GEOTECHNICAL

ENGINEERING

CONTENTS

DRAINED DEFORMATION CHARACTERISTICS OF SAND

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SYNOPSIS

Deformation of sand when subjected to isotropic stresses was studied under loading and unloading conditions. A relationship between the elastic volumetric deformation and the ambient pressure is developed. Deformation behaviour of sand was also investigated by conducting strain-controlled drained triaxial compression and extension tests. A unique relationship exists between the rate of dilatation at failure and the void ratio for these two types of test. Comparison of the compression and extension tests shows that the Mohr-Coulomb criterion describes failure much closer than the extended von Mises or the extended Tresca criteria. The angle of internal friction determined from extension tests for any void ratio is a few degrees higher than that of the compression tests, except at the very loose state when they are equal. Thus, the Mohr-Coulomb failure criterion underestimates the strength of soils in many stability problems. The corrected deviator stresses, which represent the frictional component only, satisfy the Mohr-Coulomb failure criterion.

INTRODUCTION

The peak or ultimate values of shear strength determined in the laboratory using the standard triaxial (axi-symmetric) apparatus are used in the analysis of practical problems in soil mechanics such as the design of foundations, slopes and retaining structures. The soil is assumed to satisfy the Mohr-Coulomb failure criterion which considers that the intermediate principal stress has no influence on the failure strength. The deformations that occur in the soil are neglected in the conventional analysis and the problem is treated purely from the strength aspect.

In the recent past, some research workers (ROSCOE et al, 1958; ROWE, 1962) have emphasised the need for stress-deformation analysis of soil mechanics problems. Two basically different approaches have been developed so far. In the approach developed by Roscoe and his co-workers at Cambridge the soil is assumed to be an elastic-plastic continuous medium, while the other approach developed by Rowe and his co-workers at Manchester is based on particulate mechanics. Extensive research into shear behaviour of soils has been carried out in both Cambridge and Manchester

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using the standard (axi-symmetric) triaxial apparatus, and stress-strain relationships have been developed.

The standard triaxial apparatus has been used by many to investigate whether the widely used Mohr-Coulomb failure criterion is applicable to soils, and the results obtained are rather conflicting. Some (BISHOP & ELDIN, 1953; CORNFORTH, 1964; KIRKPATRICK, 1957) concluded that the Mohr-Coulomb failure criterion is applicable to sands. Therefore, for any void ratio, the angle of internal friction, ϕ', determined from triaxial compression tests in which the intermediate principal stress is equal to the minor principal stress, is equal to ϕ' determined from triaxial extension tests in which the intermediate principal stress is equal to the major principal stress. But GREEN & BISHOP (1969) and ROSCOE et al (1963) found that ϕ' measured in triaxial extension is a few degrees higher than ϕ' measured in triaxial compression for any void ratio.

Shear properties of soils have also been studied using other types of shear apparatus in which the intermediate principal stress can be controlled independent of the major and minor principal stresses. Results obtained show that ϕ' determined from these tests is a few degrees higher than ϕ' determined from the triaxial compression tests for any void ratio. A summary of these results was presented by MESDARY & SUTHERLAND (1970), from which it is evident that the question of the influence of intermediate principal stress on failure strength still remains unresolved.

The deformation characteristics of a fine sand when subjected to isotropic loading and to shear loading under drained conditions in the triaxial (axi-symmetric) apparatus have been studied herein. The types of test carried out were (i) loading and unloading under isotropic stress conditions, (ii) compression and extension tests at constant cell pressure, σ_r, and (iii) compression and extension tests at constant mean principal effective stress, p. The behaviour of this fine sand during undrained triaxial shear tests has been already presented (THURAIRAJAH & LELIEVRE, 1971).

SOIL MATERIAL

The soil material selected for testing was the portion of a fine Ottawa silica sand that passed through a No. 60 sieve (0.25 mm) and was retained on a No. 200 sieve (0.074 mm). 21% of this material passed through a No. 100 sieve (0.149 mm). The sand grains had a specific gravity of 2.65. This fine sand was particularly chosen for the tests in order to minimise errors due to membrane penetration (NEWLAND & ALLELY, 1959). Every test was conducted with a fresh sample of the sand since shearing might have caused some fracturing of the sand grains.

TESTING PROCEDURE

An improved triaxial testing technique was used to carry out drained triaxial tests on saturated samples of the sand set at different initial densities. The samples tested were 4 in. in height and 2 in. in diameter. The conventional end platens of the triaxial sample were replaced by enlarged stainless steel platens of 2.25 in. diameter, which were carefully ground and polished. The friction at the end platens was minimised by placing a 0.01 in. thick rubber disc on each platen with a thin layer of silicone grease between the rubber disc and the polished surface of the platen (BISHOP & GREEN, 1965; ROWE & BARDEN, 1964). Drainage of the sample was facilitated through a 0.5 in. diameter porous stone fixed to the centre of the bottom end platen.

The error in measured axial load due to friction between the loading piston and the collar of the triaxial cell top in the conventional apparatus was eliminated by measuring the load internally using an electrical resistance strain gauge type load cell attached to the bottom end of the loading piston. The membrane penetration in the triaxial sample was reduced to a negligible value by testing fine sand and using a 0.015 in. thick rubber membrane.

A sand former was made to accommodate the enlarged end platens. Saturated triaxial samples were obtained in a loose state of packing by gently spooning freshly boiled sand into the sand former filled with water. Denser samples were obtained by tamping and mechanical vibration. Samples thus obtained had void ratios ranging from about 0.7 in the loose state to about 0.5 in the dense state.

Isotropic consolidation and swelling tests were performed on the samples of sand to study their elastic behaviour under such stress conditions. A series of conventional drained triaxial compression and extension tests was carried out at constant cell pressure, σ_r, up to a maximum value of 80 lb/in^2. A few drained triaxial compression and extension tests were also conducted with the mean principal effective stress, p, kept constant during the tests at a value of 60 lb/in^2. This was achieved by controlling the cell pressure during the tests, and the variation of p from the value of 60 lb/in^2 was less than 0.5 lb/in^2.

All the shear tests were performed under strain-controlled conditions. An axial deformation rate of 0.004 in/min was used for the σ_r-constant tests and a rate of 0.002 in/min was used for the p-constant tests.

STRESS AND STRAIN PARAMETERS

The stress parameters used herein are the mean principal effective stress, $p = \frac{1}{3}(\sigma_a' + 2\sigma_r')$, and the deviator stress, $q = (\sigma_a' - \sigma_r')$, where σ_a' and σ_r' are the

axial and radial effective stresses respectively. The strain parameters used are the natural volumetric strain, v, and the natural shear strain, ε, given by:

$$\delta\varepsilon = \delta\varepsilon_a - \frac{\delta v}{3} \quad \ldots \ldots \ldots \ldots \ldots \quad (1)$$

where $\delta\varepsilon_a$ is the incremental natural axial strain. Compressive strains are considered to be positive.

ISOTROPIC CONSOLIDATION AND SWELLING TESTS

A series of consolidation and swelling tests were carried out on triaxial samples of the fine sand set at different densities of packing. The samples were initially allowed to reach equilibrium under a cell pressure of 5 lb/in² and then consolidated under cell pressures of 10, 20, 30, 45, 60, 80, 100, and 120 lb/in². The changes in height and volume of the samples were measured accurately at these pressures once equilibrium was reached. The samples were then allowed to swell under these pressure up to 5 lb/in², and changes in height and volume were again measured.

In Figs. 1 and 2 the volumetric strains, v, are plotted against the axial strains, ε_a, as observed during six consolidation and swelling tests, 5 lb/in² being the datum for zero strain. The line in these figures has a slope of one-third; if the sand is isotropic, axial strains will be one-third the volumetric
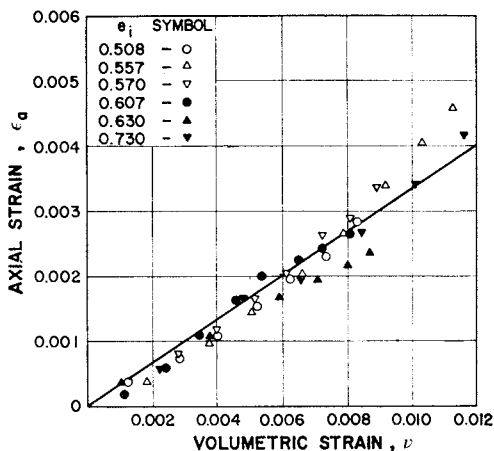

Fig. 1. Relationship between axial strain and volumetric strain during consolidation tests.
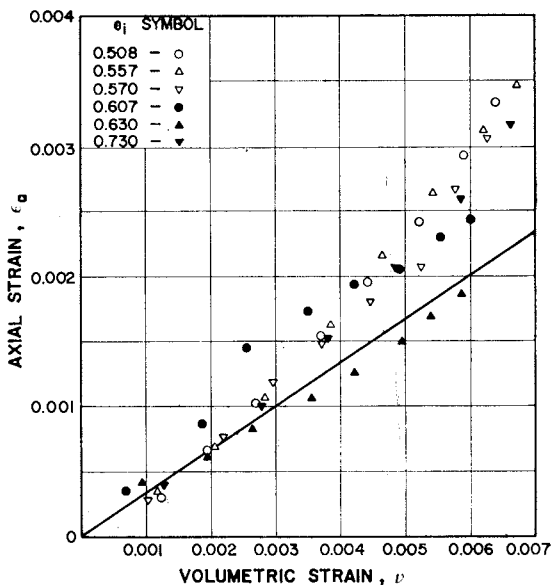
Fig. 2. Relationship between axial strain and volumetric strain during swelling tests.

strains and the experimental points will lie on this line. For the consolidation tests, the ratio between axial strain and volumetric strain varies from about 0.25 to 0.4, while for the swelling tests, the ratio varies from about 0.3 to 0.5. These results show that the sand exhibited a high degree of isotropy during the compression tests but some anisotropy during the swelling tests.

RELATIONSHIP BETWEEN ELASTIC VOLUMETRIC STRAIN AND AMBIENT PRESSURE

The application of Hertz's contact theory to the elastic behaviour of randomly packed spherical particles subjected to an ambient pressure, P, shows that the elastic volumetric strain, V, is related to P by (WILSON & SUTTON, 1948):

$$V = c\,P^{2/3} \qquad \ldots \ldots \ldots \ldots \quad (2)$$

where c is a constant. For a packing of sand grains which are not spherical, the elastic volumetric strain is given by (EL SOHBY, 1969):

$$V = c\,P^m \qquad \ldots \ldots \ldots \ldots \quad (3)$$

where m is a constant for the sand and depends on the rate of increase in the contact area between the particles with the ambient pressure. m has a value less than two-thirds.

In practice, the volumetric strain is measured with increase in ambient pressure starting from an initial consolidation pressure. In the series of tests described herein, the triaxial samples were allowed to reach equilibrium under a cell pressure of 5 lb/in², and volumetric strains were measured from this datum with increase in ambient pressure. If V' is the volumetric strain measured for an ambient pressure P, then V' is given by:

$$V' = c\,(P^m - P_o^m) \qquad \ldots \ldots \ldots \ldots \quad (4)$$

where $P_o = 5$ lb/in².

An analytical calculation to determine the constants c and m for the sand is tedious to perform. A trial and error graphical solution has been suggested by EL SOHBY (1969). An alternate approach for determining these constants is used in this paper.

Differentiating Eq. (4) gives:

$$\frac{dV'}{dP} = c\,m\,P^{(m-1)} \qquad \ldots \ldots \ldots \ldots \quad (5)$$

Therefore:

$$\log\left(\frac{dV'}{dP}\right) = \log cm + (m-1)\log P \qquad \ldots \ldots \quad (6)$$

If $\log (dV'/dP)$ is plotted against $\log P$, the points would lie on a straight line. This line would have a gradient of $(m-1)$, and the intercept on the

log (dV'/dP) axis would be log cm. Hence, the value of the constants c and m can be determined directly from the gradient and intercept of this line.
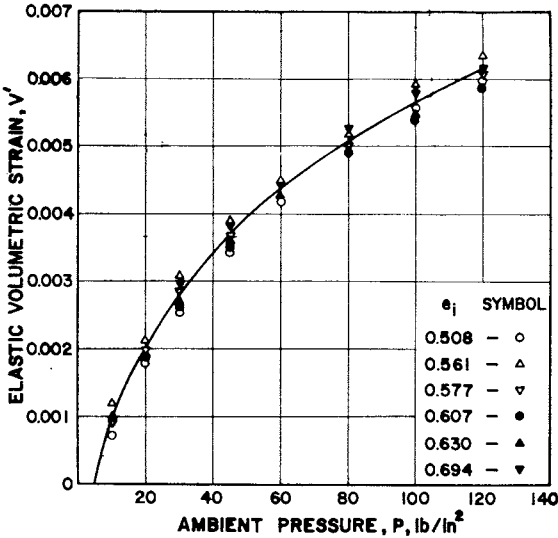

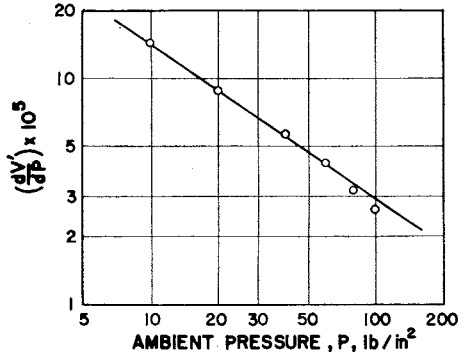Fig. 3. Variation of elastic volumetric strain with ambient pressure.

Fig. 4. Variation of dV'/dP with ambient pressure.

In Fig. 3, the elastic volumetric strain, V', is plotted against the ambient pressure, P, for six swelling tests, and the mean curve is drawn through these points. The values of dV'/dP determined from the gradients of this curve is plotted against P in a logarithmic plot in Fig. 4. A straight line is drawn through these points, and the values of the constants c and m determined from this line are 0.00227 and 0.31 respectively. Thus, the elastic volumetric strain, V, for this sand under ambient pressure P follows the power law given by:

$$V = 0.00227 \, P^{0.31} \quad \ldots \ldots \ldots \ldots (7)$$

This relationship is used later for estimating the "elastic energy correction" applied to the deviator stress to obtain the corrected deviator stress (ROSCOE et al, 1963).

ENERGY CORRECTIONS

Consider a triaxial sample of soil which is in equilibrium under a mean principal effective stress p and deviator stress q. The sample is allowed to undergo volumetric strain δv and shear strain $\delta \varepsilon$ due to stress increments δp and δq. The total energy, δE, supplied to the sample is related to the recoverable elastic energy stored, δU, and the dissipated energy, δW, by (ROSCOE et al, 1963):

$$\delta E = \delta U + \delta W = q \cdot \delta \varepsilon + p \cdot \delta v \quad \ldots \ldots \ldots (8)$$

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If $\dfrac{dW}{d\varepsilon}$ is defined as q_w, then:

$$q_w = q + p \cdot \frac{dv}{d\varepsilon} - \frac{dU}{d\varepsilon} \qquad \cdots \cdots \cdots \quad (9)$$

where $p \cdot \dfrac{dv}{d\varepsilon}$ and $-\dfrac{dU}{d\varepsilon}$ are called the "boundary energy correction" and the "elastic energy correction" respectively. q_w is referred to as the corrected deviator stress.

From Eq. 7:

$$\delta V = 0.000705 \, P^{-0.69} \cdot \delta P \qquad \cdots \cdots \cdots \quad (10)$$

If it is assumed that the change in deviator stress, δq, has negligible effect upon the recoverable elastic energy, δU, then:

$$\frac{dU}{d\varepsilon} = p \cdot \frac{dV}{d\varepsilon} = 0.000705 \, p^{0.31} \cdot \frac{dp}{d\varepsilon} \qquad \cdots \cdots \quad (11)$$

The "elastic energy correction" is calculated using Eq. 11.

DRAINED TRIAXIAL COMPRESSION TESTS

A series of conventional drained triaxial compression tests carried out on samples of sand with different initial void ratios and consolidated under cell pressures of 40, 60 and 80 lb/in² are discussed in this paper. Results of a few drained triaxial compression tests during which p was kept constant at 60 lb/in² are also presented.

The triaxial samples were subjected to a conventional axial strain of about 20%. q was calculated on the assumption that the sample remained a right circular cylinder during the test. It was observed from the experiments that a loose sample generally bulged in the middle at large strains. In a dense sample, failure planes were formed soon after the peak deviator stress was reached. As the failure planes developed, the rate of volume change diminished and became small. It was also noticed that several visible failure planes were initially formed and a single preferred plane then emerged after the peak deviator stress was reached. It should be noted from these observations that:

(i) deformation in the sample was not uniform after the peak value of q was reached,

(ii) the estimated rate of dilatation was not correct after the peak value of q was reached.
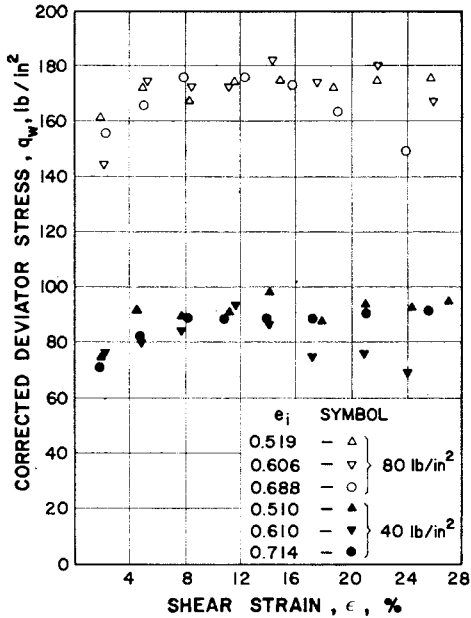
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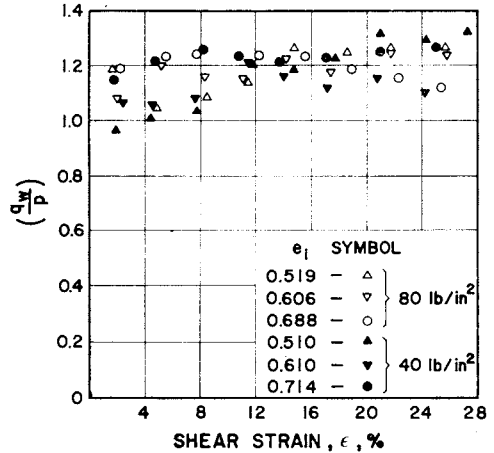Fig. 5. Variation of q_w with shear strain during drained triaxial compression tests.

Fig. 6. Variation of q_w/p with shear strain during drained triaxial compression tests.

Figure 5 shows the variation of q_w with ε for dense, medium dense and loose samples of sand at cell pressures of 40 and 80 lb/in². q_w seems to reach an approximately constant value after about 4% shear strain. The variation of q_w/p with ε for these six tests is shown in Fig. 6. q_w/p is found to be reasonably constant, except for small values of ε, and has a value of about 1.20 for these tests. This value represents the frictional characteristics of the material and is independent of the void ratio and the cell pressure within the pressure range of testing.

It should be noted here that the "elastic energy correction" is large during the initial stages of the test and is negligible close to the peak value of q. For a test with an initial void ratio 0.51 and consolidated under a cell pressure 40 lb/in² the elastic energy correction at 0.2% shear strain is 17.5 lb/in² for a value of $q = 36.2$ lb/in² and 0.4 lb/in² for the peak value of $q = 137.5$ lb/in² at a shear strain of 8.5%. The corresponding values of the "boundary energy correction" for this test are 13.8 lb/in² at 0.2% shear strain and 55 lb/in² at 8.5 shear strain. Hence, the peak strength of sand under drained conditions consists mainly of frictional and dilatational components.

In Fig. 7, the rate of dilatation, $dv/d\varepsilon$, at the peak values of q is plotted against the void ratio, e, for the triaxial compression tests. It is evident that a unique relationship exists between these parameters at failure.
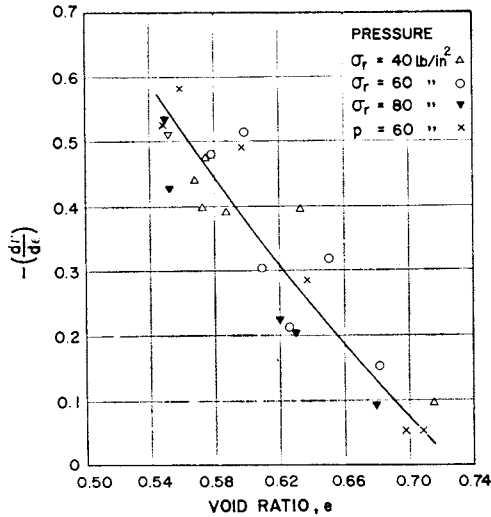
Fig. 7. Dilatation rate at failure for drained triaxial compression tests.

DRAINED TRIAXIAL EXTENSION TESTS

A series of conventional drained triaxial extension tests performed on samples with different initial void ratios consolidated under cell pressures of 40, 60 and 80 lb/in^2 are discussed herein. Results of a few drained triaxial extension tests during which p was kept constant at 60 lb/in^2 are also presented. During the extension tests, a neck was formed in the sample soon after the peak value of the deviator stress was reached and further axial strain produced very little dilatation. The peak deviator stresses for the extension tests were attained at shear strains of about 5 to 10%, whereas shear strains of about 7 to 15% were required for the compression tests.
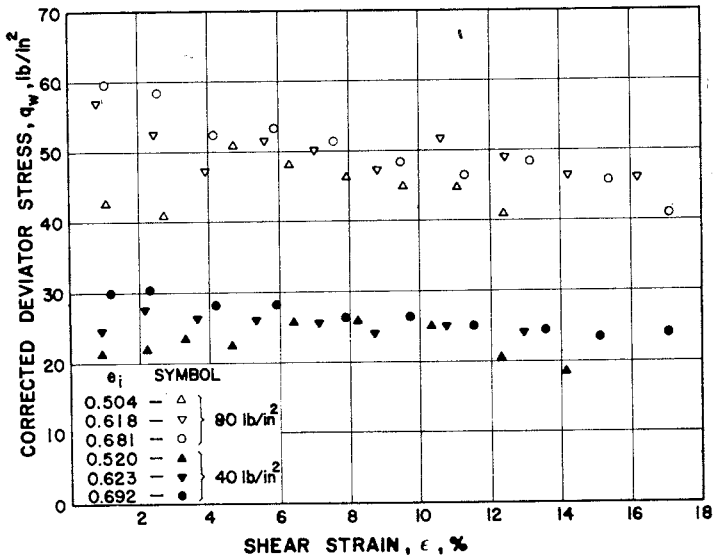

Fig. 8. Variation of q_w with shear strain during drained triaxial extension tests.

Figure 8 shows the variation of q_w with ε for dense, medium dense and loose samples of sand at cell pressures of 40 and 80 lb/in². The variation of q_w/p with ε for these tests is presented in Fig. 9. The value of q_w/p is found to be approximately constant except at small values of ε; it is also not constant at higher values of ε due to the formation of the neck in the sample. q_w/p is found to have a constant value of about 0.85. This value represents the frictional characteristics of the material and is independent of void ratio or consolidation pressure within the pressure range of testing.
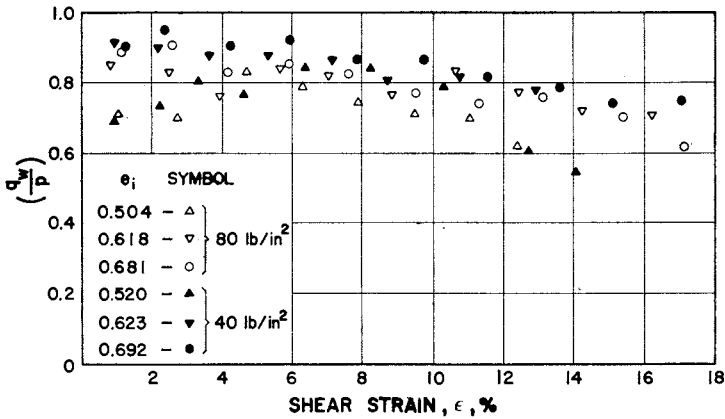
Fig. 9. Variation of q_w/p with shear strain during drained triaxial extension tests.

The rate of dilatation, $dv/d\varepsilon$, corresponding to the peak values of q is plotted against the void ratio, e, in Fig. 10 for the triaxial extension tests. Here again, a unique relationship is found to exist between these parameter at failure.
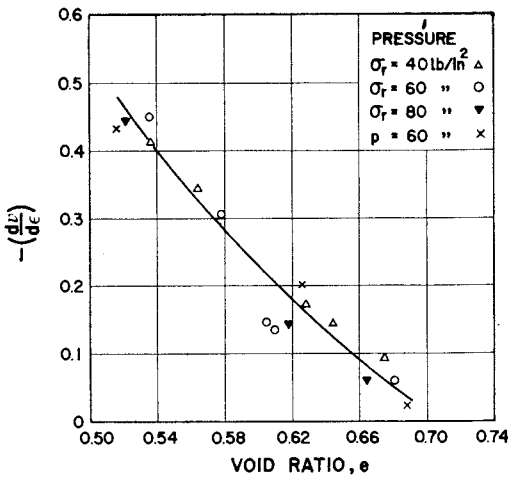
Fig. 10. Dilatation rate at failure for drained triaxial extension tests.
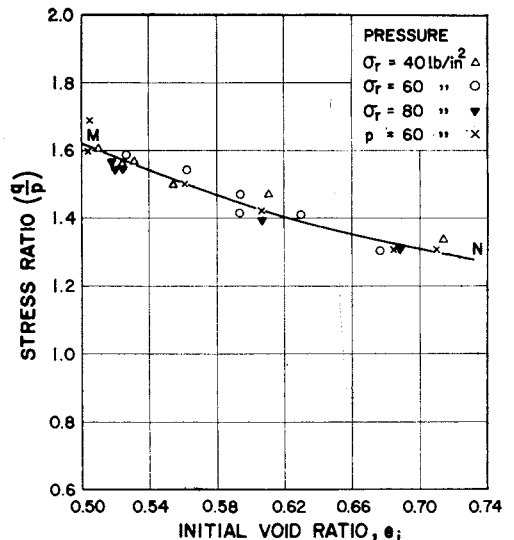
Fig. 11. Peak values of q/p for drained triaxial compression tests.

COMPARISON OF COMPRESSION AND EXTENSION TESTS

The results of drained triaxial compression and extension tests are now compared to study which failure criterion is applicable to sand. The three common classical failure criteria that are applied to soils are the extended von Mises, the extended Tresca and the Mohr-Coulomb criteria. Of these three criteria, the Mohr-Coulomb failure criterion is the only one which assumes that the intermediate principal stress has no influence on strength (ROSCOE et al, 1963).

Let α_c and α_e be the peak values of the stress ratio, q/p, for compression and extension tests respectively at the same void ratio and consolidation pressure. If the extended von Mises or the extended Tresca failure criterion is valid, then:

$$\alpha_e = \alpha_c \qquad \ldots \ldots \ldots \ldots \ldots (12)$$

while if the Mohr-Coulomb failure criterion is valid, then (ROSCOE et al, 1963):

$$\alpha_e = \frac{3\,\alpha_c}{(3 + \alpha_c)} \qquad \ldots \ldots \ldots \ldots (13)$$

The peak values of q/p obtained from the drained triaxial compression tests are plotted against the initial void ratios in Fig. 11 and MN is the mean curve drawn through these points. Figure 12 shows the variation of the peak
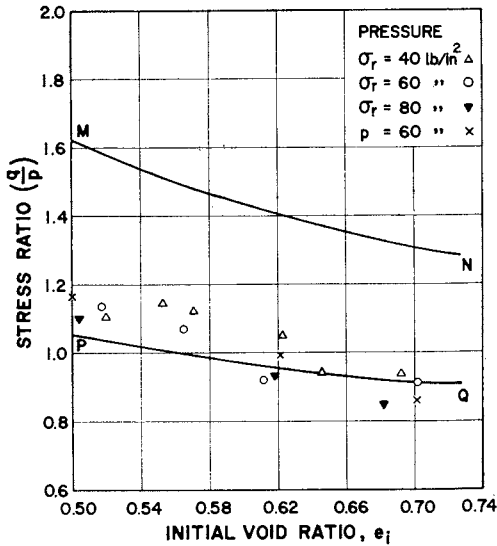

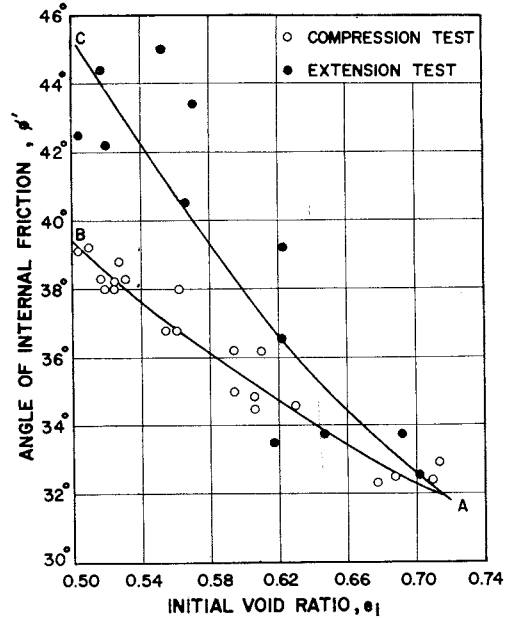Fig. 12. Peak values of q/p for drained triaxial extension tests.

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

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values of q/p with the initial void ratios for the drained triaxial extension tests. The curve MN in this figure is a reproduction of the curve in Fig. 11. The curve PQ in Fig. 12 gives the variation of the peak values of q/p with the initial void ratios for the extension tests as predicted from curve MN obtained for the compression tests, using the expression given in Eq. (13). If the extended von Mises or the extended Tresca criterion is valid for sand, the experimental points for extension tests would lie on the curve MN while, if the Mohr-Coulomb failure criterion is valid, these points would lie on curve PQ. It is evident from Fig. 12 that the extended von Mises and the extended Tresca failure criteria are not applicable to sand and the Mohr-Coulomb criterion describes failure much closer.

The results of triaxial compression and extension tests are again compared in Fig. 13 in terms of the angle of internal friction, ϕ'. The curves AB and AC show the variation of ϕ' with initial void ratio for the compression and extension tests respectively. ϕ' measured in extension is found to be higher than ϕ' measured in compression. In the dense state, ϕ' in extension is about 6° higher than that in compression, and this difference decreases with increase in initial void ratio. In the loose state, ϕ' in extension and compression are equal. GREEN & BISHOP (1969) have also reported that ϕ' measured from drained triaxial (axi-symmetric) extension tests on a dense sand is 2° higher than ϕ' measured from drained triaxial compression tests.

These results show that the intermediate principal stress has some influence on the failure strength. Thus, the widely used Mohr-Coulomb failure criterion underestimates the strength of soils in many stability problems encountered in practice.

The failure strength of sand under drained conditions is mainly composed of the frictional component and the dilatational component. The dilatational component at failure is negligible for loose soils, and the difference in ϕ' between triaxial compression and extension tests is found to be negligible for such states of packing. Thus, the Mohr-Coulomb failure criterion is seen to be valid for very loose sand. As the sand becomes denser, the dilatational component at failure increases. This component depends on the intermediate principal stress and, therefore, on the type of test (e.g. triaxial compression, triaxial extension or plane strain). Thus, for dense sand, ϕ' in compression and extension are found to be unequal.

The triaxial compression tests described earlier show that q_{w}/p has an approximately constant value equal to 1.20. The corresponding value of q_{w}/p for triaxial extension tests that would satisfy the Mohr-Coulomb failure criterion, as determined from Eq. 13, is 0.86. The value of q_{w}/p

obtained from extension tests is 0.85, showing that the frictional component satisfies the Mohr-Coulomb failure criterion and is independent of the intermediate principal stress.

CONCLUSIONS

Consolidation and swelling tests carried out on triaxial samples of sand under isotropic stress conditions showed that the ratio between the volumetric strain and the axial strain lies close to the value 3.0, indicating that the material was isotropic under such stress conditions. Some degree of anisotropy was observed under isotropic swelling stresses. Very little structural rearrangement occurred during the swelling tests and the volume change was presumed to be entirely elastic. The elastic volumetric strain, V, for this sand subjected to an ambient pressure P followed the power law given by Eq. 7, viz:

$$V = 0.00227 \ P^{0.31} \quad \ldots \ldots \ldots \quad (7)$$

For the series of drained triaxial compression and extension tests, the above relationship was used to determine the "elastic energy correction" applied to the deviator stress, q, to obtain the corrected deviator stress, q_w. The stress ratio, q_w/p, was found to remain approximately constant during these tests except at small strains. The compression and extension tests gave constant values of about 1.20 and 0.85 respectively. These constants represent the frictional characteristics of the sand.

Unique relationships were found to exist between the rate of dilatation, $dv/d\varepsilon$, at failure and the void ratio for the triaxial compression and extension tests in the consolidation pressure range of 40 to 80 lb/in^2.

Comparison of the failure points of the triaxial compression and extension tests showed that failure is described much closer by the Mohr-Coulomb criterion than by the extended von Mises or the extended Tresca criterion. The angle of internal friction, ϕ', determined from the extension tests was higher than ϕ' determined from the compression tests for any void ratio. In the dense state, ϕ' in extension was about 6° higher than that in compression, and this difference decreased with increase in void ratio. In the very loose state, ϕ' in extension and compression were equal. Thus, the widely used Mohr-Coulomb failure criterion underestimates the strength of soils in many stability problems met in the field.

It was found that the corrected deviator stress, q_w, at failure, which gives the frictional component of the deviator stress, satisfied the Mohr-Coulomb failure criterion and was independent of the intermediate principal stress.

ACKNOWLEDGEMENTS

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PAVEMENT DESIGN FOR ROADS IN BANGKOK

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SYNOPSIS

An analysis of the subgrade strength for the soil conditions occurring in Bangkok is made from considerations of the soil suction characteristics of clays, the surcharge of the pavement structure and the water table. It is shown that this is not likely to be greater than 1.5% CBR. On this basis comparative designs for flexible and rigid pavements are developed using three widely used design methods for four levels of traffic. The differences between the pavement thicknesses given by the different design methods are discussed. It would appear that unreinforced conerete slabs should be marginally more economical than flexible construction. The machinery required is simpler and the construction lends itself to labour-intensive methods. An experiment is recommended to compare the comparative designs.

INTRODUCTION

With the increasing traffic in the city area of Bangkok, a number of roads are being reconstructed and new ones are being built. Much of the work is carried out by filling in the canals and widening existing roads over the filled canals. In order to ensure the best value for money spent on these roads, it is necessary to make a realistic approach to the design of the pavement structure. Over the last two decades a lot of work has been carried out in many parts of the world on the design of pavements, but the majority of it has been undertaken in the temperate zone. Although most of the results are applicable to the tropics, some modifications should be made to take into account the different climatic and soil conditions. The purpose of this paper is to consider the possible alternative designs for the pavements of roads in Bangkok.

GENERAL TRAFFIC AND PAVEMENT CONDITIONS

The Greater Bangkok Metropolis has a population approaching 3.5 million and covers an area of over 300 km². It is situated in the flood plain and on the banks of the Chao Phraya river, some 14 km from the sea. The ground level is approximately 1 m above mean sea level and flooding occurs at times of heavy rain and at periods of high river flow when the waters are backed up by high spring tides between October and December.

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Bangkok is the principal port of the country through which imports and exports pass, and an increasing percentage of the traffic is handled by road transport. The major road development has occurred over the last two decades. Existing roads have been widened by piping and filling in the canals. The traffic is heavy on the main roads, many of which have six or more lanes and are running to capacity. The main roads disect the area into relatively large blocks, and there is a dearth of interconnecting secondary roads.

The main commercial load-carrying vehicles are the ten-wheel, three-axle and six-wheel, two-axle vehicles. Rear axle loads on these vehicles vary from 2000 kg (4500 lb) unladen to 13,000 kg (28,000 lb) laden, while the front axle loads probably do not exceed 4000 kg (9000 lb) laden. The main heavy buses are six-wheel, two-axle vehicles which may carry up to 120 passengers each in the rush hour. Since the average weight of an Asian is about 55 kg (120 lb), this loading results in a maximum rear axle weight of about 6000 kg (13,500 lb). Lorries carrying timber to the docks and saw mills, and concrete piles used in building, are truck-trailer combinations, and axle loads in the case of timber lorries are sometimes in excess of 13,500 kg (30,000 lb).

Although they do not occur in large numbers, the larger vehicles which transport goods on the main roads in western countries have begun to make their appearance in the last year or two. These are the twelve-wheel, four-axle trucks and the eighteen-wheel, five-axle truck-trailer combinations which are used for special purposes in Bangkok. The individual axle loads of these rarely exceed 8,000 kg (18,000 lb) when laden.

The traffic growth over the last few years has been fairly rapid, sometimes exceeding 10% per annum. In the absence of reliable trends, it is difficult to predict future growth. Government policy could be a major factor in this. As far as the more heavily trafficked roads are concerned, the increase in traffic will necessitate more traffic lanes or an alternative facility. Since the design of the pavement structure for such roads is based on the slow lane traffic, the rate of traffic increase for structural design purposes will probably not exceed 5%, since this lane will be running near capacity. The effect of this will be considered later.

The soil conditions of the area comprise a crust of weathered clay, overlying some 10 to 20 m of soft clay which, in turn, overlies about 10 m of stiff clay. Below this there are alternate layers of sand, sandy gravel and clay down to about 500 m (MUKTABHANT et al, 1967).

The early roads in Bangkok were built on the banks formed by excavating the clay from the canals which were used for transport, irrigation, etc. Initially, the roads were made of lateritic gravel-sand-clay, burnt brick and

block stone, with later development into water-bound and bitumen-grouted Macadam. In recent years (from about the early 1950's), many roads have been widened by piping the canals, where the soft clay is frequently exposed, and filling with sand. Some of the widened roads were made with traditional forms of construction using "laterite", grouted stone and surface treatments with little attempt at design. In the last few years, new construction outside Bangkok has employed graded crushed stone bases with 5 to 10 cm of pre-mixed bituminous surfacing using 80 to 100 pen. bitumen, 30-35% passing U.S. No. 8 sieve and no added filler.

Most of the main roads constructed within the metropolis in the last two decades have been concrete. Generally, slabs are 20 cm thick (with 5 cm edge thickening) and 6 m long, with 0.9 cm diameter steel at 20 to 25 cm spacing transversely and 25 to 30 cm spacing longitudinally. Slab lengths up to 18 m are used on some main roads radiating from Bangkok. Transverse joints are doweled with 1.2 or 1.9 cm diameter bars, 50 cm long, at 25 cm spacing for both expansion and contraction joints. Not enough care is taken to ensure that dowels are properly aligned or, in the case of expansion joints, that concrete does not bridge the gap. Longitudinal joints are tongued and groved, with 1.2 cm diameter tie rods, 50 cm long, at 50 cm spacing. The slabs are poured on a layer of sand 3 cm thick, and no concreting paper is used. There is seldom any effective drainage from the underside of the slab.

Both concrete and flexible construction are used on minor roads. Because of the general low-lying situation of Bangkok, many of the roads flood. Some of those not yet reconstructed are underwater for periods of days at a time. Trenches made by statutory authorities are fairly frequent, even in new roads. Insufficient care is taken to shore the sides, and back filling is not adequately compacted. This leads to pavement failures. The pipe rings used in piping the canals are inadequately jointed. This leads to sand being washed into the drains and, finally, the collapse of the pavement. The use of clay for back filling would probably prevent this to a large extent since it would prevent the flow of water through the ground.

FACTORS AFFECTING ROAD PAVEMENT DESIGN

The design of a road pavement depends on a number of factors of which the following are the most important:
  (a) Traffic
  (b) Subgrade strength
  (c) Materials available
  (d) Type of pavement

These factors will be considered in turn with respect to road pavement design in Thailand in general and Bangkok in particular.

Traffic

Developments over the last two decades have shown that both the numbers and weights of axles which are expected to pass over a road during its design life must be taken into account when designing the pavement structure. Individual axle loads are converted into an equivalent number of repetitions of a standard axle load. The design is then based on the number of standard axle loads anticipated during the life of the road. A standard axle load is usually taken as 18,000 lb (8180 kg) or, in one common pavement design method (SHELL, 1963), as 10,000 kg (22,000 lb); an 18,000 lb load is equivalent to 0.42 repetions of a 10,000 kg load (SHOOK et al, 1963) and this figure is used to give equivalent traffic levels throughout this paper.

There is very little data on the distribution of axle loads of vehicles using the streets of Bangkok. Before an engineer can attempt to design a pavement, an attempt must be made to determine not only the frequency but the spectrum of axle loads which will use the facility. The difficulty of obtaining a random sample for roads in Bangkok is complicated by the fact that there is a ban on the use of ten-wheel trucks from 0600 to 2100 hrs. However, if traffic is considered in terms of repetitions of standard axle loads instead of time, the engineer can make a legitimate design. The actual life will depend on the time over which the number of design axle loads is reached. If the rate of increase of traffic is faster than anticipated, the life will be shortened. If the spectrum of axle weights shifts to give a higher mean axle load than anticipated, the life will also be shortened. The engineer should pick a level of repetitions suitable to the economic position of the country, the present level of traffic, and the possibilities of stage construction.

On multi-lane roads, the pavement is usually designed for the traffic in the slow lane. This does lead to some over design in the fast lane, and it is possible to use a tapering construction. The construction difficulties, however, often exceed any potential saving.

While it is possible to consider the design of a pavement for any number of repetitions of a standard wheel load, it is usual to consider levels from 10^4 repetitions increasing by multiples of 10. A main road which warrants the construction of two lanes in each direction will probably carry at least 2,500 commercial vehicles (i.e. all vehicles over 1,500 kg unladen weight) on the slow lane per day. If a composition of 60% commercial traffic is assumed for the slow lane, the total traffic in that lane will be 4170 vehicles. After allowance for the commercial vehicles in equivalent passenger car units, this

gives the equivalent of about 6,000 passenger car units per day. At a 5% growth rate, this means that the flow will be 15,900 p.c.u./day, which is well over saturation. What would probably occur is that more of the heavy vehicles would move into the fast lane.

Without the data on axle weights, it is only possible to estimate the equivalent 18,000 lb wheel loads for traffic in Bangkok. The ROAD RESEARCH LABORATORY (1970) give equivalence factors for commercial vehicles on different classes of road in the U.K. ranging from 0.45 to 1.08. It is probable that the figure for Bangkok will be between these limits. The least number of repetitions of an 18,000 lb axle load for a ten year period with a rate of increase of 5% will therefore be 5.4×10^6, while the maximum number for a twenty year period with a rate of increase of 10% will be 1.13×10^8, based on the lower and higher equivalence factors respectively. It is reasonable, therefore, to suppose that the slow lane of a multi-lane main road should be designed for somewhere between 5×10^6 and 10^8 repetitions of an 18,000 lb wheel load. At the lower level of traffic, if stage construction were contemplated in which 5 cm of surfacing were to be left off, the delay in adding the 5 cm would only be two years. This would suggest that stage construction would hardly be worthwhile. At the higher level (10^8 repetitions), the 5 cm could be left off for between eight and ten years depending on the rate of increase of traffic.

For heavily trafficked roads, like the main roads in Bangkok, private cars and light pickups can be discounted for pavement design purposes, and the design can be based on commercial vehicles only (i.e. vehicles over 1,500 kg unladen weight). Even though little is known about the distribution of axle loads in Bangkok, it is probable that the slow lane of a main street should be designed for 10^7 repetitions of an 18,000 lb axle load, or 4.2×10^6 repetitions of a 10,000 kg axle load for an anticipated life of 20 years; at a 5% annual growth rate, this would allow for a present day traffic of some 5,000 commercial vehicles per day in one direction. Roads to the docks and approaches to river crossings should perhaps be designed for ten times this traffic, while some connecting roads, which do not carry such a heavy volume of traffic, could be designed for a tenth of this traffic.

In this paper structural designs will be considered for roads carrying 10^5, 10^6, 10^7 and 10^8 repetitions of an 18,000 lb axle load.

Subgrade Strength

When considering the subgrade strength in Bangkok, one must consider the soil profile of the upper layers of the Bangkok Clay. Because there is a weathered crust overlying the thick deposit of Soft Bangkok Clay (MUKTA-

BHANT et al, 1967), roads are founded on the Weathered Clay or on fill (which may be clay dried out to around the plastic limit) where a canal has been filled in. If the Soft Clay has a moisture content of about 80% of the liquid limit, use of the relationship given by MUKTABHANT et al (1967) between plasticity index and liquid limit (PI = 0.74 LL − 9.21) together with the method of BLACK (1962) for the calculation of soil strength gives a CBR value of 0.1%.

If it is assumed that under the highest water table conditions the water table is just at the surface of the road, and that the pavement layers consist of 0.2 m with a density of 2.4 ton/m^3, 0.50 m with a density of 2.1 ton/m^3 and 0.30 m with a density of 1.7 ton/m^3 (clay dried to about the plastic limit), the extra loading (assuming that there is no crust over the soft clay) will be equivalent to 1.04 m of water. On the basis of Black's method, the final moisture content of the soil below these layers would be about 7% above the plastic limit, and the CBR would be about 3%. The change in moisture content between the above two conditions would be about 28%, and this would occur as a result of the settlement that would take place when the Soft Clay was fully consolidated under the pavement loading. In fact, this would take some time to achieve, and the subgrade strength would be lower than that indicated by a 3% CBR for sometime.

If the subgrade clay under the example pavement were dried out to its plastic limit and compacted to 5% air voids or less, Black's method shows the CBR at this moisture content would be about 12%. However, if only 0.7 m of pavement were used over the dried clay, and if the water table were at the road surface, the surcharge would be only 0.83 m of water. Under these conditions, the clay would eventually absorb water until its CBR was down to about 1.5%. If the highest water table could be kept 0.5 m below the road surface, then the surcharge would be equivalent to 1.33 m of water and the CBR of the clay would be just less than 2%. If the highest water table was kept 1 m below the road surface, the surcharge plus suction at the surface of the compacted clay layer would be 1.83 m of water, and the maximum CBR would be about 2.4%.

It would appear, therefore, that the maximum CBR which could be considered for the subgrade clay layer would be 2% in any design method. In fact, it would probably be better to use 1.5% CBR to take account of site conditions. This does not mean that there would be no advantage in using clay that had been dried out to about the plastic limit for the bottom layers of fill where, for example, a canal had to be filled in. The partially dried and compacted clay would provide a better bed on which to compact the subsequent layers. The swelling of this clay would partially offset the settlement

due to consolidation of the deeper layers and would slow down movement of moisture from the sides caused by a fluctuating water table.

Materials Available

The structural layers of the pavement must be placed and compacted over the clay. It is important that, immediately over the clay, the first layer of sub-base should be impermeable and should have as few free voids as possible in which water could pond on top of the clay surface. Where there is old road construction in which clay has worked up into stone boulders, this will effectively have little more strength than the clay. Where the old layers are dense crushed stone which has become impregnated with clay, this can be taken as a sub-base with a strength of about 20% CBR. Of the materials available for new construction, the clayey lateritic gravel-sand-clay would be preferable to sand, and both could be expected to give a CBR of the order of 20%. The sand, being highly permeable, would allow free water to be directly in contact with the clay, and this would reduce the effective strength of the clay.

For the base and surfacing layers, crushed stone, bituminous-bound materials or Portland cement-bound materials can be considered in various combinations.

Type of Pavement

There are two main types of pavement which will be considered here, flexible and rigid, both of which have sub-divisions.

Flexible pavements comprise layers of materials which essentially have little or no tensile strength or resistance to bending. The traffic load is transmitted and spread through the pavement layers to the subgrade. In the tropics, where the bituminous surfacing does not exceed a thickness of 5 or 7.5 cm, this can be considered to be the case since, during the hot part of the year, the thin bituminous surface has little flexural strength. Where greater thicknesses of bitumen-bound materials are used, the lower layers do not reach such a high temperature even at midday, and the bound layer has flexural strength which reduces the vertical loads transmitted to the lower layers; it also has sufficient tensile strength to resist cracking, provided deformations are kept within reasonable limits. This is the basis on which "deep strength" and "full depth" asphalt pavements are constructed. Whether or not "full depth" asphaltic concrete can be used depends on costs and on whether the subgrade has sufficient strength against which the lower layers of asphalt can be compacted. As stated above, it is unlikely that the subgrade

strength would be more than 3% CBR in Bangkok, and this would mean that a 10 cm (4 in.) layer of asphaltic concrete would be required as a working platform, which would be in addition to the required depth of asphaltic concrete. It would almost certainly be cheaper to provide the working platform from the lateritic gravel-sand-clay. In this paper, for the design of a flexible pavement, various depths of bituminous-bound materials in combination with crushed stone bases will be considered over the depth of lateritic gravel-sand-clay (or similar material) required over the Bangkok Clay to give an adequate bearing surface for construction traffic.

Rigid pavements refer to Portland cement concrete which may be reinforced or unreinforced. Modern concrete road design requires a base to provide a reasonable working platform on which construction traffic can run, to provide a smooth surface on which the slabs can be laid and compacted, and not to 'pump' under slight movements of the joints.

A 25 cm thick layer of lateritic gravel-sand-clay could be expected to provide a base with a modulus of subgrade reaction, k, of the order of 200 to 250 lb/in^3 for a deflection of 0.2 in. after ten repetitions on a 12 in. diameter plate, though a value equivalent to 100 lb/in^3 has been assumed for the designs given later in this paper. The admixture of 2 to 3% of Portland cement by weight into the top 10 to 12 cm (4 to 5 in.) would prevent any pumping taking place, as well as giving a slightly better base. The stabilization of this layer would very largely make the operation independent of weather. In place of this stabilized layer it might be cheaper to use a 10 cm (4 in.) layer of compacted, dense, graded, crushed stone.

The design of a rigid pavement will be considered here for both reinforced and unreinforced slabs. It will be assumed that limestone aggregate will be used since this is readily available and it allows greater spacing of joints.

PAVEMENT DESIGN

There are various methods of pavement design which are advocated by different authorities. For flexible pavements, the most frequently used methods of design are given in publications by the ASPHALT INSTITUTE (1970) and SHELL (1963). These have been prepared by the asphalt industry and take into account the results of various full-scale experiments and theoretical analysis. The ROAD RESEARCH LABORATORY (1970) suggests a slightly different design approach and covers both flexible and rigid pavements; the design given for rigid pavements incorporates the latest thinking of the concrete industry.

To cover the traffic conditions likely to occur in Bangkok, designs will be given here for four levels of traffic in terms of repetitions of 18,000 lb (8165 kg) axle loads; this is satisfactory for comparative purposes. The subgrade strength will be taken as equivalent to a CBR of 1.5%, which is a value that could be expected to be achieved on an actual job where the water table was above subgrade level for about six months of the year. This strength is below the minimum figure considered in many designs, and either a nominal extra thickness of sub-base is added or an attempt is made to extrapolate the design curves to cover this. If there are soft spots, a greater thickness of sub-base would be required.

To cover the different materials available, a number of different designs will be given for both flexible and rigid pavements.

Flexible Pavement Design

The thickness of layers for the design of the pavement structure is given in Tables 1, 2 and 3 for the Asphalt Institute, Shell and Road Research Laboratory design methods respectively for the four traffic levels of 10^5, 10^6, 10^7 and 10^8 repetitions of an 18,000 lb (8165 kg) axle load. In the Asphalt Institute method, the traffic is expressed in terms of a *Design Traffic Number* (DTN) for a twenty year life. The equivalent values for the four levels of traffic are 1.37×10, 1.37×10^2, 1.37×10^3 and 1.37×10^4. The highest level requires extrapolation of the scale. Traffic in the Shell method is expressed in terms of 10,000 kg axle loads. The equivalent levels are 4.2×10^4, 4.2×10^5, 4.2×10^6 and 4.2×10^7 repetitions.

The basic design thickness in the case of the Asphalt Institute Method is given in terms of the thickness of full depth asphaltic concrete using material equivalent to an ASPHALT INSTITUTE (1969) type IV mix (i.e. a mix designed to Marshall asphalt standards with 35 to 50% passing a U.S. No. 8 sieve). The scale is extrapolated to allow for a subgrade strength of 1.5% CBR, the lowest strength given being 2% CBR. This method is based on the results of full-scale experiments in the U.S.A. and, while it is probable that similar results would apply in Thailand, it has not yet been proven that the effects of temperature variations throughout the year on the load distribution properties of asphaltic concrete will be the same in Thailand as in the U.S.A. It is also questionable whether such materials could be laid and compacted directly on subgrades with strengths as low as 1.5% CBR.

The alternative designs given in Table 1 are derived from the basic design thicknesses using equivalency factors (1 in. of asphaltic concrete is equivalent to 1.3 in. of sand asphalt, 1.4 in. of other bitumen-bound materials, 2 in. of high quality unbound material or 2.7 in. of low quality unbound material).

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Minimum thicknesses of asphaltic concrete are given for different traffic levels.

Table 1. Flexible pavement designs for Bangkok based on the ASPHALT INSTITUTE (1970) method for a subgrade strength of 1.5% CBR

DTN		1.37×10	1.37×10^2	1.37×10^3	1.37×10^4
Thickness of asphaltic concrete,	cm	29.2	38.2	48.3	59.7
Minimum cover over high quality unbound material,	cm	11.5	14.0	16.5	19.0
Minimum cover over low quality unbound material,	cm	17.8	22.8	26.7	30.5
Alternative Designs					
1. Asphaltic concrete,	cm	11.5	14.0	16.5	19.0
High quality bound material,	cm	35.5	48.2	63.5	81.5
2. Asphaltic concrete,	cm	11.5	14.0	16.5	19.0
High quality bound material,	cm	12.7	17.8	20.3	22.8
Low quality bound material,	cm	34.3	44.5	62.2	89.0
3. Asphaltic concrete,	cm	18.0	22.8	26.7	30.5
Low quality bound material,	cm	30.5	40.5	58.5	79.0

Table 2. Flexible pavement designs for Bangkok based on SHELL (1963) design charts and a subgrade strength of 1.5% CBR

No. of 10,000 kg axles		4.2×10^4	4.2×10^5	4.2×10^6	4.2×10^7
Thickness of dense asphaltic concrete, cm		22	28	35.5	42.5
Alternative Designs:					
1. Sub-base (20% CBR) for 3% CBR subgrade,	cm	22.5	23	25	26
Extra for 1.5% CBR subgrade,	cm	25	26	27.5	30
Total sub-base,	cm	47.5	49	52.5	56
Base crushed stone (80% CBR),	cm	20	29	32.5	41.5
Surfacing,	cm	5	7.5	13	18
2. Sub-base (20% CBR) for 3% CBR subgrade,	cm	23	25	27	29
Extra for 1.5% CBR subgrade,	cm	25	26	27.5	30
Total sub-base,	cm	48	51	54.5	59
Bituminous-bound base and surfacing,	cm	12.5	17.5	25	33.5

Table 3. Flexible pavement designs for Bangkok based on Road Note No. 29 of the ROAD RESEARCH LABORATORY (1970) and a subgrade strength of 1.5% CBR

No. of 18,000 lb axles		10^5	10^6	10^7	10^8
Sub-base (20% CBR) for 2% CBR subgrade,	cm	38	44	52	62
Extra for 1.5% CBR subgrade,	cm	15	15	15	15
Total sub-base,	cm	53	59	67	77
Dense bitumen Macadam base,	cm	6.8	9.2	14	27
Surfacing (base course and wearing course),	cm	5.2	6.6	9.8	10
OR *over same sub-base*:					
Rolled asphalt base,	cm	6.5	8.2	12	21.5
Surfacing (base course and wearing course),	cm	5.2	6.6	9.8	10
OR *over same sub-base*:					
Crushed stone base (80% CBR),	cm	11.5	15	22	26
Surfacing (base and wearing course), (Thickness in excess of 10 cm may be bitumen-bound base)	cm	5	6.7	11.5	21.5

The Shell design charts are based largely on theoretical analyses of multilayer systems in conjunction with observations from full-scale experiments. The multilayer analysis shows that, for a given deformation, the critical condition is either the tensile stress at the bottom of the bound layer or the vertical stress at the subgrade. The minimum subgrade strength considered is a dynamic modulus of 320 kg/cm^2 (approximately 3% CBR), and the curves have been extrapolated to cover a subgrade strength of 1.5% CBR. The curves have been interpolated to cover the different traffic levels.

The Road Research Laboratory design curves require an additional thickness of 15 cm of sub-base for subgrades with a strength of less than 2% CBR. They are based mainly on the results of full-scale road experiments modified to take account of theoretical analyses. They make greater distinctions in the quality of the bitumen-bound materials.

Comments on the Flexible Pavement Designs

The Shell design charts give consistently thinner designs for the full depth asphaltic concrete pavement than those given by the Asphaltic Institute method. In both cases, the thicknesses have been extrapolated to meet the very weak subgrade conditions. The difference in thickness varies from 7 cm for the lowest traffic level to 17 cm for the highest traffic level considered. For a

Table 4. Thickness of asphaltic concrete required for subgrade strength of 3% CBR

No. of 18000 lb axles	10^5	10^6	10^7	10^8
	Thickness of asphaltic concrete, cm.			
Shell Design Chart	18	25	32.5	41
Asphalt Institute	21.5	28	35.5	43.5

subgrade CBR value of 3%, which is covered by both design methods, the required thicknesses of asphaltic concrete are as given in Table 4. The differences here vary from 2.5 cm for the highest traffic level to 3.5 cm for the lowest level. Although this is large for a high quality material like rolled asphalt, it almost certainly arises from the difficulty of assessing the exact thicknesses required when spreading and compacting high quality materials on a weak foundation.

When pavements comprising asphaltic concrete and crushed stone or gravel bases are considered, the differences in total thickness given by the three design methods for any given thickness of bituminous bound material is not so very large and result from different thicknesses of the lowest cost material. Here again, the difficulty of working to exact thicknesses when spreading layers of material on very weak foundations would mask the differences given by the different design methods.

All the design methods state clearly that the asphaltic concrete should be a high quality, dense material. This means a material with over 35% passing a U.S. No. 8 sieve and at least 2% of mineral filler, such as Portland cement or hydrated lime or their equivalent. This type of material is seldom used in Thailand where the asphaltic concretes usually have about 30% passing a No. 8 sieve and only crusher dust is used as mineral filler. Such materials tend to crack at a lower number of traffic load repetitions than those envisaged in the design charts used to compile Tables 1, 2 and 3.

Deflection Measurements and Pavement Design

The deflection beam, which was first developed by A.C. Benkleman to assess the performance of the WASHO road test sections (HIGHWAY RE-SEARCH BOARD, 1953), has considerable use as a design tool. The beam measures the vertical deflection in the road structure during the passage of a wheel load. The deflection varies with road temperature. Correction factors developed for different types of pavement structure in the northern temperate zone are usually used to correct readings to a standard temperature of 20 to 21°C (68 to 70°F) as measured 4 cm (1.5 in.) below the road surface. Some

authorities (e.g. LISTER, 1960) recommend that readings should not be made above 35°C (95°F). Also, deflections vary with the time of year, reflecting mainly changes in subgrade conditions. In the northern temperate zone, measurements are usually made during the spring and autumn months with the spring conditions being regarded as critical.

Correlation of deflection measurements on different types of pavement whose structures are accurately known, and where the weights and numbers of axle loads are also known, has shown that there is a relation between deflection and number of repetitions of standard axle loads at failure. The relations vary somewhat for different types of pavement structure. Thus, from a knowledge of the structure of the pavement, the present deflections and the repetitions of traffic already applied, it is possible to predict the remaining life of the road.

The beam can also be used in the control of construction as a nondestructive test for the pavement structure. It can be used on any layer which will not give a deflection of more than 0.1 in. It can, therefore, be used to indicate the thickness required above a certain layer to reduce the deflection to any given level which may be required for a pavement to give a certain life. With certain types of base, crushed stone in particular, the deflection usually decreases after the first two months of traffic, and it is this reduced figure which is operative in fixing the life of the pavement. The deflection beam is also a very valuable tool in the maintenance of roads, where it can give accurate information on the thickness of overlay required to extend the residual life of a road (LISTER, 1972).

While it is probable that the deflection beam will be of great use in tropical areas, as far as the author knows insufficient correlations have been made between the performance of pavements of known structural design under known traffic conditions and their deflections to cover the differences from temperate zones which occur in the tropics. The temperatures in the top 1.5 in. of a bituminous surfaced road in many tropical areas rarely fall below 30°C (86°F) and, for most of the year, are operating at temperatures above 35°C (95°F) (BULMAN, 1972; WONG, 1971). BULMAN & SMITH (1972) also showed that the temperature-deflection relations in West Malaysia were markedly different for different pavement structures and that there appeared to be a discontinuity at about 30°C. They were also different from those reported by LISTER (1960) and by the CANADIAN GOOD ROADS ASSOCIATION (1965). There is no published information showing how deflections vary for different types of structure throughout the year. In the Bangkok area, such variations might occur because of changes in the water table or changes in the mean temperature of the top few inches of the road structure which covers the

depth of the bituminous material. It has been shown (LISTER, 1972a) that the number of applications of heavy wheel loads to cause failure of a bituminous surfaced pavement decreases markedly as temperature increases. The period of time during the year when pavement surfaces in Bangkok are at elevated temperatures is considerably longer than it is in the more temperate climates of the northern hemisphere since there is no really cool season. For these reasons, there is a need to correlate the deflection measurements of different road structures in the Bangkok area before this method can be applied with confidence to design.

Rigid Pavement Design

The designs for unreinforced and reinforced concrete pavements in Bangkok are given in Table 5 for a subgrade strength of 1.5% CBR. These are based on mixes using ordinary Portland cement with a minimum strength of 179 kg/cm^2 (2500 lb/in^2) at 7 days and 280 kg/cm^2 (4000 lb/in^2) at 28 days. With the relatively small temperature variations in Bangkok, these designs may be conservative. Expansion joints have been omitted because of the small temperature range and because, provided fixed structures such as bridges and culverts were isolated by a short length of flexible pavement, expansion joints would not be necessary.

The stabilization of the top 10 cm of the lateritic gravel-sand-clay which is normally used as a sub-base is suggested as the most economic way of producing a base which would not 'pump'. The present practice of spreading 3 cm of sand on the base, on which the concrete slab is cast, is not recommended. It is very difficult to compact this layer of sand, and it allows the water to drain out of the concrete mix. It can also lead, with movements of the slab, to stresses set up by dilatance of the sand. Alternatively, the top 10 cm of the lateritic gravel could be replaced with a similar thickness of crushed stone base. If the maximum size of aggregate was limited to 1.9 cm (0.75 in.), there would be no problem of segregation, and the use of a dense mix would prevent water ponding in this layer.

It is recommended that, to prevent drainage of water from the slab into the base during pouring and compaction and to reduce friction between the slab and the base, concreting paper or polythene sheet should be spread over the base before the concrete is poured.

Comments on the Rigid Pavement Designs

In Bangkok, one big advantage in the design of rigid pavements thicker than 16 cm is that expansion joints can be omitted provided that rigid struc-

Table 5. Rigid pavement designs for roads in Bangkok based on a subgrade strength of 1.5% CBR

No. of 18,000 lb Axles		10^5	10^6	10^7	10^8
Sub-base (lateritic gravel sand clay), cm		25	25	25	25

(Top 10 cm to be stabilized with 3% cement or lime to prevent pumping, or to be replaced with dense graded layer of crushed stone).

1. *Unreinforced Concrete*

Slab thickness,	cm	18	19.5	24	31.5

Transverse joints, Contraction joint spacing (for limestone aggregate),	m	6	6	6	6

(3 joints in succession may be warping joints with contraction joints spaced at 24 m. No expansion joints are necessary in Bangkok climatic conditions provided fixed structures are isolated).

Dowels for joints : length,	cm	40	50	60	60
at 30 cm centres : diameter,	cm	1.2	2	2.5	2.5
Warping joints, 1.2 cm × 140 cm rods at,	cm	36	30	24	18

Longitudinal joint tie bars — 1.2 cm diam., 1 m long at 60 cm spacing.

2. *Reinforced Concrete*

Slab thickness,	cm	16	19	24	31.5

Reinforcement (minimum), kg/m²		2	2.5	3.6	5.55
cm²/m width		2	2.7	4.1	6.4

Transverse joints Contraction joint spacing (for limestone aggregate),	m	15	18	26	42

(Spacing may be increased if more than minimum longitudinal reinforcing is used).

Dowels for joints : length,	cm	40	50	60	60
at 30 cm. centres : diameter,	cm	1.2	2.0	2.5	2.5

Longitudinal joint tie bars — 1.2 cm diam., 1 m long at 60 cm spacing.

tures are isolated by a short length of flexible pavement. With unreinforced concrete slabs made of limestone aggregate, contraction joints may be spaced at 24 m with intermediate warping joints at 6 m. There is, therefore, little difference in the spacing of contraction joints for reinforced or unreinforced slabs except for the thickest design.

Contraction joints are a common point of distress in concrete slabs because of poor alignment of the dowel bars. Dowel bars are relatively easy to install with hand methods of construction. When alternate bay construction

is used, the bond to the concrete can be broken by turning the bar 24 hours after casting, thus saving the cost of painting one end of the dowel with bitumen. Warping joints present no difficulty with hand methods of construction and relatively little with machine methods. The main consideration with warping joints is that the bars are long enough to ensure that the bond stress is not exceeded in transfering the stresses between concrete and steel.

The present practice in Bangkok of using very short (6 m) slabs, with longitudinal and transverse steel almost equally spaced would seem to be wasteful of steel. The dowel bars between slabs are frequently misaligned so that they do not perform properly as dowels but are not long enough to act as tie bars. Distress at joints is frequently due to this cause.

One of the arguments put forward for using reinforced concrete slabs is that the reinforcement prevents cracking over a non-uniform subgrade. With the weight of reinforcement normally used in reinforced concrete slabs, it is unlikely that it will prevent cracking due to this cause. It would be better engineering practice to pay more attention to the control of subgrade construction and possibly use a slightly thicker sub-base layer than to try to prevent this cracking with reinforcement.

Comparison of Reinforced and Unreinforced Concrete Pavements

The thickness of slab required for traffic levels over 10^6 repetitions of an 18,000 lb axle load is the same for both reinforced and unreinforced concrete slabs (Table 5). With unreinforced concrete slabs, there would be a joint (contraction or warping) every 6 m. Within the metropolitan area of Bangkok, where traffic speeds should not be high, this frequency of joint spacing should not adversely effect riding qualities   If three warping joints were used in succession followed by a contraction joint, there would be little difference in the spacing of contraction joints for levels of traffic of 10^6 and 10^7 repetitions.

The weight of steel required in reinforced slabs, including joints, varies from a minimum of 3.4 kg/m^2 of road surface for 10^6 repetitions to 6.2 kg/m^2 for 10^8 repetitions. In the case of unreinforced concrete slabs, the steel in the warping and contraction joints for these two levels of traffic would be 1.4 and 2.0 kg/m^2 respectively. At the level of traffic of 10^5 repetitions, the reinforced slab could be 2 cm thinner, and a minimum of 2.6 kg/m^2 of steel could be used against 1.2 kg/m^2 of steel in the unreinforced slab.

Pavement quality concrete costs about U.S. $ 25/m^3 in Thailand (based on 1972 rate of exchange of U.S.$1 = 20 baht), and reinforcing steel costs about U.S. 25 cents/kg. At the lowest level of traffic (i.e. 10^5 repetitions) the nominal costs of reinforced concrete and unreinforced concrete pavements (materials

only) are $ 4.65/m^2$ and $ 4.78/m^2$. Thus, nominally, the reinforced slab should be slightly cheaper, although the difference is small and other factors could tip the balance either way.

In the case of the thicker pavements in Table 5, the thickness of concrete is the same for reinforced and unreinforced slabs. The materials for the reinforced concrete would cost about $ 0.50, 0.70$ and $1.05/m^2$, for 10^6, 10^7 and 10^8 repetitions respectively, more than for unreinforced concrete. An unreinforced pavement should definitely be more economical for these levels of traffic. Since steel is not produced in Thailand, this saving would also be a saving in foreign exchange.

COMPARISON OF FLEXIBLE AND RIGID PAVEMENTS

Design Thicknesses

Because of the very weak subgrade conditions in Bangkok, it would probably not be practical to consider "full depth" asphaltic concrete pavements since it would not be possible to compact the lower layers properly. A working platform of some kind would be required, and this should be taken into account when comparing the designs given in this paper.

For the four levels of traffic considered, the thickness of asphaltic concrete given for "full depth" construction is about one-and-a-half times the thickness of the concrete slab. If the asphaltic concrete thickness is chosen to be equal to the depth of the concrete slab, then the thickness of sub-base required is greater than that required under the concrete slab.

One of the main advantages of flexible pavements over rigid pavements is that it is possible to increase the strength of the pavement by adding further thicknesses of bituminous surfacing. If a relatively stiff asphaltic concrete type of mix were to be used in the surfacing (wearing course and base course) and formed part of the design, it would be important that the subsequent layer(s) should be added before the original surface started to crack. If this were not done, much of the structural value of the asphaltic concrete would be lost.

The stage construction approach should be used at the two ends of the traffic scale. For relatively low traffic counts (i.e. less than 10^5 standard axles), when the first surfacings would be comparatively flexible (e.g. multiple seal coats, semi-grout, or open-textured macadam types of surface), traffic could be used to carry out final compaction of the pavement and to find out any weak spots. The final surfacing of a dense asphaltic concrete type could be delayed for several years without the road suffering undue damage. This is because the relatively thick binder films in these surfacings can tolerate more

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movement without fracturing, and because their thickness contribution to the pavement structure would usually be taken as equal to the same thickness of good quality crushed rock.

At the other end of the traffic scale (i.e. designs for 10^7 or 10^8 standard axles), a reduction in the thickness of the surfacing by about 5 cm would reduce the life in repetitions by about one-half. This would mean that, with rates of traffic increase in the order of 5 %, this thickness could be omitted for about ten years. For designs for the order of 10^6 repetitions in which asphaltic concrete is laid as part of the initial surfacing, it would seem dangerous to omit a thickness of 5 cm for a period of longer than two to three years if the original surfacing were not to be cracked.

From Tables 1, 2, 3 and 5, it would seem, therefore, that in the designs for the higher levels of traffic, the thickness of unbound layers would be greater in the case of the flexible pavement when the thickness of concrete and bituminous-bound materials were similar. This would be so even with allowance for any saving in flexible pavement thickness initially by the use of stage construction.

Comparison of Costs of Flexible and Rigid Pavements

Cost comparisons between flexible and rigid pavements is always difficult because the costs are only marginally different. If comparable alternative designs were put out to tender, the lowest tender would depend on factors other than direct material costs. Some of the factors which influence the type of pavement chosen are as follows.

(1) *Cleanliness and colour.* In an urban area, a concrete pavement is easier to keep clean than an asphalt pavement. The lighter colour of a concrete surface means that it does not heat up quite as much as an asphalt surface in a tropical environment. The ligher coloured surface of concrete pavements makes street lighting easier. Against these advantages, the lighter colour of the concrete can result in more glare for the short period when the sun is at a low angle in the morning and evening.

(2) *Pavement levels.* In an urban area where floor levels of property adjacent to the road already exist, the fact that the total thickness of construction for a rigid pavement is less than for a flexible pavement means a saving either in reduced excavation or in reduced work in raising kerbs, etc.

(3) *Reinstatements.* All roads in urban areas are liable to excavations by statutory undertakings for the installation and repair of water mains, cables, etc. Unfortunately, the damage caused by these excavations is not only restricted to the area of the trench but frequently spreads to a very much wider

area as a result of partial collapse of the sides of the trench during excavation and because of incomplete compaction of the back filling of the trench. Both types of pavement are adversely affected by these conditions and neither type has a definite advantage in this respect.

(4) *Material costs.* Materials costs can be considered in relation to total materials used and to the relative costs of bitumen-bound versus Portland cement-bound materials. The lesser thickness of unbound materials used in the rigid pavement design results in a small saving as compared to the flexible design when the thicknesses of the bound materials are the same. In the case of the bound materials, the costs are made up of materials, i.e. mineral aggregates crushed to certain sizes and binder (either Portland cement or asphalt), and processing, including mixing, laying and compacting.

The mineral aggregates used in both cases will be the same and there is no reason why their costs should be different. In the case of the Portland cement concrete, the mineral aggregate comprises 82 to 86% of the mix, while the cement comprises the remainder. In the case of bitumen-bound materials, the mineral aggregate comprises 94 to 97% of the total material, while the asphalt only comprises 3 to 6% (the lower layers will usually have less asphalt while the surfacing will have more). Portland cement costs about U.S.$ 25/ton in Bangkok, while asphalt costs about $50/ton. Asphaltic concrete often contains 2% of Portland cement as mineral filler. Portland cement concrete pavements also include some steel for joints, and reinforcing in the case of reinforced slabs. This steel amounts to about 7 kg/m^3 for unreinforced slabs and 20 kg/m^3 for reinforced slabs. On a straight comparison of materials costs, therefore, bitumen-bound materials should be some $ 3/m^3 cheaper than unreinforced concrete and $6.25/m^3 cheaper than reinforced concrete in the Bangkok area.

Against the lower raw materials costs for the flexible pavement, the processing cost of bitumen-bound material is much higher than that for Portland cement concrete since both the binder and aggregate have to be heated. This involves fuel and plant costs (about 30 to 50 litres of fuel oil is required to dry and heat a cubic metre of bitumen-bound material to the mixing temperature), and this fuel cost alone will very largely balance the difference in the cost of materials.

(5) *Foreign exchange costs.* Both asphalt and cement involve foreign exchange in their manufacture. In the case of asphalt, the crude oil is imported and the asphalt is produced along with petroleum products. Asphalt cannot be regarded as a waste product since, with modern refinery equipment, it is possible for it to be 'cracked' to produce lighter fractions. In the case of Portland cement, the raw materials are obtained locally (clay and limestone)

but these have to be calcined and ground. The fuel used (oil) in the calcining process carries a similar foreign exchange cost to that involved in asphalt. In both cases, the plant used has to be imported but, in the case of the refinery plant, only a small proportion of the cost would be attributable to asphalt. The difference in foreign exchange costs in production of the two binders is probably marginal.

It is a very different situation with the plant required to mix and lay the two materials. In the case of asphaltic concrete, both the asphalt and the aggregate have to be heated to enable them to be mixed. This involves a dryer and heater for the aggregate and, to ensure that temperatures remain within reasonably close limits, the aggregate is roughly proportioned before drying in addition to being screened after drying. In an urban environment, it is necessary to have a dust collection unit. The binder is heated separately in a boiler and circulated through heated pipes. In addition, for high quality asphaltic concrete, mineral filler has to be added to the mix separately, which calls for more sophistication in the batching plant. Even at the mixing temperatures used for asphaltic concrete, the viscosity of the binder is 300 to 500 times that of water, and more power is required to mix bituminous-bound materials than Portland cement concrete. Mixing is achieved in a pug-mill type mixer. Portland cement concrete, on the other hand, uses water as the medium to distribute the cement through the aggregate. The aggregate does not have to be dried and there is no dust problem. Batching can be carried out by simply weighing the aggregate and cement, and by volume batching the water. Because of the low viscous drag, mixing requires relatively low power, the mixing being achieved by the tumbling action of the aggregate. For equivalent outputs, concrete batching and mixing plants are much simpler and cheaper than asphalt mixing plants.

The spreading and compaction of asphaltic concrete has to be completed before the material cools. While the spreading can be accomplished by hand, it is difficult to get it spread fast enough so that it can be compacted to the correct density. It is necessary, therefore, to have a spreading machine in addition to the rollers required for compaction. Portland cement concrete can be adequately spread and compacted by relatively inexpensive hand-operated tools to secure adequate compaction and a level of finish which is satisfactory in an urban environment. Output can be increased by increasing the number of gangs working. Here again, the equipment required for spreading and compacting Portland cement concrete is simpler and less costly than for asphaltic concrete. For the type of road construction in Bangkok, the highly mechanised slipform pavers would not normally be

considered, but these are of the same order of cost as the spreading and compaction equipment required for asphaltic concrete.

(6) *Quality control.* Because of the simpler mixing and spreading processes involved, it is probably easier to control the quality of Portland cement concrete than asphaltic concrete.

(7) *Curing.* While asphaltic concrete can be used by traffic as soon as it cools, Portland cement concrete has to be cured for at least seven days before being opened to traffic.

(8) *Maintenance.* Asphaltic concrete would probably require a seal coat after 10 to 15 years to renew the non-skid surface. The maintenance required for Portland cement concrete would mainly consist of resealing of the joints.

From the above considerations of cost, it would appear to the author that there are not likely to be any highly significant differences in the costs of initial construction for asphaltic concrete and Portland cement concrete pavements designed for traffic levels exceeding 10^6 repetitions of an 18,000 lb wheel load. Under the existing conditions in Bangkok, where there is a plentiful supply of labour and where no asphalt mixing or spreading equipment is manufactured locally, concrete construction would probably be marginally more economical. However, the actual prices bid on tenders by contractors will reflect a number of other items which may change from month to month, such as whether or not he has equipment lying idle. It would be useful, therefore, to lay comparable sections of both types of pavement and to compare their construction costs and performances in practice on a heavily trafficked road in Bangkok.

CONCLUSIONS

Comparable designs for different levels of traffic, based on repetitions of an 18,000 lb axle load, have been studied for the subgrade conditions in Bangkok. It is concluded that:

(1) Unreinforced concrete should be cheaper than reinforced concrete for the heaviest traffic conditions.

(2) For designs for traffic in excess of 10^6 repetitions of an 18,000 lb wheel load, the difference in cost between unreinforced Portland cement concrete and deep strength asphaltic concrete would be marginal.

(3) It would be useful to construct an experiment on a heavily trafficked road in Bangkok with sections of comparable design in the different types of pavement so that their performance could be studied.

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SURFACE AREA DETERMINATION OF CLAYS

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INTRODUCTION

With advances in the field of soil technology, the specific surface area of soils is being widely recognised as one of the fundamental properties necessary for an understanding of the engineering behaviour of soils. It is well-known that clay particles are usually of a size less than two microns and that most clay mineral particles are thin platelets. Consequently, the *surface area-mass* ratio of most clay particles is sufficiently high for the electric forces at the particle surfaces to strongly influence the engineering behaviour of these particles and their aggregations. A knowledge of the specific surface area of a soil is necessary to compute certain basic properties, such as charge density and particle spacing, which in turn help the prediction of some engineering properties. Swelling and shrinkage due to seasonal change in water content depend on the surface area of soils since this determines the water retention capacity. The surface area has also been correlated with other physical properties of soils, viz. Atterberg limits and base exchange capacity(e.g. Farrar & Coleman, 1967; Mortland, 1954; Van Amerongen, 1967), the mineralogy of some clays (Farrar, 1963) and soil structure (Coleman et al, 1964). Specific surface area may also provide a more convenient basis than mechanical analysis for specifying the texture of soils.

Several methods have been proposed by various investigators for determining the specific surface area of soils. Most of the methods require elaborate and delicate equipment and sophisticated techniques to measure the specific surface area and, therefore, a simple method which would give results of reasonable accuracy is greatly needed. Such a simple procedure for determining the specific surface area of a soil is presented in this paper.

METHODS OF DETERMINING SURFACE AREA

The fundamental principle involved in the measurement of surface area is that when a soil sample is maintained in equilibrium with an atmosphere of water vapour, or any other vapour, the vapour molecules are adsorbed on the

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surface of the soil particles. With a knowledge of the amount of vapour adsorbed at various partial pressures, the surface area of the soil may be calculated. The nitrogen and water vapour adsorption methods (BRUNAUER, 1945; HENDRICKS et al, 1940; KEENAN et al, 1951; ORCHISTON, 1953), the glycol retention method (DYAL & HENDRICKS, 1950; VAN AMERONGEN, 1967), and the glycerol retention method (DIAMOND & KINTER, 1958; MOORE & DIXON, 1970) are some of the methods employed for determining the surface area of soils. COLEMAN & FARRAR (1956) have given an excellent collection of results obtained by various adsorbates both polar and nonpolar in nature. Of these methods, the glycol and glycerol retention methods, though simple in nature, are based on some simplifying assumptions and, as such, may not always lead to accurate results (VAN OLPHEN, 1970). Nitrogen and water adsorption methods are used in this study because of their wide acceptance (e.g. QUIRK, 1955).

BRUNAUER, EMMETT & TELLER (1938) proposed a theoretical equation (*BET* equation) for adsorption isotherms. From this, the amount necessary to form a monolayer, and hence the specific surface area of the adsorbate, can be obtained. The surface area values obtained by them for a variety of adsorbents by means of the *BET* equation agreed in all cases within a few per cent with the surface areas determined by direct visual means (e.g. by electron microscope) where such determinations were possible. Thus, at present this method is regarded as the most reliable one for the determination of surface areas of finely divided substances and, largely because of this, the *BET* equation is used most widely in the field of adsorption (BRUNAUER, 1966).

NITROGEN AND WATER ADSORPTION METHODS

The nitrogen and water vapour adsorption methods using the *BET* equation require elaborate and delicate equipment and sophisticated techniques. The apparatus used and the procedures adopted are adequately described in any book on surface chemistry (e.g. BRUNAUER, 1945; GREGG & SING, 1967).

Figure 1 compares the results obtained by the nitrogen and water adsorption methods. Apart from the authors' results, it includes those of FARRAR & COLEMAN (1967) and the results collected by QUIRK (1955). The six soils used by the authors are of widely varying plasticity characteristics, the plasticity index varying from 20 to 244. In the surface area measurements, outgassing was performed at 250°C at 10^{-5} mm Hg for three hours. The water vapour adsorption isotherms were obtained at 25°C and the nitrogen adsorption isotherms at −184°C.
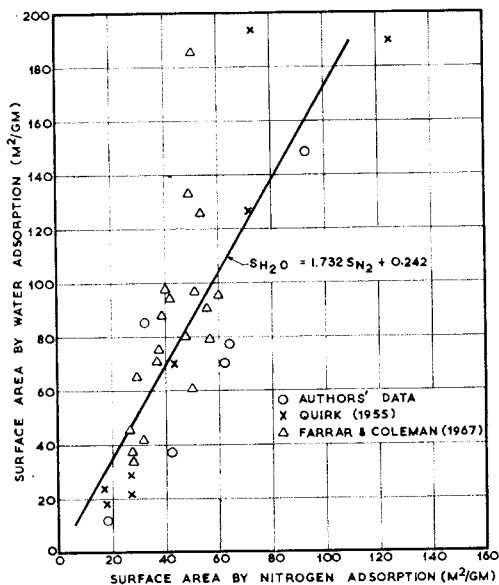
Fig. 1. Comparison of surface areas of soils obtained by nitrogen and water vapour adsorption methods.
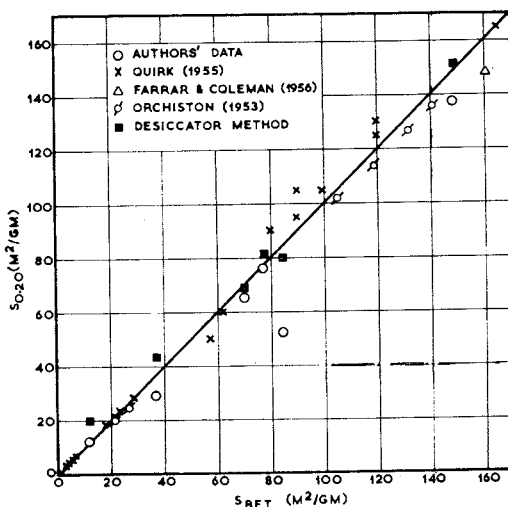
Fig. 2. Comparison of surface areas of soils by water vapour adsorption methods.

From Fig. 1 it is seen that the surface areas given by the water adsorption method are invariably more than those found by nitrogen adsorption. A statistical straight-line fit by the method of least squares gives the following relationship:

$$S_{H_2O} = 1.732\, S_{N_2} + 0.242 \quad \cdots \cdots \cdots (1)$$

where S_{H_2O} and S_{N_2} are the specific surface areas in m^2/gm measured by water adsorption and nitrogen adsorption respectively. The difference in the specific surface areas determined by the two methods has been explained by several investigators (e.g. COLEMAN & FARRAR, 1957; QUIRK, 1955; VAN OLPHEN, 1970) as the inability of the nitrogen molecules (being larger in size) to penetrate the clay particles. Although surface areas obtained by the water vapour adsorption method are more realistic for problems connected with soil mechanics, it is more common for the nitrogen adsorption method to be used. In such circumstances Eq. 1 may be of use in converting the measured values.

RECOMMENDED PROCEDURE

By testing various New Zealand soils using the water vapour adsorption method ORCHISTON (1953, 1954) observed that, except for some types of montmorillonite, a unimolecular layer was present at a partial pressure of 0.21. The data collected by QUIRK (1955) also showed the completion of a

unimolecular layer at a partial pressure of 0.20 for a number of soils. From the authors' data for Indian soils, specific surface areas have been computed on the assumption of the completion of the unimolecular layer at a partial pressure of 0.20; these are shown in Fig. 2, along with the results of OR-CHISTON (1953, 1954) and QUIRK (1955). It is seen from Fig. 2 that the agreement between surface area obtained by the conventional *BET* method and that calculated from the weight of water vapour adsorbed at a partial pressure of 0.20 is quite good. This observation led to the adoption of the following simple method for the determination of the surface area of soils.

The Desiccator Method

Weighed quantities (1 to 5 gm) of representative oven-dried samples of the soils were kept in a desiccator at a constant relative humidity of 20% (partial pressure of 0.20). An aqueous solution of sulphuric acid (as per ASTM E 104-51) was used to maintain the required relative humidity (for a relative humidity of 20% at 25°C, density of sulphuric acid required is 1.4789 gm/cm^3). Preliminary experiments conducted have shown that equilibrium is attained within 24 to 48 hours depending upon the type of soil, the time increasing with plasticity index. The equilibrium moisture content was determined using a balance with a sensitivity of 0.0001 gm. The specific surface area was calculated, by assuming that this water formed a unimolecular layer and that the area of a water molecule was equal to 10.8 Å^2, from the simple relationship:

$$S = \frac{w}{M} \frac{N}{10^4} A \cdot 10^{-16} \qquad \ldots \ldots \ldots \quad (2)$$

where S is the specific surface area in m^2/gm, w is the equilibrium moisture content in gm water adsorbed per gm of soil, N is Avogadro's number (6.025 × 10^{23}), M is the molecular weight of water (18.016 gm), and A is the area in square Angstroms per water molecule (10.8 Å^2). Hence:

$$S = 3612 \; w \; (\text{m}^2/\text{gm}) \qquad \ldots \ldots \ldots \quad (3)$$

The results for the six Indian soils obtained by this method show reasonably good agreement when compared with those obtained by the *BET* method (Fig. 2). A possible source of error in the desiccator method results from changes in relative humidity because of temperature variations. But it can be seen from ASTM E 104-51 that the effect of small variations in temperature on relative humidity is practically negligible. The best results could be obtained by using the desiccator at a place where temperature changes are least. The time taken to attain equilibrium can be considerably reduced if the desiccator is evacuated.

The desiccator method recommended herein for the determination of specific surface area of a soil is very simple both in the apparatus required and in the procedure adopted. Moreover, the results obtained are reasonably good and are sufficiently accurate for most soil engineering purposes.

ACKNOWLEDGEMENTS

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SOUTHEAST ASIAN SOCIETY NEWS

Report on the Third Conference

The Third Southeast Asian Conference on Soil Engineering was held in Hong Kong from 6 to 10 November, 1972. About 150 delegates representing many countries took part in the proceedings. More than 50 papers were selected for publication, and a high proportion of these were presented orally at the Conference. The proceedings were divided into six technical sessions entitled *Site Investigation, Foundations, Slope Stability, Soil Testing, Research and Theoretical Studies,* and *Roads and Pavements.*

The guest lecturer for the occasion was Professor Victor F.B. de Mello of São Paulo University in Brazil who spoke on "Thoughts on Soil Engineering Applicable to Residual Soils"; this was particularly applicable to Hong Kong. Professor T. William Lambe of Massachusetts Institute of Technology did an excellent job as *reporter at large* for the entire Conference proceedings. Chairmen of the sessions were Mr. S.G. Elliott of Scott, Wilson, Kirkpatrick & Partners, Hong Kong; Mr. H.K. Chang of the Hong Kong Public Works Department; Mr. J.C. Faber of S.E. Faber & Son, Hong Kong; Professor Chin Fung Kee of the University of Malaya; Dr. S.B. Tan of the Singapore Public Works Department; Dr. Za-Chieh Moh of the Asian Institute of Technology; and Mr. Jose C. Santos of the Philippine Bureau of Public Highways.

During the Conference, technical visits were made to the High Island Tunnel, the Plover Cove Reservoir and the Kotewall Road Landslide. Small earthquake tremors felt throughout Hong Kong at the time of the Conference, together with Typhoon Pamela, emphasized the varied types of loading to which earth structures and foundations in Hong Kong are subjected.

The conference was held under the auspices of the Southeast Asian Society of Soil Engineering, the Hong Kong Engineering Society, the Asian Institute of Technology and the Institution of Civil Engineers, London. Mr. Peter Lumb of the University of Hong Kong was chairman of the Organizing Committee and Mr. D.M. Allingham was Secretary. These gentlemen and all those involved with the organization are to be congratulated on having staged an excellent Conference.

Meeting of the General Committee

The General Committee of the Society met during the Third Conference in Hong Kong. Three decisions were made which are of importance to Members of the Society, viz:

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(1) Professor Chin Fung Kee of the University of Malaya was elected President of the Society as from 1 January 1973.

(2) Dr. John D. Nelson of A.I.T. offered to remain as Secretary and this was gladly accepted by the new President and the Committee.

(3) Mr. Sawarso Wignjosajono of Soilens, Bandung was elected to the General Committee as the representative for Indonesia.

(4) The Committee accepted with pleasure the offer from Malaysia to hold the Fourth Conference in Kuala Lumpur in April 1975.

CONFERENCE NEWS

Eighth International Conference

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

(1) Up-to-date methods for investigating the strength and deformability of soils (laboratory and field testing of soils for their strength, deformative and rheological properties).
Chairman : L. Suklje. Reporter : T.W. Lambe

(2) Interaction of soil bases and structures (prediction of settlement, design of massive foundations based on the limiting state, design of flexible foundation beams and slabs).
Chairman : E. de Beer. Reporters : M.I. Gorbunov-Posadov and
S.S. Dadidov

(3) Deep foundations, including pile foundations (design and new methods of construction).
Chairman : Á. Kézdi. Reporter : W.L. Zeevaert

(4) Problems of soil mechanics and construction on structurally unstable and weak soils (collapsible, expansive, loess, saline soils, etc.).
Chairman : G.A. Leonards. Reporters : L. Bjerrum and G.D. Aitchison

In addition, eight *Specialty Sessions* will be held on the subjects of :

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

Each main session will be preceded by a lecture given by an eminent specialist on the latest advancements in that particular field.

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

Conference on Settlement of Structures

A Conference on the Settlement of Structures will be held at Cambridge University from 2 to 4 April, 1974. The subject matter of the Conference will be dealt with under the following headings :

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

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

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

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

International Symposium on River Mechanics

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

The Symposium will be divided into three technical sessions :

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

Detailed information about the Symposium is given in Bulletins No. 1 and 2 which are now available from Dr. Subin Pinkayan, International Symposium on River Mechanics, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand.

NEWS OF PUBLICATIONS

Proceedings of the Seventh International Conference

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

Proceedings of the Southeast Asian Conferences

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

Proceedings of the Fourth Asian Regional Conference

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

Proceedings of the First Australia-New Zealand Geomechanics Conference

Copies of the *Proceedings of the First Australia-New Zealand Conference on Geomechanics*, which was held in Melbourne in August 1971, are available from the Institution of Engineers Australia, 157 Gloucester Street, Sydney, N.S.W. 200, Australia. The price is $ 40 (Aust.) plus postage.

Proceedings of the Fourth Panamerican Conference

The *Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering*, held in Puerto Rico in June 1971, have been printed in three volumes. Volume I contains the six state-of-the-art papers, Volume II contains the conference papers, and Volume III contains discussion and the proceedings of the session on the Business and Practice of Foundation

Engineering. These volumes are available separately at U.S. $ 8 each, or as a set for $20, from American Society of Civil Engineers, 345 East 47th Street, New York, N.Y. 10017, U.S.A.

Proceedings of the Fifth African Conference

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

Proceedings of the Roscoe Memorial Symposium

The Proceedings of the Symposium held at Cambridge University in March 1971 in honour of the late Professor K.H. Roscoe are now available under the title *Stress-Strain Behaviour of Soils*. The volume of Proceedings, containing 36 papers, four general reports and discussion is available at a price of £12.80 from the publisher, G.T. Foulis & Co. Ltd., 50a Bell Street, Henley-on-Thames, Oxfordshire, England.

Geotechnical Abstracts

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

BOOK REVIEWS

Vibrations of Soils and Foundations by F.E. Richart, J.R. Hall and R.D. Woods, Prentice-Hall, New Jersey, U.S.A., 1970, U.S. $ 16.95.

Vibrations of Soils and Foundations fills a gap which has existed for a long time in the range of available books in geotechnical engineering. The authors have clearly set out to compose a text book aimed largely at the needs of graduate students, and in this they have succeeded admirably. A number of chapters deal from first principles with the dynamics of systems, wave propagation in ideal media and theories of surface vibrations on elastic media. Two chapters are devoted to the dynamic behaviour of real soils. The chapter on instrumentation is excellent, and a very long chapter entitled *Design Procedures for Dynamically Loaded Foundations* is of inestimable value to the practicing engineer.

It is refreshing to find a geotechnical engineering book which is of such value to both students and practicing engineers. It is scholarly in its approach and contains much that is original. It is so written and produced as to be easy to read and to use. The authors are well-known for their many published papers in the field of soils and foundations vibrations, and their book rings with the authority with which they write. It must certainly establish itself as a standard work on the subject.

E.W. Brand

Practical Aspects of Soil Mechanics edited by M. Arnold, Department of Adult Education, University of Adelaide, Australia, 1972, Aust. $ 5 inc. postage.

Collected together in this 170 page cyclostyled volume are the texts of a series of eight lectures given as a university extension course in soil mechanics. The lectures are entitled *Site Investigation, Soil Identification and Classification* by M. Arnold, *Laboratory Testing—Its Relevance to Design and Construction* by K.C. Pile, *Pavement Design* by D.A. Cumming, *Earth Pressures and Retaining Walls* by R.G. Perry, *Slope Stability Analysis* by M. Arnold, *Shallow Foundations—General Aspects* by P.J. Fargher, *Shallow Foundations—Expansive Soils* by J.A. Woodburn, and *Deep Foundations* by S. Wawryk.

If it is remembered that the lectures were intended for practicing engineers working in the Adelaide area, the printed text can be described as good. This book might be useful to engineers working with soil conditions similar to those in South Australia, but others will find most of the written material to be rather elementary. The lectures on expansive soils and deep foundations are good, and a useful design approach is explained in detail in the lecture on slope stability.

E.W. Brand

NOTES ON CONTRIBUTIONS TO THIS JOURNAL

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

Original Papers

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

Technical Notes

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

Reprints

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

Discussions

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

News Items

As the official organ of the Southeast Asian Society of Soil Engineering this journal will publish any news item of interest to the Society members. Items to be included in the next issue (June, 1973) should be sent so as to reach the Editor not later than (1 May, 1973).