

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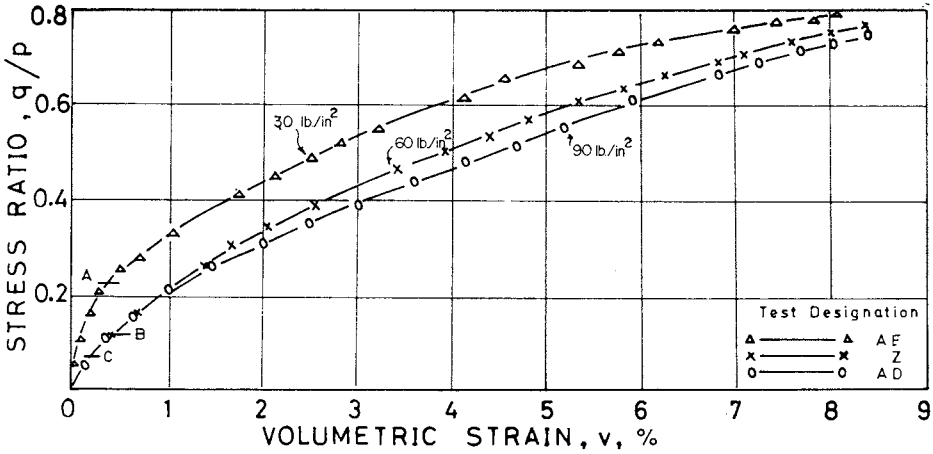


Fig. 25. The $(q/p, v)$ characteristics of Specimens AF, Z and AD prepared under an initial one-dimensional stress of 22 lb/in² and sheared under fully drained conditions from isotropic stresses of 30, 60 and 90 lb/in² respectively.

carried out on specimens AF, Z and AD sheared from isotropic stresses of 30, 60 and 90 lb/in². These characteristics are all different. A similar behaviour is also noted in the $(q/p, v)$ characteristics in Fig. 25. In both figures the specimen sheared from the 30 lb/in² isotropic stress had smaller strains than the other two specimens.

The assumptions made for the correlation of the stress-strain behaviour of the fully drained tests are the same as those mentioned for the correlation of p -constant test results. These results are presented in the $(q^*/p^*, \epsilon^*)$ plot and $(q^*/p^*, v^*)$ plot in Figs. 26 and 27. The unique relationships in these plots indicate that the (v^*, ϵ^*) characteristic must be unique and, furthermore, that the assumptions made are valid for the correlation of the fully drained test results.

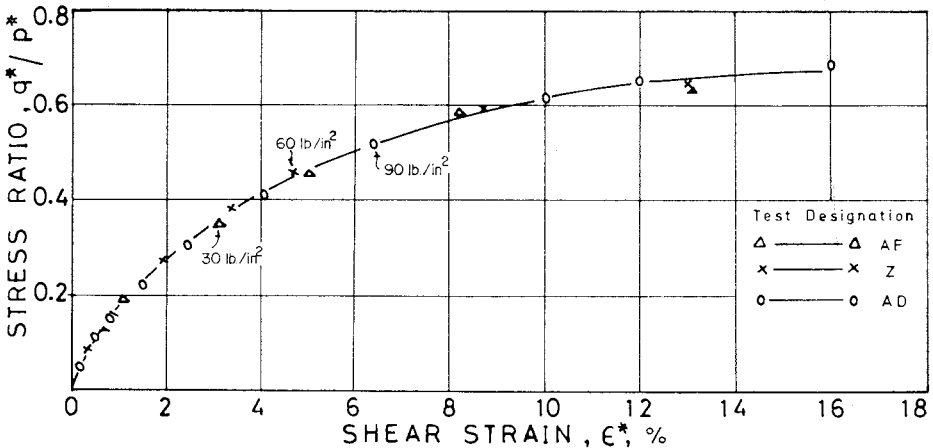


Fig. 26. The $(q^*/p^*, \epsilon^*)$ characteristics of Specimens AF, Z and AD.

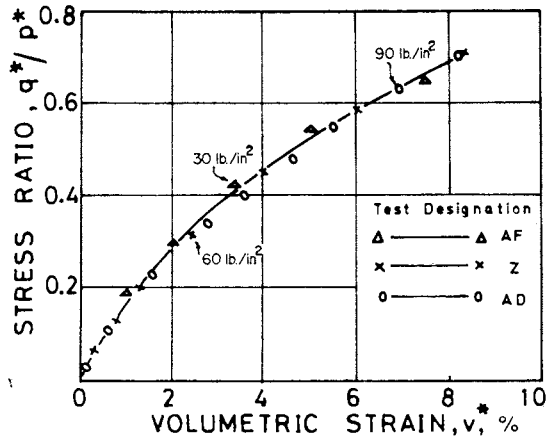


Fig. 27. The $(q^*/p^*, v^*)$ characteristics of Specimens AF, Z and AD.

CONCLUSIONS

Fully drained tests on specimens prepared with different initial one-dimensional stresses and subsequently consolidated under the same isotropic stress showed that the $(q/p, v)$ and the $(q/p, \epsilon)$ relationships were dependent on the magnitude of the initial one-dimensional consolidation stress. The stress-strain behaviour of all these samples was found to be unique when presented in terms of the alternative parameters q^*, v^* and ϵ^* . A set of undrained tests performed on samples prepared in a similar manner showed that the $(q/p, \epsilon)$ relationship was unique at all stress levels.

The stress-strain behaviour of drained test specimens ($dq/dp = 3$) was found to be dependent on the load increment size, and this effect was shown to be entirely due to the difference in stress paths followed by specimens which had been subjected to different load increment sizes. The load increment size was then reduced to such an extent that its effect on the (q, ϵ) and the (q, v) relationships during any application of load was within the limits of the experimental accuracy.

The stress-strain behaviour for three types of test (undrained, p -constant and fully drained with $dq/dp = 3$) on specimens prepared with the same initial one-dimensional stress condition and subsequently sheared from three isotropic stress levels were studied in detail. The observed $(q/p, \epsilon)$ relationship for undrained tests was found to be independent of the previous isotropic consolidation stress level. The $(q/p, v)$ and $(q/p, \epsilon)$ relationships for p -constant tests and the fully drained tests were also found to be dependent on the magnitude of the isotropic consolidation subsequent to the initial one-dimensional consolidation stress prior to shear. Based on the initial shear stress due

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to one-dimensional consolidation, the results were successfully correlated using the parameters q^* , v^* and ϵ^* to give unique behaviour.

ACKNOWLEDGEMENTS

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ON THE DETERMINATION OF THE HORIZONTAL SUBGRADE REACTION

ANTONY B. SIMON* and HANS FRISCH⁺

INTRODUCTION

The rapid development of boring techniques has resulted in a rapid increase in the dimensions of cast-in-place piles. The available large diameters permit the use of vertical instead of batter piles for the support of horizontal loads. The stress analysis of these piles can be carried out on the basis of beam-on-elastic-foundation theory which involves the subgrade reaction concept. The increasing importance of this concept makes it appropriate to summarize and evaluate the methods available for the determination of this coefficient, and to suggest a quick and reliable procedure for the selection of the horizontal subgrade reaction.

THEORETICAL CONSIDERATIONS

The load transmitted onto the soil by a footing will be distributed under the base of the footing as a contact pressure, which is also called *subgrade reaction*. The distribution of this pressure is not uniform over the bearing areas, its distribution depending on many factors. As a crude simplification, it was suggested by WINKLER (1867) that the ratio between the unit contact pressure and the corresponding settlement be regarded as the same for every point on the bearing area. This ratio is known as the "modulus (or coefficient) of subgrade reaction", C . As TERZAGHI (1942) and TERZAGHI & PECK (1968) pointed out repeatedly, this is a highly artificial and incorrect concept, and computations based on this concept result in rather crude estimates only. The modulus of subgrade reaction is defined as the ratio between a vertical load per unit area on a horizontal surface and the corresponding settlement of this surface; or, in other words, the surface pressure required to cause a unit settlement. Since the theory of subgrade reaction has been used since about 1920 for computing the stresses in piles subjected to horizontal forces, the ratio between a horizontal pressure per unit area on a vertical surface and the corresponding horizontal displacement is called the "modulus of horizontal subgrade reaction".

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In general, the modulus of subgrade reaction can be expressed as :

$$\frac{\sigma}{\Delta} = C \quad \dots \dots \dots (1)$$

where σ is the contact pressure and Δ is the displacement. The contact pressure, σ , is a function of the rigidity, shape and size of the footing as well as the soil type. The displacement, Δ , depends on the size and shape of the footing, the soil type and the load. While the displacement can be calculated fairly accurately, the contact load is simply assumed to be uniformly distributed on the contact area. Hence, this equation disregards the absence of strict proportionality between load and settlement as well as the influence of soil type, and size and shape of the loaded area on the contact pressure. There exists, nevertheless, the erroneous but widespread conception among engineers that the numerical value of the modulus of subgrade reaction depends entirely on the nature of the soil.

The modulus of subgrade reaction is usually measured by loading a rigid plate and determining the load-settlement characteristics. For the sake of uniformity, it is customary to determine C by a plate bearing test with a 30 cm diameter plate.

The settlement, Δ , of a rigid plate of radius r on an elastic half-space can be expressed by the equation:

$$\Delta = \frac{\pi p 2r}{4E} (1 - \mu^2) \quad \dots \dots \dots (2)$$

where p is the average applied pressure, and E and μ are the modulus of elasticity and Poisson's ratio of the elastic medium. For an incompressible material ($\mu = 0.5$) this equation can be simplified to :

$$\Delta = 1.18 \frac{pr}{E} \quad \dots \dots \dots (3)$$

and the modulus of subgrade reaction, $C (= p)$, is therefore:

$$C = \frac{E}{1.18 r} \quad \dots \dots \dots (4)$$

Hence, it is evident that C is a function of the size of plate and the compressibility of the elastic medium.

The influence of the diameter was investigated by Stratton (KEZDI, 1970) who compared theoretical and experimental results and showed that C decreases as the plate diameter increases. However, the condition that C is inversely proportional to $1/r$ is seldom satisfied, particularly in the case of cohesive soils. The influence of the soil type is expressed in Eq. 3 by its

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modulus of elasticity, E , although it is a well-established fact that for soils the laws of elasticity are valid only in a very limited stress range. The modulus of elasticity can be related theoretically (SZECHY, 1963) to the modulus of compressibility, M , by :

$$M = \frac{m^2 E}{m^2 - 1} \dots \dots \dots (5)$$

where m is Poisson's number. This relationship can also be written as :

$$\beta = \frac{E}{M} \dots \dots \dots (6)$$

where the value of β ranges between 0.47 and 0.98 depending on the soil type. The modulus of compressibility can be readily obtained from a consolidation test on an undisturbed sample as:

$$M = \frac{p_2 - p_1}{\epsilon_2 - \epsilon_1} \dots \dots \dots (7)$$

where ϵ_1 and ϵ_2 are the strains caused by pressures of p_1 and p_2 respectively (Fig. 1). Empirical relationships are also used for values of M ; one such

method correlates the modulus of compressibility to the standard penetration test (MENZENBACH, 1959; SCHULTZE & MELZER, 1965).

In the case of isotropic materials, the elasticity and compressibility are equal in all directions. Granular soils can be considered as isotropic materials, but laminated sediments, particularly clays, must be regarded as anisotropic substances and their compressibilities are different in the horizontal and vertical directions. Test results and theoretical considerations

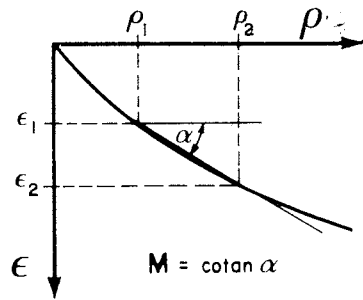


Fig. 1. Calculation of the modulus of compressibility.

(SIMON, 1970) show that the compressibility of laminated sediments is about three to four times higher in the direction perpendicular to the laminations than in the parallel direction. Since a linear relationship exists between compressibility and modulus of subgrade reaction, there must be a difference between the horizontal and vertical moduli of subgrade reaction. In cohesive laminated soils the modulus of horizontal subgrade reaction might be up to four times as high as the vertical value, all other conditions being equal. TERZAGHI (1955) proposed a subgrade reaction for stiff clays independent of depth or direction and, although this is on the safe side, the horizontal subgrade reaction is probably 300 to 400 % in error.

CUSTOMARY DETERMINATION OF MODULUS OF SUBGRADE REACTION

The only completely satisfactory way of measuring C would be by loading the foundation soils with a footing of the same shape and size as that to be used in the structure. For obvious reasons, this is an impractical proposition. Even if such tests could be performed on piles, the results would be valid for single piles only and the substantial group action would be neglected.

For practical purposes the modulus of subgrade reaction can be (a) chosen from compiled tables, (b) extrapolated from field tests, (c) calculated from various formulae, and (d) determined by empirical correlations from other soil test data.

Compiled Tables

Recommended average values of C have been published in many textbooks (e.g. TITZE, 1970; BOWLES, 1969). Selection is made according to the soil type and its consistency or density, but the size and shape of the contact area is disregarded. The values proposed range from 0.5 to 10 kg/cm³ (occasionally to 20 kg/cm³). They are on the conservative side as indicated by field measurements (KERISEL & SIMONS, 1962; BROMS & INGELSON, 1972). Since the modulus of subgrade reaction occurs under the fourth root in most of the formulae applied in the stress analysis, an error in the choice of its value is not very significant. In general, no differentiation is made between the vertical and horizontal subgrade reactions.

Extrapolation from Field Tests

Tests are performed on the soil with the recommended 30 cm diameter bearing plate, and the results are extrapolated by means of the correction factors proposed by Stratton (KEZDI, 1970). The extrapolation should pay attention to the stratigraphy of the soil. An increase in the dimensions of the loading area increases the depth influenced by the stresses and might lead to disproportionate deflections if there were a pronounced change in the compressibility of the soil with depth. This method is generally applicable only for a horizontal surface. However, there are reports of tests made on the vertical faces of trenches (SLACK & WALKER, 1970), which is a convenient procedure to evaluate the effectiveness of backfilling.

Use of Formulae

Several methods have been proposed for the calculation of the modulus of subgrade reaction. They are based on the deflection caused by a unit load.

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The settlement is calculated from the modulus of compressibility according to various assumptions.

According to SZECHY (1963a), Koegler proposed, for rectangular footings:

$$C = M \frac{L + 2z}{Lz} \dots \dots \dots (8)$$

and for strip footings:

$$C = \frac{2M}{B \ln \frac{B + 2z}{B}} \dots \dots \dots (9)$$

where *M* is the modulus of compressibility of the soil, and *L*, *B* and *z* are the length, width and depth of the footing.

TERZAGHI (1955) recommended the use of certain basic moduli determined by bearing tests of 1 ft square plates modified by a factor which depended on the geometry of the footing. He also differentiated between sandy and clayey soils as well as between the vertical and horizontal subgrade reactions. In the case of sand, Terzaghi recommended:

$$C_v = C_o \left(\frac{B + 30}{2B} \right)^2 \text{ kg/cm}^3 \dots \dots \dots (10)$$

for the modulus of vertical subgrade reaction, where *B* is the width of the footing and the factor *C_o* is selected from Table 1. For the horizontal subgrade reaction at depth *z* in cohesionless sand, he gave for a pile of width *B*:

$$C_h = n_h \frac{z}{B} \dots \dots \dots (11)$$

where *n_h* is a constant selected from Table 2.

Table 1. Values of *C_o* for 30 cm square plate on sand

Condition of Sand	Bulk Density, ton/m ³		
	1.3	1.6	1.9
Dry or moist	1.3 kg/cm ³	4	16
Submerged	0.8	2.5	10

Table 2. Value of *n_h* for 30 cm wide pile embedded in sand

Condition of Sand	Relative Density of Sand		
	Loose	Medium	Dense
Dry or moist	0.22 kg/cm ³	0.67	1.8
Submerged	0.13	0.45	1.1

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In the case of clays, Terzaghi suggested that $C = C_h = C_v$ and that the modulus is a function of footing shape. For strip footings:

$$C = C_o \frac{20}{B} \quad \text{kg/cm}^3 \quad \dots \dots \dots (12)$$

where C_o is related to the unconfined compression strength as shown in Table 3.

Table 3. Values of C_o for 30 cm square plates or 30 cm wide strips on stiff clay

Unconfined Strength, kg/cm ²	1-2 (stiff)	2-4.5 (very stiff)	>4.5 (hard)
Basic C_o Value, kg/cm ³	2.5	5	10

Jaky suggested a very simple formula for modulus of subgrade reaction (SZECHY, 1963a) by assuming a triangular distribution of vertical stress to a maximum depth equal to three times the width of the footing. The compression caused by a unit pressure σ is then:

$$\Delta = \frac{\frac{\sigma}{z} \times 3B}{M} \quad \dots \dots \dots (13)$$

and the modulus of subgrade reaction, therefore, is:

$$C = \frac{\sigma}{\Delta} = \frac{2}{3} \frac{M}{B} \quad \dots \dots \dots (14)$$

This simple equation takes account of the two important influences of footing width and soil type, but is independent of footing shape.

RAUSCH (1969) proposed:

$$C = \frac{E}{f \sqrt{F}} \quad \dots \dots \dots (15)$$

where E is the modulus of elasticity of the soil, F is the area of the footing, and f is a constant which is a function of the shape of the footing (Table 4).

Table 4. Shape factors (RAUSCH, 1969)

Width/Length Ratio	1.0	0.25	0.5
Factor f	0.45	0.35	0.42

Correlation with Soil Test Data

YOKOYAMA (1971) reports that Kubo proposed a relationship between the horizontal subgrade reaction and the standard penetration test based on the

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empirical relationship between the blows/ft, N , of the SPT and the unconfined compression strength ($q_u = N/8 \text{ kg/cm}^2$) and on the equation $C = M/B$, both attributed to TERZAGHI (1942). The expression $C = M/B$ is quite similar to the one established by Jaky (Eq. 14). Since the horizontal subgrade reaction is obtained directly from the N -value ($C_h = N/5$) without consideration of either the diameter of the pile or the type of soil, this procedure does not appear to be very satisfactory, especially for large diameter piles.

DISTRIBUTION OF HORIZONTAL SUBGRADE MODULUS

For the stress analysis of piles, several computer programs are available based on assumed distributions of the subgrade modulus. For convenience, simple geometrical distributions are used (e.g. triangular, rectangular, parabolic, trapezoidal). In the case of a triangular or parabolic distribution, the modulus of horizontal subgrade reaction is assumed to be zero at the surface. Since the subgrade modulus is a function of compressibility, a zero subgrade modulus can only occur if the material is fully compressible. In the case of soils, this condition is seldom satisfied and, therefore, a triangular or parabolic distribution of the modulus of horizontal subgrade reaction should be regarded as exceptional. Based on theoretical considerations, TERZAGHI (1955) proposed a trapezoidal distribution in sands and a rectangular one in clayey soils. Japanese specifications generally assume a rectangular distribution. During the drilling of several hundred boreholes, the authors have not encountered any distribution other than rectangular or trapezoidal.

EFFECT OF DYNAMIC LOADING

The methods considered above were developed to calculate the subgrade reaction induced by static loads. Repeated loading causes some deterioration of the soil resistance, effectively reducing the modulus of subgrade reaction. Reductions in C have been deduced from measured deflections of structures. Reduced moduli of 30% and 66% of the original have been reported by DAVISSON (1970) and OKAMOTO (1971) respectively.

A suitable dynamic modulus of subgrade reaction should be selected for each locality with due consideration for the soil type as well as the magnitudes and durations of dynamic loads.

PROPOSED CALCULATION OF MODULUS OF HORIZONTAL SUBGRADE REACTION

The purpose of pile foundations is to transmit loads to deep strata of adequate bearing capacity. Piles are normally used in loose or soft sediments

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in which the compressibility generally decreases with depth and, therefore, the modulus of subgrade reaction must also be a function of depth. The influence of laminations in the strata or the dynamic nature of the loads should also be taken into account in the determination of the modulus of horizontal subgrade reaction.

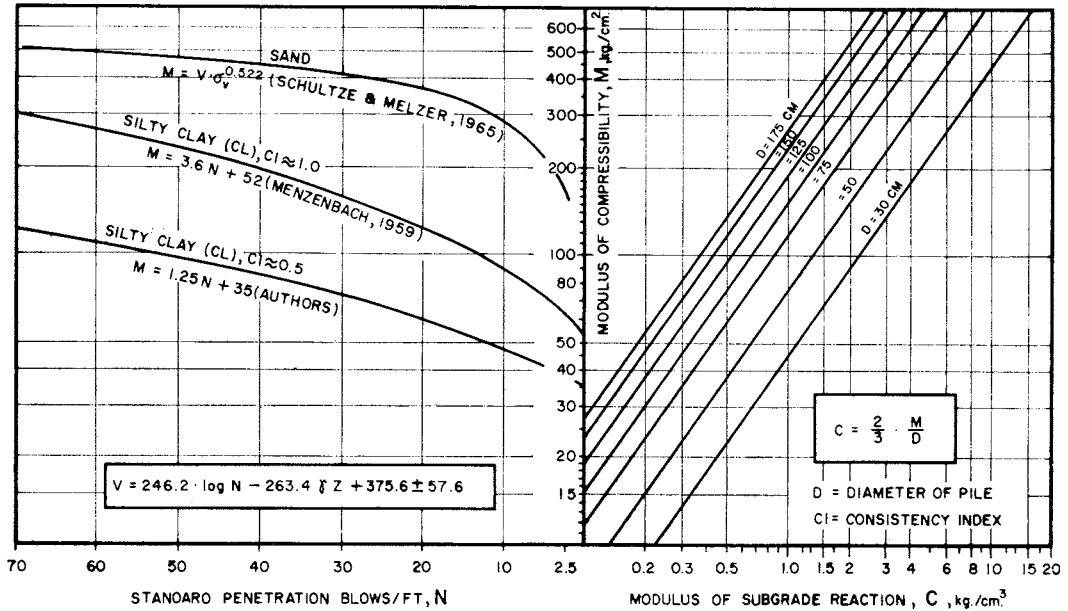


Fig. 2. Proposed determination of the modulus of horizontal subgrade reaction.

The authors suggest the following procedure for the rapid evaluation of the modulus of horizontal subgrade reaction:

- (1) The modulus of compressibility of the soils can be determined empirically from the *N*-values of the standard penetration test (Fig. 2). This method gives reliable results in granular deposits, as established by SCHULTZE & MELZER (1965), although the scatter is greater in cohesive soils with low plasticity (MENZENBACH, 1959). The curve suggested by the authors and given in Fig. 2 is valid for soft cohesive soils only; this was established from the evaluation of test data from several hundred boreholes.

The most accurate values of compressibility can be obtained from consolidation tests on undisturbed samples taking the actual stress conditions into consideration. In order to supplement test data or to check upon compressibilities obtained from *N*-values, the compressibility, *M*, of cohesive soils can be computed from:

$$M = (160 - 2PI) CI \quad \text{kg/cm}^2 \quad \dots \dots (16)$$

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where PI is the plasticity index in percent and CI is the dimensionless consistency index. This relationship was suggested by Kopacsy (KARAFIATH, 1953) for pleistocene sediments with a plasticity index lower than 60%.

- (2) After determination of the modulus of compressibility, the isotropy of the deposit should be established, Granular deposits may be assumed to be isotropic as far as compressibility is concerned, and soft cohesive soils are generally isotropic except in the case of laminated or varved deposits. This anisotropy is mainly encountered in glacial or lacustrine sediments where the ratio between the horizontal and vertical compressibilities varies between about 3 and 5.

The flake-shaped clay minerals or micas of cohesive soils tend to be oriented perpendicularly to the direction of the major principal stress, and this might result in anisotropy. The ratio of the two moduli of compressibility might range from 1 (no orientation) to about 3 (well oriented particles).

- (3) The modulus of compressibility should be multiplied by the selected factor of anisotropy. From the convenient Jaky equation (Eq. 14), the adjusted modulus of compressibility leads to the modulus of horizontal subgrade reaction for the particular pile width. This step in the calculation can be conveniently performed using the right-hand side of Fig. 2.
- (4) The modulus of horizontal subgrade reaction is valid for static loads. Should any dynamic effect due to vibrations or seismic action be present, this value should be reduced to between 30 and 66% of its original value. For this purpose, the soil type and its density or consistency, as well as the magnitude and duration of the dynamic forces, should be taken into account.
- (5) On the basis of the moduli of horizontal subgrade reaction calculated at various depths, the distribution of C_h with depth can be expressed in a convenient form to permit stress analysis by available programs or tables.

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STRESS PATH DEPENDENT PORE PRESSURE PARAMETER A_f

YUDHBIR* and A. VARADARAJAN⁺

INTRODUCTION

The settlement analysis of normally consolidated, saturated deposits involves the computation of pore water pressures by the use of Skempton's pore water pressure parameter A at failure, A_f (SKEMPTON & BJERRUM, 1957; SCOTT, 1963). In general, A_f is for the particular stress path where the vertical principal stress, σ_1 , is increasing in the triaxial (axially symmetric) stress system with the horizontal principal stress, $\sigma_2 (= \sigma_3)$, being kept constant during undrained shear. For stress paths other than this, the parameter A_f has had to be obtained by resorting to graphical construction. Now that powerful computational methods are available, it has become necessary to obtain stress path dependent values of A_f to determine the pore water pressure distribution. With this in mind, relationships are herein derived to predict the stress path dependent A_f . Also, a method is suggested for obtaining the undrained shear strength of a clay sample from its effective stress failure envelope.

PORE WATER PRESSURE PARAMETERS

The pore water pressure change, Δu , for a triaxial stress system can be obtained from the relationship given by SKEMPTON (1954) :

$$\Delta u = B [\Delta\sigma_3 + A (\Delta\sigma_1 - \Delta\sigma_3)] \dots \dots \dots (1)$$

for changes in the total principal stresses of $\Delta\sigma_1$ and $\Delta\sigma_3$, where the pore water pressure parameter B is unity for saturated clays, and the pore water pressure parameter A is dependent on the type of material and the total stress path followed.

HENKEL (1960) gave Eq. 1 in a generalised form, where the pore water pressure developed is a function of (i) compression or expansion, and (ii) shear, as:

$$\Delta u = B(\Delta p + a\Delta\tau_{oct}) \dots \dots \dots (2)$$

where the parameter B is the same as Skempton's, a is the pore water pressure parameter due to shear only, Δp is the change in the octahedral normal stress and $\Delta\tau_{oct}$ is the change in the octahedral shear stress.

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Δp in Eq. 2 is given by :

$$\Delta p = \frac{\sigma_{11} + 2 \sigma_{31}}{3} - \frac{\sigma_{10} + 2 \sigma_{30}}{3}$$

where σ_{10} and σ_{30} are the total principal stresses after consolidation, σ_{11} and σ_{31} are the final total principal stresses, and $\sigma_{20} = \sigma_{30}$ and $\sigma_{21} = \sigma_{31}$ for the axially symmetric case.

This can be rewritten as:

$$\Delta p = \frac{\Delta\sigma_1 + 2 \Delta\sigma_3}{3} \dots \dots \dots (3)$$

where $\Delta\sigma_1 = (\sigma_{11} - \sigma_{10})$ and $\Delta\sigma_3 = (\sigma_{31} - \sigma_{30})$.

$\Delta\tau_{oct}$ in Eq. 2 is given by :

$$\Delta\tau_{oct} = \tau_{oct(1)} - \tau_{oct(0)}$$

where $\tau_{oct(1)} = \sqrt{[(\sigma_{11} - \sigma_{21})^2 + (\sigma_{21} - \sigma_{31})^2 + (\sigma_{31} - \sigma_{11})^2]}$
 $= \sqrt{2} (\sigma_{11} - \sigma_{31})$

and $\tau_{oct(0)} = \sqrt{2} (\sigma_{10} - \sigma_{30})$

Hence :

$$\Delta\tau_{oct} = \sqrt{2} (\Delta\sigma_1 - \Delta\sigma_3) \dots \dots \dots (4)$$

For saturated clays, substitution of Eqs. 3 and 4 into Eq. 2 gives :

$$\Delta u = \frac{\Delta\sigma_1 + 2\Delta\sigma_3}{3} + a \sqrt{2} (\Delta\sigma_1 - \Delta\sigma_3) \dots \dots \dots (5)$$

For the case where $(\Delta\sigma_1 + 2\Delta\sigma_3)/3 = 0$:

$$\Delta u = a\sqrt{2}(\Delta\sigma_1 - \Delta\sigma_3) \text{ and } a = \frac{\Delta u}{\sqrt{2} (\Delta\sigma_1 - \Delta\sigma_3)}$$

Equation 4 is very useful in the computation of pore water pressures.

STRESS PATH AND PORE PRESSURE PARAMETER A_f

Figure 1 shows samples consolidated to a pressure designated by R and sheared to failure following four different total stress paths noted as RB, RC, RD and RE. The effective stress path is also shown in Fig. 1. The total stress paths can be defined by the slopes $\Delta q/\Delta p$, as shown in the figure. For σ_3 constant and σ_1 increasing (path RB), $\Delta q/\Delta p = 3$; for $(\Delta\sigma_1 + 2\Delta\sigma_3)/3 = 0$ (p constant) (path RC), $\Delta q/\Delta p = \infty$; for σ_1 constant and σ_3 decreasing (path RD), $\Delta q/\Delta p = -3/2$; for σ_1 and σ_3 decreasing with $\Delta\sigma_1/\Delta\sigma_3 = 0.4$ (path RE), $\Delta q/\Delta p = -3/4$.

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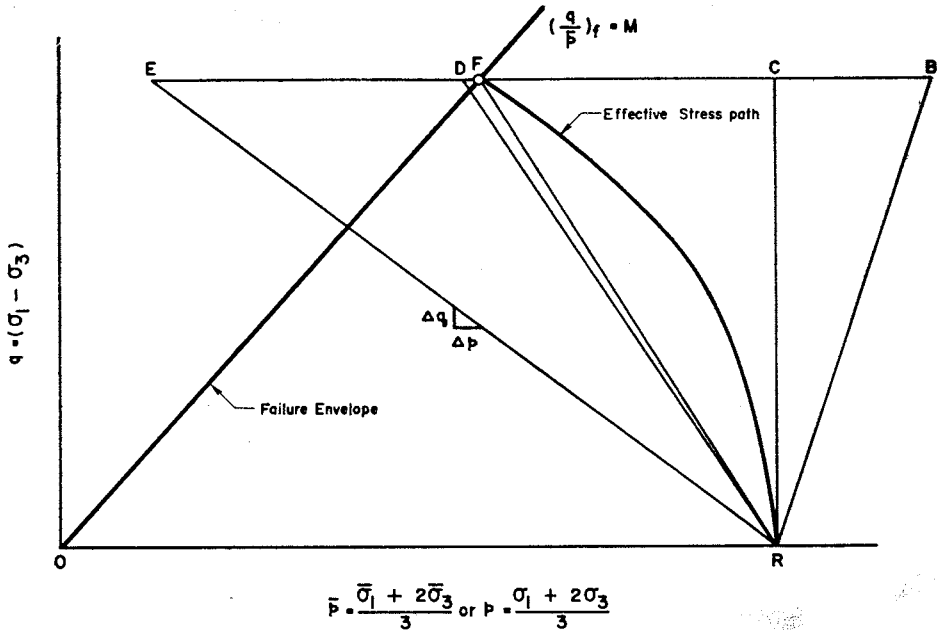


Fig. 1. Effective stress failure envelope.

The pore water pressure for path RC is given by FC in Fig. 1, and $a\sqrt{2} = \Delta u / (\Delta\sigma_1 - \Delta\sigma_3)$. Also, $a\sqrt{2} = A_f$ in this case. For path RB, the pore water pressure is given by :

$$FB = FC + CB = a\sqrt{2} (\Delta\sigma_1 - \Delta\sigma_3) + \frac{\Delta p}{\Delta q} (\Delta\sigma_1 - \Delta\sigma_3)$$

where $\frac{\Delta p}{\Delta q}$ is the inverse slope of the stress path RB. Thus :

$$\begin{aligned} FB &= \left(a\sqrt{2} + \frac{\Delta p}{\Delta q} \right) (\Delta\sigma_1 - \Delta\sigma_3) \\ &= (a\sqrt{2} + 1/3) (\Delta\sigma_1 - \Delta\sigma_3) \end{aligned}$$

In this case, $A_f = (a\sqrt{2} + 1/3)$. Similarly, for paths RD and RE, the A_f values are $(a\sqrt{2} - 2/3)$ and $(a\sqrt{2} - 4/3)$ respectively.

RELATIONSHIP BETWEEN STRESS PATH AND A_f

A graphical plot is shown in Fig. 2 between the inverse slope of the stress paths, $\Delta p / \Delta q$, and the parameter A_f for several different clays. It is significant that the relationships are linear. The lines for the different clays are parallel to each other and have slopes of 45° to the horizontal. The general equation for the straight lines is :

$$A_f = \left(\frac{\Delta p}{\Delta q} + a\sqrt{2} \right) \dots \dots \dots (6)$$

Thus, if Henkel's pore pressure parameter a is known for a clay, then Skempton's pore pressure parameter A_f can be found for any stress path. It may be noted that Henkel's pore pressure parameter a , which is independent of stress path, is more fundamental for a clay. It indicates the pore water pressure development at failure due to shear, and is more useful in the prediction of pore pressure response of saturated clays at failure.

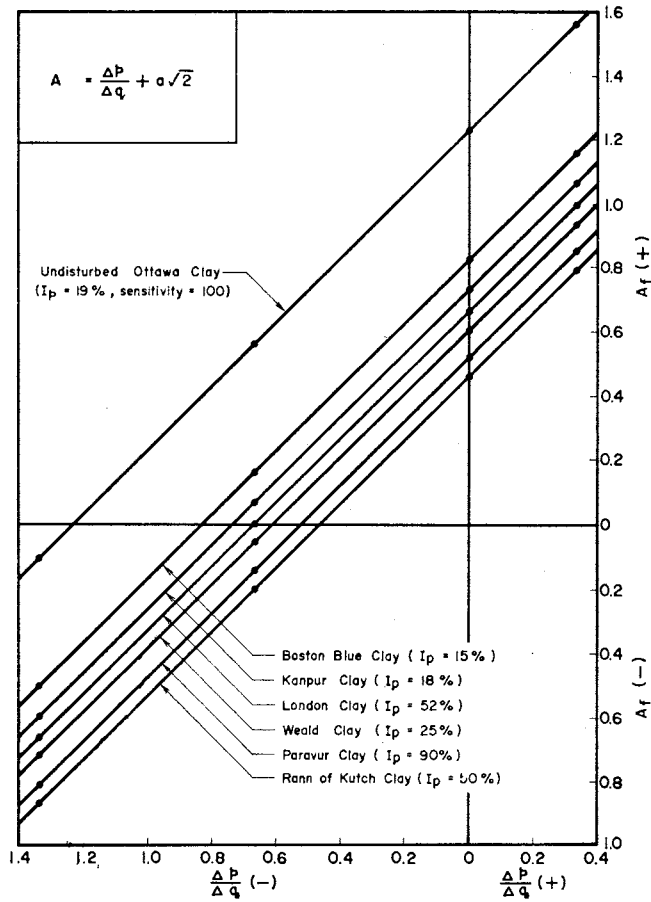


Fig. 2. Relationship between stress path and parameter A_f .

PREDICTION OF UNDRAINED STRENGTH FROM EFFECTIVE STRESS ENVELOPE

From Fig. 2, it can be seen that $A_f = 0$ for a different total stress path for each clay. For the particular total stress path for which $A_f = 0$, the failure points on the total and effective stress paths are the same since the pore pressure is zero (e.g. at point F on path RF in Fig. 1). If the effective stress failure envelope has a relationship $(q/\bar{p})_f = M$, a constant, then the undrained

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strength of a sample consolidated to a pressure \bar{p}_o (say, R in Fig. 1) is given by :

$$\frac{s_u}{\bar{p}_o} = \frac{(\sigma_1 - \sigma_3)_f}{\bar{p}_o} = \frac{\left(\frac{\Delta q}{\Delta p}\right)_{A_o} M}{\left(\frac{\Delta q}{\Delta p}\right)_{A_o} - M} \dots \dots \dots (7)$$

where $(\Delta q/\Delta p)_{A_o}$ is the slope of the total stress path for which $A_f = 0$, and it can be obtained from Eq. 2.

CONCLUSIONS

A linear relationship has been obtained between Skempton's pore pressure parameter A_f and the total stress path. The inverse slope of the total stress path in this relationship is shown to be constant at 45° to the horizontal for all clays. For the common triaxial test conditions ($\sigma_2 = \sigma_3$), $A_f = (\Delta p/\Delta q + a\sqrt{2})$.

The undrained strength of samples under any given consolidation pressure can be readily determined from the suggested Eq. 7 if the total and effective stress paths are known.

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ON RESIDUAL SHEAR STRENGTH OF SATURATED REMOULDED CLAYS

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INTRODUCTION

Renewed interest in the postpeak behaviour of cohesive soils, stimulated by the Fourth Rankine Lecture (SKEMPTON, 1964), has led to a number of investigations in the recent past. A close examination of the various investigations concerned with residual strength of clays indicates the possibility of grouping them into the following categories:

- (1) *Development of a suitable laboratory technique to measure residual strength*—(a) reversing direct shear (BISHOP & LITTLE, 1967; SKEMPTON & PETLEY, 1967; MARSLAND & BUTLER, 1967; CULLEN & DONALD, 1971; JAMES, 1971), (b) triaxial compression (BISHOP et al, 1965; CHANDLER, 1966; WEBB, 1969), (c) ring shear (BISHOP et al, 1971).
- (2) *Influence of geomechanical features on residual strength*—(SKEMPTON & DE LORY, 1967; GOULD, 1960; SKEMPTON, 1964; RINGHEIM, 1964; BISHOP et al, 1965; BJERRUM, 1966; HUTCHINSON, 1967; JAMES, 1971).
- (3) *Laboratory studies directed to an understanding of the basic mechanisms involved in the mobilization of shear strength during the residual stage of deformation* (SKEMPTON, 1964; MORGENSTERN & TCHALENKO, 1967; KENNEY, 1967; SMART, 1970; FOSTER & DE, 1971).

The present investigation pertains to the last category, and is primarily concerned with the study of the influence of the following factors on the residual strength of saturated remoulded clays:

- (1) clay composition,
- (2) mode of pre-consolidation stress history (isotropic and K_0 -consolidation),
- (3) orientation of sample (vertical and horizontal to major principal stress directors),
- (4) stress history (normally consolidated and overconsolidated),
- (5) test conditions (undrained and drained).

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Saturated clays in the remoulded state were used to eliminate the effects of macro-structural geomechanical features. On the basis of earlier investigations, a micro-model for the failure plane at the residual shearing resistance is suggested.

GENERAL CONSIDERATIONS

In a broader sense, the shearing strength of a cohesive soil is a function of several factors, such as void ratio (or average particle spacing), stress history and strain rate. Shearing tends to slide and reorient the particles parallel to one another to result in the minimum potential energy condition. Before reaching this stage, during the initial stages of shearing, the peak shear strength mobilized by the system is partly influenced by interference effects. With the inducement of large shearing strains, the oriented matrix reaches a stable configuration.

SKEMPTON (1964) indicated that a decrease in strength of a clay from peak to residual is associated with the development of preferred orientation in thin zones. MORGENSTERN & TCHALENKO (1967) studied several cases of shear-induced fabrics by optical methods. The amount of particle orientation at residual depended on the initial particle orientation. From a detailed investigation (optical and electron microscopic) of shear induced structures in lightly overconsolidated (soft) and heavily overconsolidated (hard) kaolinite, FOSTER & DE (1971) observed a high degree of preferred orientation compared with the original materials. It was noticed that, particularly in the case of soft material, there was a 5μ band of very well oriented particles immediately adjacent to the moving boundaries.

The increased degree of clay particle alignment in the zone of failure at large displacements is associated with the residual state and results in the formation of Coulomb slip planes. YONG & MCKYES (1971) showed by overlapping individual scanning electron photo micrographs that discrete slip planes were formed in relatively thin zones. The particles were seen highly disturbed in this zone, whereas the rest of the sample had fabric indistinguishable from that of the undisturbed specimens.

Though there is ambiguity regarding the extent to which the fabric in the vicinity of the failure plane is altered by shear-induced movement, there is agreement on the fact that clay particles attain a greater degree of ordered orientation parallel to the failure plane when deformed to their residual shearing resistance. The nature of the alignment of particles in the zone of failure, and the changes in the particle orientation at the peak and residual states of deformation are indicated in Fig. 1; α is the inclination of the plane

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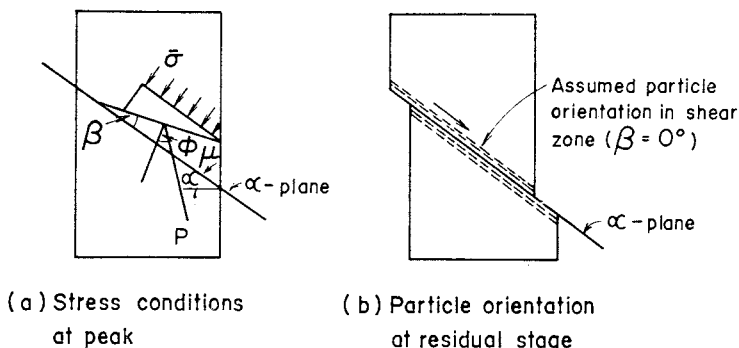


Fig. 1. Contact forces and the assumed particle alignment.

of failure to the major principal plane, and β is the average inclination of the fabric elements (clay clusters, clay aggregates or particles) to the failure plane.

Cohesion and friction form the physical components of the strength of cohesive soils. In the study of a mechanistic picture of the shear strength of clays, LAMBE (1960) recognized that cohesion is mobilized at very small strains, and, after cohesion is destroyed, it contributes no further to shearing resistance. The magnitude of the cohesion component of strength at the peak depends upon how far the cementation bonds and the bonds developed due to prestressing effects are disrupted in the shearing process. SKEMPTON (1964), in his detailed analysis of long term stability of slopes, observed that the cohesion intercept, c , tends to zero at the residual stage of deformation. The experimental data of JAMES (1971) and BISHOP et al (1971) also indicated the same trend in most cases but not in the case of cemented clays.

Friction is an important physical factor in clays (LAMBE, 1960; TROLLOPE, 1961) which is thought to result from the surface steps and dislocations of clay-sized mineral particles. Physical tangling-up due to interference between particles when sheared would contribute to frictional resistance. Recent studies (LEE & SEED, 1967; KOERNER, 1970) regarding the nature of the shear strength of cohesionless soil indicate that:

$$\bar{\Phi}_d = f(\Phi_\delta, \Phi_\mu, \Phi_{deg}, \Phi_R) \dots \dots \dots (1)$$

where $\bar{\Phi}_d$ is the drained angle of shearing resistance, and Φ_δ , Φ_μ , Φ_{deg} and Φ_R are the components of $\bar{\Phi}_d$ due to dilation, physical friction, particle degradation and particle reorientation respectively. If the same components of frictional strength as proposed for granular soils are considered valid for clays, the component due to particle crushing (degradation) will be negligible. The dilatancy component reaches a maximum value for peak stress conditions (LAMBE, 1960), beyond which it tends to decrease and approach zero when the samples are sheared to their residual stage. The value of β reflects the

magnitude of the reorientation component, Φ_R , of the measured angle of shearing resistance for peak stress conditions. At the residual state of deformation, the particles tend to align in a direction parallel to the failure plane both by sliding and reorientation. This reduces the magnitude of β which tends to zero, necessarily implying that there is particle orientation towards stable configuration. This state is reached both under drained and undrained conditions of testing since a reduction in strength corresponding to the residual state has been experimentally observed for both cases.

The above considerations indicate that, when samples of non-fissured clays are sheared to their residual stage, the cohesion component tends to zero. Since the dilatancy and orientation components also tend to zero, the frictional component is akin to material friction. When the residual conditions are attained, the soil is separated by the failure plane into two rigid blocks which slide over one another (SMART, 1970). It is possible that the value of the residual angle of internal friction is intrinsic in nature, the magnitude of which is primarily dependent on compositional factors. The experimental investigation described in this technical note is directed towards examining the validity of this under different environmental conditions. It is hoped that the above considerations might provide a basis, from particulate mechanics, to analytically account for the difference in strength between the peak and residual stages of deformation.

EXPERIMENTAL WORK

Samples of saturated kaolinite (liquid limit 49%, plastic limit 29%, shrinkage limit 27%) and montmorillonite (liquid limit 580%, plastic limit 85%, shrinkage limit 12%) clay samples were obtained by K_o -consolidation of clay slurry by an external load of 1 ton/ft². In order to study the influence of the geometrical aspects of fabric, a 6 in. diameter sample was made by consolidating a slurry (liquid limit consistency) isotropically under a hydrostatic pressure in a large triaxial cell to obtain an initial random fabric. K_o -consolidation of a slurry with a high initial water content (three times liquid limit) was used to induce an ordered orientation in the clay. Cylindrical samples of 1.5 in. diameter and 3 in. high were trimmed from the block samples for use in triaxial tests.

Triaxial compression tests were carried out using the method by WEBB (1969) to compute the normal stresses at the residual state of deformation. The testing procedures adopted were those described by BISHOP & HENKEL (1962). An overconsolidation stress history was induced by consolidating samples to the desired value of ambient pressure and allowing them to

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rebound under a lower value of stress. The strain rate adopted was 0.01 % per minute for the undrained tests in all the systems and for drained tests on the kaolinite clay; these fulfil the requirements discussed by BLIGHT (1964). The corresponding rate of strain for the drained tests on montmorillonite clay was 0.001 % per minute.

TEST RESULTS AND DISCUSSION

The area of contact between the wedges was calculated from considerations of the geometry of the shear planes as shown in Fig. 2 (WEBB, 1969). Figures 3 to 5 are typical stress-strain-pore water pressure curves. In the case of samples consolidated isotropically (Fig. 3) the postpeak behaviour tends to be independent of the consolidation pressure. For samples subjected to K_0 -consolidation, the stress-strain-pore pressure curve approaches the same value

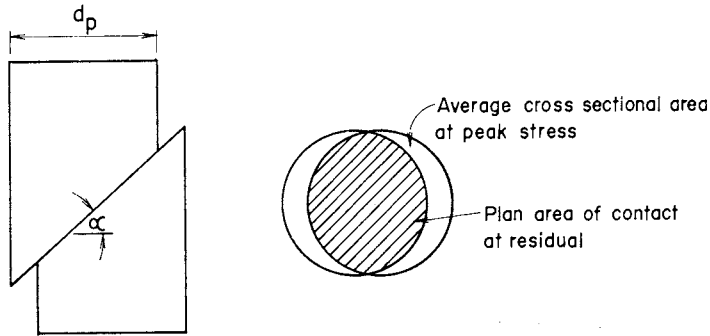


Fig. 2. Geometry of the shear plane.

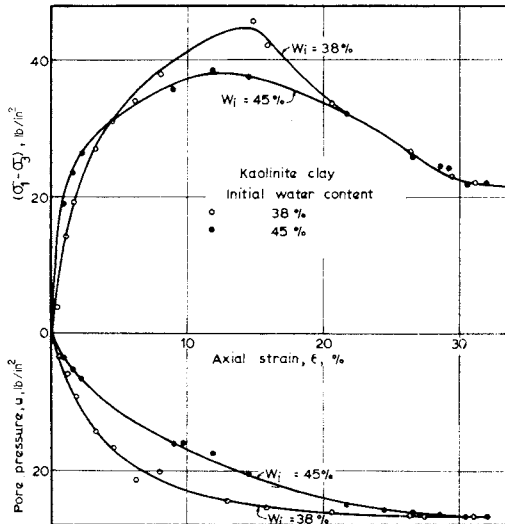


Fig. 3. Typical stress-strain-pore pressure curves from CIU tests on kaolinite clay moulded at different water contents ($\bar{\sigma}_3 = 45 \text{ lb/in}^2$).

towards the residual stage, even though the peak stresses are different (Fig. 4). In all cases, irrespective of initial conditions with respect to stress history, direction of sample and test conditions, when the samples were deformed to their respective residual levels as indicated by the stress-strain curves, changes of pore pressure and volume approached zero. It is possible that the strength contributions due to the dilatancy and reorientation components diminishes with the progress of the shearing deformation.

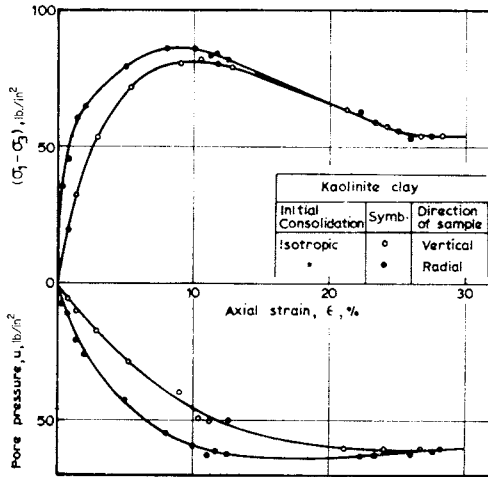


Fig. 4. Typical stress-strain-pore pressure curves from CIU tests on isotropically consolidated kaolinite clay ($\bar{\sigma}_3 = 120 \text{ lb/in}^2$).

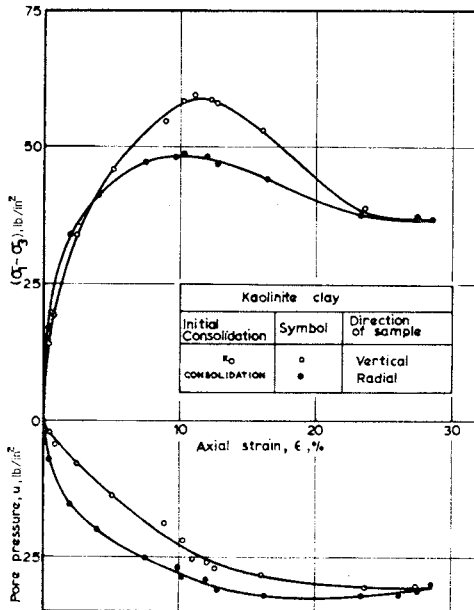


Fig. 5. Typical stress-strain-pore pressure curves from CIU tests on anisotropically consolidated kaolinite clay ($\bar{\sigma}_3 = 60 \text{ lb/in}^2$).

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The modified Mohr-Coulomb diagrams obtained are shown in Figs. 6 to 8. It can be seen that the value of the residual friction angle, $\bar{\Phi}_r$, for the kaolinite clay is 19.3° , whereas the corresponding combined diagram for different initial conditions in the case of the montmorillonite clay (Fig. 7) yields a value of 9.6° , thereby showing the influence of the type of clay. It is shown in Fig. 8 that the residual frictional angle of 20.5° encompasses all the test data, irrespective of the initial conditions and stress history (isotropic and K_0 -consolidation).

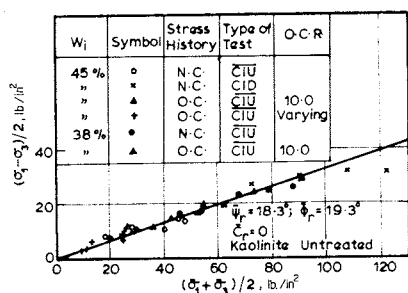


Fig. 6. Mohr-Coulomb strength line for all tests on kaolinite clay.

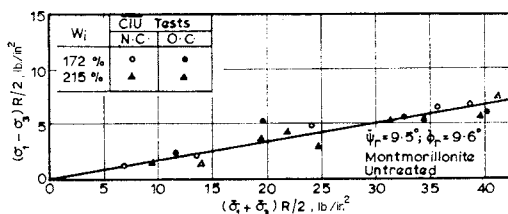


Fig. 7. Mohr-Coulomb strength line for all tests on montmorillonite clay.

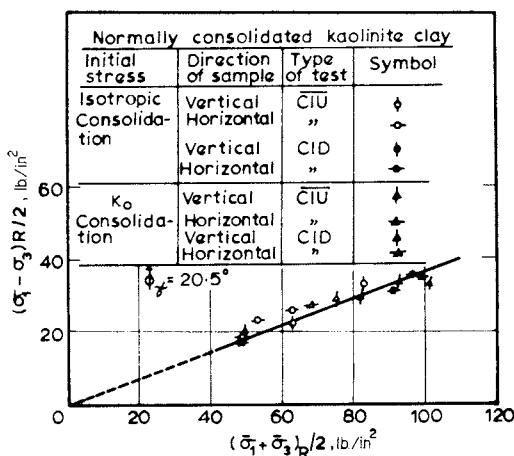


Fig. 8. Mohr-Coulomb strength line for all tests of kaolinite obtained under controlled fabric conditions.

An examination of the stress-strain-pore pressure-volume change curves indicated that, at deformations corresponding to the residual state, both the pore pressure change and volume change approach zero. This indicates that the dilatancy and reorientation components approach zero as the oriented matrix reaches a stable configuration. Since the dilatancy and reorientation components of the friction angle tend to zero, the remaining component of the frictional parameter in Eq. 1 must correspond to the residual frictional parameter, $\bar{\Phi}_r$. The limited data from this investigation indicates that the residual angle of friction is dependent on the clay type, and is independent of other factors in which the fabric conditions would play an important role. The data indicate that the cohesion intercept tends to zero for the residual stage of deformation.

Typical X-ray diffraction patterns of 002 and 020 kaolinite reflections of carbowaxed polished samples are shown in Figs. 9 to 11; these represent the initial condition of the kaolinite clay and that corresponding to the failure zone at the residual state. The orientation of platy minerals enhances basal reflections but decreases the intensity of reflections from lattice planes oriented in other directions. The degree of orientation is expressed in terms of the peak ratio intensities of mutually perpendicular planes. An increase in peak intensity ratio indicates the change towards ordered orientation (MARTIN, 1966; QUIGLEY & THOMSON, 1966). The increased peak ratio intensities from 3 to 9 indicate the shear induced ordered orientations at large deformations.

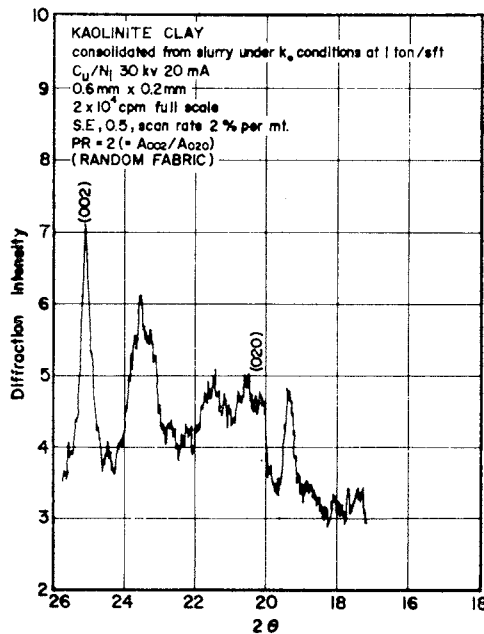


Fig. 9. X-ray diffraction pattern of carbowax polished clay before shearing.

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This would lend some support to the statements made with respect to the basic mechanisms regarding the failure pattern assumed for the residual stage of deformation.

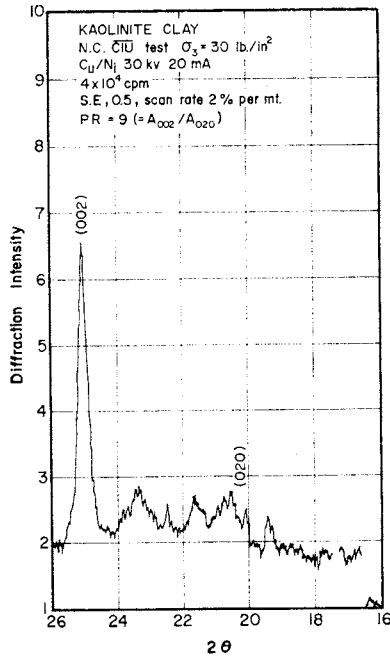


Fig. 10. X-ray diffraction pattern of sample sheared to residual stage.

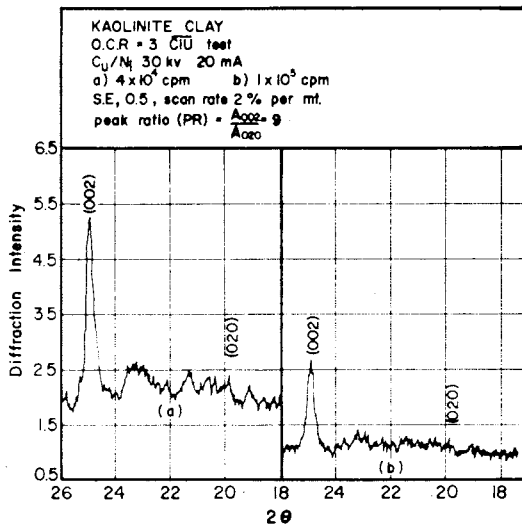


Fig. 11. X-ray diffraction pattern of sample sheared to residual stage.

CONCLUSIONS

Experimental data indicate that, as the samples were deformed to their respective residual strength level, pore pressure and volume tend to become constant, thereby indicating that the strength contribution due to dilatancy and reorientation tends to zero. Apart from type of clay, the residual frictional angle, $\bar{\Phi}_r$, was found to be independent of the variables (i) mode of preconsolidation stress application (isotropic or K_o -consolidation), (ii) initial water content of the sample, (iii) orientation of sample (vertical or horizontal), (iv) stress history (normally consolidated or overconsolidated), (v) undrained and drained conditions of test. The analyses of the data indicate that the cohesion intercept tends to zero for the residual stage of deformation.

The experimental findings lend support to the general considerations made regarding the shear strength of clays for the residual stage of deformation. Limited X-ray diffraction data show that large deformations induce ordered particle orientation. The experimental data of this study and logical analyses of the experimental findings of other investigators lend support to the basic considerations regarding the failure pattern assumed for the residual stage of deformation.

ACKNOWLEDGEMENTS

This forms part of the work done by the junior author for his Ph.D. thesis under the joint guidance of the senior author and Dr. A. Sridharan, with financial support given by the University Grants Commission and the Council of Scientific and Industrial Research. The authors thank Prof. B.V. Ranganatham for his keen interest and encouragement, and Dr. A. Sridharan for the helpful suggestions he made at all stages of the work and for his critical review of the manuscript. The facilities provided by the Indian Institute of Science are gratefully acknowledged.

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INTERNATIONAL SOCIETY NEWS

The following is the text of a newsletter received in October 1973 from the Secretary General of the International Society, Professor J.K.T.L. Nash.

8th International Conference — The 8th ISSMFE International conference took place in Moscow from 6-11 August, 1973. These conferences are the most important part of the life of the Society and 1750 delegates plus 600 accompanying persons from 50 different countries attended this one. The organisational problems must clearly have been very considerable and the Russian National Society is to be warmly congratulated for their part in making it a success.

The general arrangement was to have lectures from specialists first thing each morning, followed by Main Sessions. Then each afternoon there were two Specialty Sessions where the discussion was less formal. On the final morning the President, Dr. Ralph Peck, spoke impressively about the state of the art of soil mechanics, and his address which will be published in the final volume of the Proceedings deserves to be widely read.

Proceedings—The first six volumes of the Proceedings were issued to those who attended the Conference and the complete set may be ordered from any bookseller who handles Soviet literature or who deals with V/O 'Mezhdunarodnaya Kniga' Moscow G-200, USSR. Alternatively, orders may be placed with Collet's Russian Bookshop, 39 Museum Street, London WC 1, who are prepared to post to any country in the world. The cost of the full set of the Proceedings will be 30 roubles plus postage.

The volumes are in soft covers and it is planned to issue a specification for binding so that these will appear uniform on bookshelves throughout the world.

Executive Committee — The Executive Committee of the Society met in Moscow on 2nd and 3rd August and copies of the minutes in English and French will be published in the Proceedings of the Conference. The German Democratic Republic (G.D.R.), Iran, Pakistan and Tunisia were welcomed into membership, so that we now have 48 National Societies with some 10,000 members in all.

Subscriptions to ISSMFE, which since 1965 have remained constant at US. \$ 25 + \$ 0.75 per member for each country, have now reluctantly had to be revised to provide a larger budget. A great deal of time and thought was given to this matter and it was agreed that a new system should apply from 1st

January 1974, the rates depending on the intrinsic wealth of the country. Thus the fee will be \$ 100 + \$ 1.00 per member for Ghana, Morocco, Tunisia, Ecuador and Peru, whereas for Canada, France, Federal Republic of Germany (F.R.G.), Italy, Japan and the U.K. it will be \$ 100 + \$ 1.75, and \$ 100 + \$ 2.05 for the U.S.A. and U.S.S.R.

The following officers were elected for the period 1973 to 77 :

President	Jean Kerisel	(France)
Vice-Presidents :		
Africa	J.W. de Graft Johnson	(Ghana)
Asia	Z.C. Moh	(S E Asia)
Australasia	P.W. Taylor	(New Zealand)
Europe	A. Kézdi	(Hungary)
N. America	R.J. Marsal	(Mexico)
S. America	V.F.B. de Mello	(Brazil)
Secretary General	J K T L Nash	(U.K.)

The next meeting of the Executive Committee will take place in Istanbul in 1975 when the main business will be the plans for the IXth International Conference in 1977 for which the Japanese National Society have kindly agreed to act as hosts. Members are warmly invited to send suggestions for the arrangements for this conference to the Secretary General as soon as possible.

J.K.T.L.N.

SOUTHEAST ASIAN SOCIETY NEWS

Professor Moh becomes Vice President of International Society

Prof. Za-Chieh Moh, Professor and Chairman of Geotechnical Engineering at the Asian Institute of Technology, has been elected Vice President for Asia of the International Society for Soil Mechanics and Foundation Engineering. Prof. Moh was President of the Southeast Asian Society of Soil Engineering from 1967 to 1973, when he was succeeded by Prof. Chin Fung Kee.

The Fourth Conference on Soil Engineering

The Society's Fourth Conference on Soil Engineering is planned to take place in Kuala Lumpur from 7 to 10 April 1975. The Fourth Conference follows the first three held in Bangkok (1967), Singapore (1970) and Hong Kong (1972): all of these were a great success. This early notice has been given so that Society members and others may prepare their contributions and make plans to attend. The Society Conferences present an opportunity for those interested in soil engineering to spend a few days discussing problems of mutual interest. Participants have found past Conferences to be of great professional value, especially since the papers and discussions have tended to focus on problems of the region.

The Fourth Conference will be sponsored by the Southeast Asian Society of Soil Engineering, the Institution of Engineers, Malaysia, and the Asian Institute of Technology. The language of the Conference will be English. The Organizing Committee intends to invite an eminent guest lecturer in the field of soil engineering.

There will be no special theme for the Conference. Papers may be submitted on general topics of soil mechanics and engineering geology dealing with testing and site investigations, foundations, earth dams, slope stability, and roads and runways. Three copies of a preliminary title and brief summary of each paper are required to be submitted by 15 January 1974. Final manuscripts will be required by 15 July 1974.

Bulletins Nos 1 and 2, which gives full details about the Conference and the procedure for the submission of papers, are now available from The Secretary, IV SEACSE, c/o Institution of Engineers, Malaysia, P.O. Box 223, Petaling Jaya, Selangor, Malaysia.

CONFERENCE NEWS

The Fourth Southeast Asia Conference on Soil Engineering will be held in Kuala Lumpur, Malaysia from 7 to 10 April 1975. It will be sponsored by the Southeast Asian Society of Soil Engineering, the Institution of Engineers Malaysia, and the Asian Institute of Technology. The language of the Conference will be English.

There will be no special theme for the Conference. Papers may be submitted on general topics of soil mechanics and engineering geology dealing with testing and site investigations, foundations, earth dams, slope stability, and roads and runways. Three copies of a preliminary title and brief summary of each paper are required to be submitted by 15 January 1974. Final manuscripts will be required by 15 July 1974.

Bulletin No. 2 is now available from The Secretary, IV SEACSE, c/o Institution of Engineers, Malaysia, P.O. Box 223, Petaling Jaya, Selangor, Malaysia.

A Conference on the Settlement of Structures will be held at Cambridge University from 2 to 4 April 1974. The proceedings will be conducted under the headings of (1) Granular materials (2) Normally consolidated and lightly over-consolidated cohesive materials, (3) Heavily over-consolidated cohesive materials (4) Rocks, and (5) Allowable and differential settlements, including damage to structures and soil-structure interaction.

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.

The Sixth European Conference on Soil Mechanics and Foundation Engineering will be held in Vienna in March 1976 on the general theme *Deep Foundation and Deep Excavations*. The technical sessions will be :

- (1) Deep excavations; stability of temporary and permanent slopes; dewatering problems, slurry walls, walls with batter piles; bracing, freezing techniques.
- (2) Deep foundations; tunnelling.
- (3) Deep foundations in open pits; pile foundations; caisson foundations.

All correspondence should be addressed to the Secretary, VI European Conference on S.M.F.E., A 1040 Wien, Technische Hochschule, Karlsplatz 13, Austria.

A Specialty Conference on Analysis and Design in Geotechnical Engineering will be held from 9 to 12 June 1974 at The University of Texas at Austin under the auspices of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers.

There will be five general sessions, four of which will feature a state-of-the-art speaker on analysis and design relating to embankments, foundations for light structures, foundations for heavy structures, and underground structures. The fifth general session will be an overview of the Conference and will include the participation of a panel.

A feature of this Specialty Conference will be a series of workshops, in which speakers will present methods of analysis and design in a variety of areas, probably making use of documented computer programs, with audience participation. The speakers will make available copies of documentation to workshop participants at the cost of reproduction.

Papers related to the theme of the conference are solicited, and those accepted will be published in the Proceedings of the Conference along with the addresses of the state-of-the-art speakers and workshop leaders. Finished copies of papers (typed on available model paper) must be submitted no later than 15 December 1973.

Those wishing to participate in the Conference should write now to The Secretariat, 1974 ASCE Conference, Taylor Hall 161, The University of Texas, Austin, Texas 78712, U.S.A.

A European Symposium on Penetration Testing is planned for 5 to 7 June 1974 in Stockholm, Sweden. The aim of the Symposium is to document the use of penetrometer testing in soil investigations, to outline areas where further research is desirable, to investigate the need for standardisation, and to provide guidelines for future developments.

The organizers have limited the number of participants to about 80 specialists comprising one to five delegates from each national society in Europe. National societies are invited to contribute state-of-the-art reports about the practice of penetration testing in their countries and, in addition, individual papers are invited. The Proceedings will be published in two volumes.

Information can be obtained from Dr. Bengt Broms, Swedish Geotechnical Institute, Banergatan 16, Stockholm, Sweden.

The 25th International Geological Congress will be held in Sydney, Australia from 16 to 25 August 1976. Those who wish to receive a copy of the first

circular when it is available are asked to write to The Secretary-General, 25th International Geological Congress, P.O. Box 1892, Canberra City, ACT 1601. Australia.

The **Third International Congress on Rock Mechanics** of the International Society for Rock Mechanics will be held in Denver, Colorado, U.S.A. from 1 to 7 September 1974. The object of this Congress is to ascertain on an international scale the advances that have been made in rock mechanics since the Second International Congress, and to indicate directions for future effort.

Attention will be centred on the major themes (1) Physical properties of intact rock and rock masses, (2) Tectonophysics, (3) Surface workings, (4) Underground openings, and (5) Fragmentation systems. At each Plenary Session, the major theme will be reviewed by a Congress General Reporter and a panel will respond to questions submitted prior to the Congress sessions. Selected individual papers will be presented and discussed at the discussion sessions. Brief questions from the floor will be permitted if time allows. The Congress technical sessions will be presented in English, French and German by simultaneous translation.

Bulletin No. 2 contains details of the Congress, and is now available. This can be obtained from Secretary, Third International Congress on Rock Mechanics, 2101 Constitution Avenue NW, Washington, D.C. 20418, U.S.A.

The **Second Australia-New Zealand Conference on Geomechanics** will be held in Brisbane, Australia from 21 to 25 July 1975. Papers are invited on research, practical applications and reviews of the present state of knowledge in all fields of soil and rock mechanics. Topics of sessions will depend on the papers received but it is hoped they will include (1) Measurement Techniques in Soil and Rock Mechanics, (2) Application of Computer Techniques in both Soil & Rock Mechanics, (3) Environmental Aspects of Geomechanics Studies, (4) Design of Earth and Rock Structures, (5) Fracture Mechanics, (6) Foundation Behaviour, (7) Slope Stability, (8) Handling Non-Homogeneous Material, (9) Roads and Pavements (10) Engineering Geology, (11) Groundwater Hydrology.

Intending authors are requested to forward the titles of proposed papers together with a 250 word synopsis of the subject matter as soon as possible, but not later than 30 April, 1974. Authors will be notified of the acceptance of their papers in June, 1974 and will then be issued with detailed instructions on the preparation of papers. Completed papers must be received by 30 November, 1974. Those papers accepted for presentation will be preprinted and distributed before the Conference.

All correspondence relating to the Conference should be addressed to :
The Secretary, 2nd Australia-New Zealand Conference on Geomechanics, 157
Gloucester Street, Sydney, N.S.W. 2000, Australia.

The **Second International Congress on Engineering Geology**, organised by the Associacao Brasileira de Geologia de Engenharia under the patronage of UNESCO, will be held in Sao Paulo, Brazil from 18 to 24 August 1974. The themes that will be dealt with are (1) Teaching and Training in Engineering Geology, (2) Seismic Phenomena and Engineering Geology, (3) Engineering Geology Related to Urban and Country Planning, (4) Engineering Properties and Classification of Natural Materials of Construction, (5) Mass Movements, (6) Engineering Geology Relating to Dam Foundations, (7) Engineering Geology and Underground Construction.

The second bulletin is now available from Secretary-General, II Congress IAEG, Instituto de Pesquisas Tecnologicas, Caixa Postal 7141, Sao Paulo - SP, Brazil.

NEWS OF PUBLICATIONS

Proceedings of the International Symposium on Soil Structure, held in Gothenburg in August 1973, may be obtained from Konsultforetagens Service-Kontor, Greveturegaten 29, 114.38 Stockholm, Sweden, price S. Kr. 55 plus postage.

Two volumes of **Proceedings of the Specialty Session on Lateritic Soils**, held at the Seventh International Conference on Soil Mechanics and Foundation Engineering in Mexico City in 1969, are available for U.S. \$ 25 from the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

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. Orders for this volume 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".

The two volumes of **Proceedings of the Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering**, Bangkok, July 1971 are available at U.S. \$ 30 from the Geotechnical Engineering Division, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

Proceedings of the Symposium on Engineering Geological Properties of Clays and Processes in Them, held in Moscow during 15 to 23 September 1971 is now available from Intern. Assoc. of Engineering Geology, 60, Boulevard Saint Michel, Paris 6, price U.S. \$ 12 (paper bound).

Proceedings of Conferences of the International Society for Soil Mechanics and Foundation Engineering held to date are listed in the following tables :

(1) *INTERNATIONAL CONFERENCES*

Number, venue & year	Publishers	Cost
1st, Harvard, 1936	Geotechnical Engineers Inc, 934 Main Street, Winchester, Mass. 01890. U.S.A.	U.S. \$ 40.00 bound reprint
2nd, Rotterdam, 1948	-	Out of print
3rd, Zürich, 1953	Société Suisse de Mécanique des Sols et des Travaux de Fondations, Case Postale 8022, Zürich, Switzerland.	U.S. \$ 70.00
4th, London, 1957	-	Out of print
5th, Paris, 1961	Comité Français de la Mécanique des Sols et des Fondations, 31, rue Henri-Rochefort, Paris 17 ^e , France.	F.F. 213.79
6th, Montreal, 1965	University of Toronto Press, University of Toronto, Toronto 5, Canada.	Can. \$ 100.00
7th, Mexico City, 1969	Sociedad Mexicana de Mecánica de Suelos A.C. Apartado Postal 2800, Mexico 1, D.F. Mexico.	U.S. \$ 40.00
8th, Moscow, 1973	Through any bookseller that handles Soviet literature or deals with V/O 'Mezhdunarodnaya Kniga' Moscow G-200, USSR. OR Collet's Russian Bookshop, 39 Museum St. London WC1, England who will post to any country in the world.	30 Roubles plus postage

(2) *AFRICAN CONFERENCES*

1st, Pretoria, 1955	-	Not available*
2nd, Lourenco Marques, 1959	-	Out of print
3rd, Salisbury, 1963	-	Out of print
4th, Cape Town, 1967	Messrs A.A. Balkema, 93 Keerom Street, Cape Town, South Africa.	S.A.R. 20.00
5th, Luanda, 1971	The Secretary, 5th RCASMFE, Caixa Postale 6500, Luanda, Angola.	U.S. \$ 40.00

* Proceedings were published in *Trans. S.A. Instn. Civ. Engrs.*, 1955, 5, 263-322, 406-478.

(3) *ASIAN CONFERENCES*

1st, New Delhi, 1960	-	Out of print
2nd, Tokyo, 1963	Japanese Society of Soil Mechanics and Foundation Engineering, Toa Bekkan Building, 13-5, I-chome, Nishi-Shinbashi, Minato-ku, Tokyo, Japan.	U.S. \$ 20.00
3rd, Haifa, 1967	-	Out of print
4th, Bangkok, 1971	Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.	U.S. \$ 30.00

(4) *AUSTRALIA - NEW ZEALAND CONFERENCES*

1st, Melbourne, 1952	-	Out of print
2nd, Christchurch, 1956	-	Out of print
3rd, Sydney, 1960	-	Out of print
4th, Adelaide, 1963	Institution of Engineers Australia, Science House, 157 Gloucester Street, Sydney N.S.W. 2000, Australia.	Aus. \$18.50 plus postage
5th, Auckland, 1967		Aus. \$ 25.00 plus postage
6th, Melbourne, 1971 (1st Geomechanics)		Aus. \$ 40.00 plus postage

(5) *EUROPEAN CONFERENCES*

1st, Stockholm, 1954	Swedish Geotechnical Society, Banérgatan 16, 115 26 Stockholm, Sweden.	Only Vol. 3 available at S.Kr. 26.00
2nd, Brussels, 1958	-	Out of print
3rd, Wiesbaden, 1963	-	Out of print
4th, Oslo, 1967	Norwegian Geotechnical Institute, Forskningsveien 1, Oslo 3, Norway.	N.Kr. 210
5th, Madrid, 1972	Sociedad Española de Mecánica del Suelo y Cimentaciones, Laboratorio del Transporte y Mecánica del Suelo, Alfonso XII No. 3, Madrid 7, Spain.	U.S. \$ 39.00

(6) *PANAMERICAN CONFERENCES*

1st, Mexico City, 1960	Sociedad Mexicana de Mecánica del Suelos, A.C., Apartado Postal 8200, Mexico D.F., Mexico.	U.S. \$ 30.00
2nd, São Paulo, 1963	Associação Brasileira de Mecânica dos Solos, Rua Joaquim Nabuco. 254-ap. 201, Rio de Janeiro, Guanabara ZC-37, Brasil.	U.S. \$ 30.00
3rd, Caracas, 1967	Sociedad Venezolana de Mecánica del Suelo e Ingeniería de Fundaciones, Apartado 4074-Este, Caracas, Venezuela.	U.S. \$ 30.00
4th, Puerto Rico, 1971	American Society of Civil Engineers, 347 E. 47th Street, New York, N.Y. 10017, U.S.A.	U.S. \$ 20.00

ASIAN INFORMATION CENTER FOR GEOTECHNICAL ENGINEERING — AGE

The *Asian Information Center for Geotechnical Engineering* (Asian Geotechnical Engineering for short, abbreviated to AGE) has been established within the library of the Asian Institute of Technology under the joint sponsorship of its Division of Geotechnical Engineering and the Library. The idea of establishing AGE was conceived at the meeting of representatives of national societies of soil mechanics and foundation engineering in the Asian region which was convened in Bangkok in July 1971. Through one of the resolutions of the meeting, AIT was requested to undertake the task of establishing and operating AGE for the benefit of engineers in Asia and those interested in the region.

The newly established AGE is an invaluable source of information for all those concerned with investigation, feasibility, design and construction for all types of civil engineering projects. In addition, it is indispensable to those concerned with teaching and research in any aspect of geotechnical engineering. It is aiming to serve as a clearing house in the Asian region for information on SOIL MECHANICS, FOUNDATION ENGINEERING, ROCK MECHANICS, ENGINEERING GEOLOGY, EARTHQUAKE ENGINEERING, and other related fields. In cooperation with national societies, universities, governmental agencies, research organizations, engineering and consulting firms, contractors, etc., both within and outside the region, AGE is collecting information on all phases of geotechnical engineering research and projects, including published and unpublished reports which are of relevance to Asian conditions, AGE is undertaking there sponsibility of designing a computer based information storage and retrieval system for the effective handling of such information, and is providing both *Current Awareness Service and Selective Dissemination of Information Service* through its publication of journal abstracts and subject bibliographies. Dissemination of collected information takes place through photocopying and micro-filming.

The Head Librarian of AIT serves as Director of the Center, which operates as a special administrative unit of the AIT Library, and is guided by a Policy Advisory Committee and a Technical Committee. The Technical Committee meets at least once every two months to advise the Director of the Center on all important technical and operational matters. At the moment, Liaison Officers act as communication links between their country and AGE in Bangkok.

To ensure the effective functioning of AGE as an information center for geotechnical engineering in Asia, it is necessary that all organizations and

individuals who are engaged in any kind of geotechnical engineering work in Asia consider AGE as a central depository for their information and publications regardless of language. These publications and other information will be abstracted and analyzed, and will be publicized and disseminated through AGE's various channels to the benefit of the region. Among the major data files which have been started at AGE are :

- (1) Data on all design, construction and research projects in geotechnical engineering of concern to the region.
- (2) Data on organizations engaged in any kind of geotechnical engineering work in the region.
- (3) Data on individuals who are engaged in any kind of geotechnical engineering work in the region.
- (4) Data on published papers and technical literature on geotechnical work of concern to the region.

Great benefit can be gained by companies and research organizations treating AGE as a central depository and service agent to provide information when it is needed.

AGE has started to publish the following :

- (1) *Asian Geotechnical Engineering Abstracts* : a quarterly publication consisting of abstracts of available publications on geotechnical engineering relevant to Asia.
- (2) *Asian Geotechnical Engineering in Progress* : a semi-annual publication consisting of information on current design, construction and research projects in geotechnical engineering being undertaken in Asia.
- (3) *Asian Geotechnical Engineering Directory* : a bi-annual publication consisting of information on various organizations and individuals who are doing geotechnical engineering work in Asia or relevant to Asia.
- (4) *AGE Current Awareness Service* : published quarterly to inform readers of recent geotechnical engineering publications and contents of geotechnical engineering journals received at AGE.
- (5) *AGE Journal Holdings List* : published annually to facilitate the request of photocopies.
- (6) *AGE Bibliography Series* : either recurrent or demand bibliographies published as a result of general interest or demand.

In addition to its publications, AGE will provide the following three services :

- (1) *Reference Service* : for bibliographical questions.
- (2) *Referral Service* : for technical questions.
- (3) *Reproduction Service* : for photocopying or microfilming of required documents.

AGE is a non-profit making service organization. For the initial three years, it is being financially supported by a generous grant from the International Development Research Centre of Canada and by the Asian Institute of Technology. It will be necessary, however, for **AGE** to recover a very small portion of its operation costs from fees received on certain services. Much can be gained through membership which is available to Individuals and Institutions at nominal fees.

Anyone who wishes to have details about the Asian Information Center for Geotechnical Engineering should write to Dr. H.W. Lee, Director AGE, P.O. Box 2754, Bangkok, Thailand.

BOOK REVIEWS

The Strength Properties of Rocks by W. Dreyer, translated from German, Trans Tech Publications, Clausthal, Germany and Cleveland, Ohio, U.S.A., 1972, U.S. \$ 25.

The Strength Properties of Rocks is Part I of the monography "The Science of Rock Mechanics" and deals primarily with the relationship between strength and textural parameters of rocks. It contains a vast amount of information and is probably the most extensive literary work on the strength properties of halite rocks. The first half of the book deals with the mechanical behaviour of a single rock salt crystal and successfully attempts to predict the strength properties of polycrystalline rock salt from the properties of its component elements. The second half is devoted to the design of pillars and large underground storage cavities in Halite rocks. In view of its coverage, the book would be more appropriately called "The Strength Properties of Halite Rocks".

The book has no subject index. With the exception of the few classical publications cited, there is almost a complete absence of English language references and, consequently, the bibliography is incomplete. It is thought, however, that the book will be useful to civil, geological and especially mining engineers who deal with the design and construction of mine opening in halite rocks.

A. Kazi

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, 1974) should be sent so as to reach the Editor not later than 1 June, 1974.