Undrained Shear Behaviour of Calcareous Sands

by

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Introduction

Calcareous sands occur extensively on the continental shelves in the region lying between latitudes 30° N and 30° S. These sands occur as skeletal remains of marine organisms and/or as nonskeletal oolites of calcareous material. Skeletal calcareous sands crush more readily than terrigeneous sands when sheared under drained conditions and significant crushing occurs in such sands even at low confining pressures (Datta et al, 1979). Piles driven in calcareous sands encounter low resistance (McClelland, 1974) and the phenomenon of soil 'set up' has also been observed in calcareous sand layers (Aggarwal et al, 1977) i.e. when pile driving is interrupted, partially driven piles have to overcome increased resistance to redriving due to soil 'set up'. It has been suggested that particle crushing occurs during pile driving and this reduces the skin friction (Angemeer et al, 1973 and 1975). The phenomenon of soil 'set up' is, in general explained in terms of dissipation of positive pore water pressures which develop as particles crush during pile driving (Vijayvergia et al, 1977). It is therefore of interest to study the phenomenon of crushing and the development of pore water pressures in calcareous sands.

Sands are usually tested under drained conditions since they have a high permeability and the usual rates of application of loads in the field are low. However, there are conditions when the loading is so rapid that drainage may not occur e.g. during earthquakes, ocean storms and perhaps during pile driving. In such cases the development of pore water pressures is of importance since they effect the strength of the soil. Although for the examples quoted above one would require knowledge of the undrained shear behaviour of sands under cyclic loading, one must first appreciate the static shear strength behaviour of sands under undrained conditions. Lee (1976) has even shown that static undrained behaviour of sands could be used to explain the response of sands to cyclic loading.

This paper presents the results of an experimental investigation undertaken to determine the amount of crushing in four dense calcareous sands under static undrained triaxial shear and to assess the influence of crushing on their pore water pressure response and on their shear strength characteristics. It was felt that in the marine environment the insitu pore water

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pressures would be high enough to preclude cavitation during shear, hence all samples were back pressured prior to shear to prevent cavitation during testing.

Literature Review

Bishop and Eldin (1953) were the first to study the undrained behaviour of sands in noncavitating tests. They observed that the maximum value of principal effective stress ratio occurred at small strains (about 5 percent) whereas the maximum value of the deviator stress occurred at larger strains (about 15 to 30 percent) and that the values of angle of shearing resistance determined at the latter strain ($\bar{\phi}_f$) were lower by about 2 degrees from those determined at the maximum principal effective stress ratio ($\bar{\phi}_p$). Bjerrum et al (1961) showed that, while $\bar{\phi}_p$ occurred at lower strains than ϕ_f for dense sands, this trend was reversed in the case of loose sands. Values of A factor at failure (A_f) were shown to be about -0.25 for dense sands and as high as 2.7 for very loose sands.

Seed and Lee (1967) conducted a study to correlate the drained and undrained shear behaviour of sands. For this purpose they used the concept of critical confining pressure ($\bar{\sigma}_{3cr}$) which was defined as the confining pressure at which the net pore water pressure change at failure (or volume change at failure in a drained test) is zero for any given post consolidation void ratio. Samples having confining pressures lower than the critical value exhibit net negative pore water pressure change at failure whereas those with higher confining pressure yield positive values. Values of $\bar{\sigma}_{3cr}$ obtained from drained and undrained tests were found to be nearly equal. Predictions of undrained shear strength on the assumption that the effective minor principal stress at failure ($\bar{\sigma}_{3f}$) would equal $\bar{\sigma}_{3cr}$ for the particular void ratio yielded reasonably good results. $\bar{\sigma}_{3cr}$ was shown to have a unique relation with the post consolidation void ratio; it decreased with increasing void ratio. The authors observed that for dense sands, $\bar{\sigma}_{3cr}$ is lower for sands which are angular and have greater susceptibility to crush than for sands with sound rounded particles.

Newland and Allely (1959) were perhaps the first to observe that volume changes occur in undrained tests due to membrane penetration. Subsequently many investigators have studied this phenomenon, (Lade and Hernandez (1977), Kiekbusch and Schuppener (1977) and Raju and Venkataramana (1978)). Their findings may be summarised as follows: (*i*) when the initial confining pressure ($\bar{\sigma}_c$) is greater than $\bar{\sigma}_{3cr}$, positive pore water pressures develop and the effect of membrane penetration is to reduce the positive pore water pressures and therefore increase the deviator stress. When $\bar{\sigma}_c$ is less than $\bar{\sigma}_{3cr}$, negative pore water pressures develop and membrane penetration causes them to be not so negative thereby decreasing the deviator stress; (*ii*) no simple correction factor exists which can be applied to pore water pressure measurements to account for membrane penetration effects; and (*iii*) the effective strength envelope is not affected by membrane penetration.

The authors are not aware of any published data relating particle crushing to undrained shear behaviour in granular soils. Most studies on crushing behaviour have been reported for drained conditions, e.g. Lee and Seed (1967), Vesic and Clough (1968), Ramamurthy and Lal (1970), Ramamurthy et al (1974) and Datta et al (1979). These investigations have revealed that crushing increases with increasing confinement, application of shear stress, increasing angularity of particles, increasing size of particles, increasing abundance of intraparticle voids and plate like shell fragments and decrease in mineral hardness. As crushing increases, the deviator stress-axial strain behaviour changes from brittle to plastic type, volume change during shear alters from dilatant to compressive and there is a decrease in the maximum principal effective stress ratio indicating a reduction in the drained angle of shearing resistance.

Experimental Investigation

Sands Tested

Four calcareous sands of skeletal origin having a calcium carbonate content of greater than 85 percent were used for the experimental investigation. Three of these sands were obtained from a 40 km coastal belt along the west coast of Gujarat in India and the fourth sand was procured from Lakshadweep Island in the Arabian Sea. Tests were also conducted on Ottawa sand, a terrigeneous material known to be highly resistant to crushing. The nature and physical characteristics of the five sands are given in Table 1.

TABLE 1

Nature and Physical Characteristics of the Sands Tested

Sand	Location	G_s	e_{max}	e_{min}	Particle size	C_u
A*	West Coast, India	2.81	1.39	0.93	Coarse	1.50
B*	West Coast, India	2.80	1.13	0.77	Coarse medium	2.11
C*	West Coast, India	2.78	1.13	0.78	Medium coarse	2.12
D**	Lakshdweep Island, India	2.78	1.20	0.80	Medium fine	1.53
E***	Ottawa, U.S.A.	2.66	0.69	0.52	Medium coarse	1.33

*skeletal calcareous material; abundance of large intraparticle voids and plate-like shell fragments; angular to subrounded particles.

**skeletal calcareous material; coralline debris; small intraparticle voids; subrounded particles.

***terrigeneous material; rounded particles.

Testing Procedure

Two test series were conducted on all sands. Test Series I consisted of subjecting the sands to only isotropic consolidation in a triaxial cell at different cell pressures ; the maximum value used was 64 kg/cm^2. In Test

Series II, consolidated undrained triaxial shear tests with pore water pressure measurements were performed at low and elevated cell pressures, with a maximum cell pressure of 64 kg/cm^2. The terms low and elevated signify pressure ranges from 0 to 10 kg/cm^2 and 10 to 100 kg/cm^2 respectively. Samples of 3.81 cm diameter and 7.62 cm height were prepared by placing deaired sand under water and tamping it in a split former which was also tapped from the outside to produce small vibrations so as to obtain the minimum void ratio, i.e. a relative density of 100 per cent. Samples were subjected to back pressure to ensure saturation. Only samples which exhibited a B-factor of greater than 0.95 in 30 seconds were considered acceptable for testing. Back pressures were kept greater than the maximum negative pore water pressures expected to develop in the test. For Ottawa sand back pressures as high as 32 kg/cm^2 were applied but even these were inadequate and tests had to be terminated at low strains. At low pressures, samples were tested in the ordinary triaxial cells (Wykeham Farrance) with cell pressures applied through a mercury pot system and the pore water pressure measurements were made using a null indicator. For elevated pressures a high pressure steel cell along with a hydraulic cell pressure application system (Wykeham Farrance) was used and Bell and Howell pressure transducers were used to measure the pore water pressures. The testing procedure adopted was as per Bishop and Henkel (1962). All samples were sheared to an axial strain of 20 per cent. No device was used to minimise membrane penetration.

Results

Particle crushing

Grain size distribution curves were obtained after conducting tests in each of the two series. The magnitude of crushing was found by evaluating the crushing coefficient C_c, a dimensionless parameter defined by Datta et al (1979) as per equation (1).

$$C_c = \frac{\text{Percentage of particles of sand, after being subjected to stress, finer than } D_{10} \text{ of the original sand}}{\text{Percentage of particles of original sand finer than } D_{10} \text{ of the original sand.}} \quad \ldots(1)$$

(The denominator of C_c is, by definition, equal to 10). Figure 1 shows the variation of C_c with confining pressure, $\bar{\sigma}_c$, for sand A, for both Test Series I and II. It is evident that crushing increases with increasing confinement and that crushing of samples subjected to shear is greater than those of samples subjected to isotropic compression only. This observations holds for other sands also.

Figure 2 shows the plot of post shear crushing coefficient versus confining pressure for the four calcareous sands, from which one observes that sand A exhibits the maximum crushing in relation to sand B, C and D. Since Ottawa sand (sand E) could not be sheared to 20 per cent strain its crushing coefficients could not be determined. Datta et al (1979) showed that under drained conditions the values of C_c for Ottawa sand were much lower than those for calcareous sands and similar trend may be expected to hold for undrained shear. The term "susceptibility to crushing" will be used later in this paper to indicate the relative ease of crushing of different sands. The post shear crushing coefficient at a confining pressure

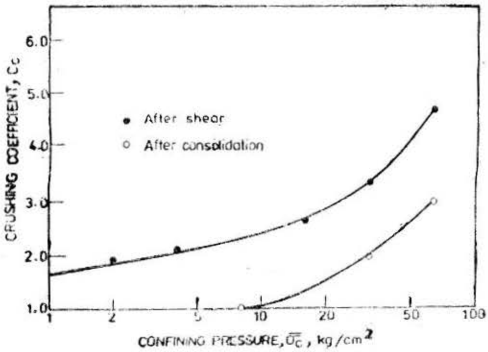

FIGURE 1 Effect of confining pressure on crushing coefficient for sand A : after consolidation and after shearing.

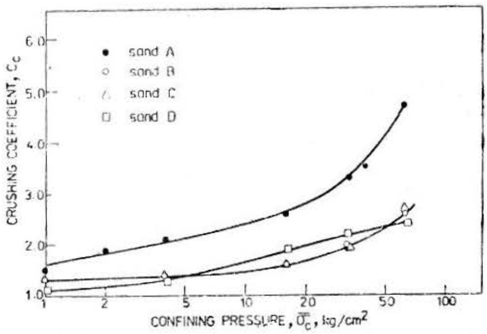

FIGURE 2 Effect of confining pressure on post-shear crushing coefficient.

of 64 kg/cm^2 will be used to quantitatively express this "susceptibility to crushing". The sands in order of decreasing susceptibility to crushing are, as such, sand A, B/C, D and then E.

Stress-strain and Pore water pressure-strain behaviour

The relationships between stress and strain and between pore water pressure and strain are shown in Figure 3 and Figure 4 respectively for sand A for different confining pressures. The variations of principal effective stress ratio $(\bar{\sigma}_1/\bar{\sigma}_3)$, A-factor and the effective minor principal stress $(\bar{\sigma}_3)$ with strain are shown in Figures 5(a), 5(b) and 6 respectively. Similar trends were observed for the other three calcareous sands. Shapes of stress-strain and pore water pressure-strain relationships for Ottawa sand were markedly different as shown in Figure 7 where data from incomplete tests are plotted along with the curves of sand A. Tables 2(a) and 2(b) present the conditions at failure for sands A to D as per the two failure criteria used (See para 1 of Discussion) as well as data from incomplete tests on sand E.

Stress-strain behaviour

Referring to Figure 3 it is observed that as the confining pressure increases the peak deviator stress increases and so also does the initial

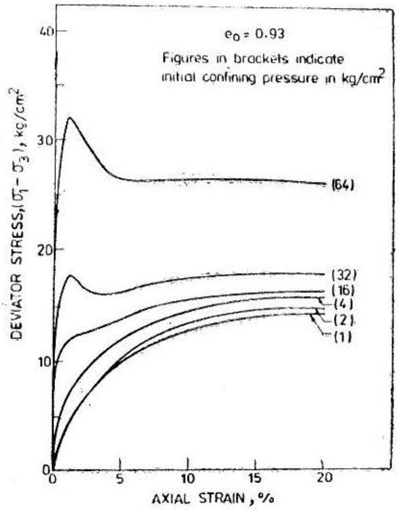

FIGURE 3. Stress-strain behaviour for sand A.

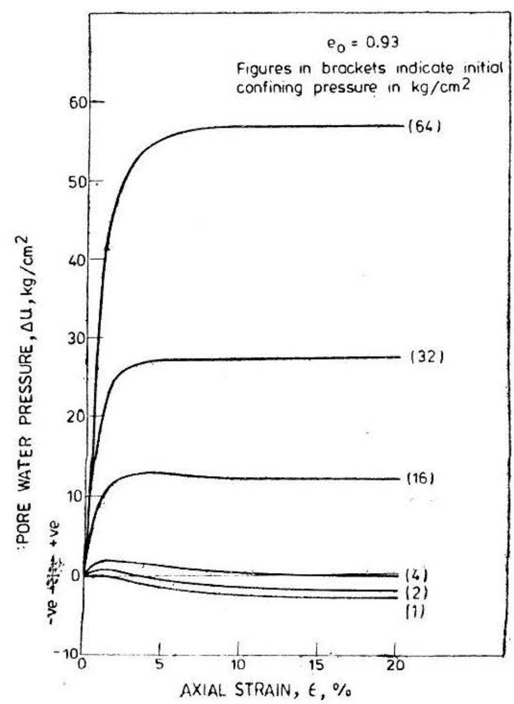

FIGURE 4. Pore water pressure-strain behaviour for sand A.

tangent modulus. The increase in the deviator stress with increase in cell pressure is marginal for low cell pressures but becomes some what more pronounced at elevated cell pressures. A peculiar characteristic of the stress strain curves at elevated cell pressures is the development of an

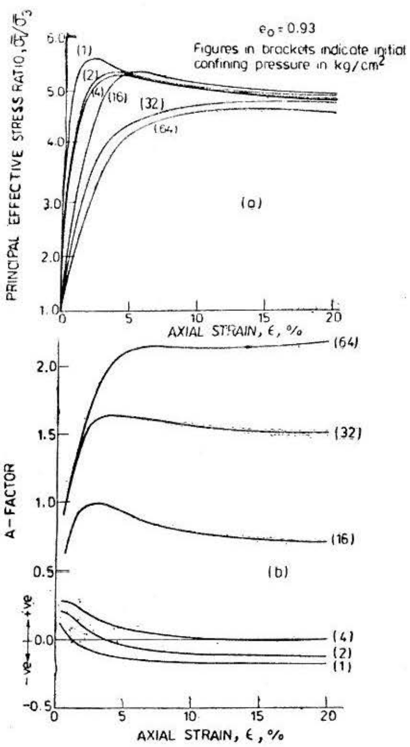

FIGURE 5(a) Variation of principal effective stress ratio with strain for sand A.
(b) Variation of A-factor with strain for sand A.

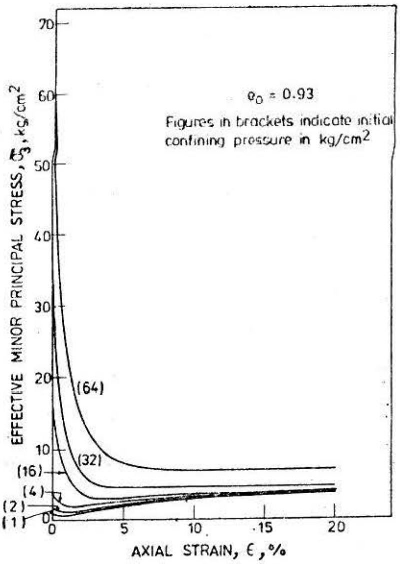

FIGURE 6 Variation of effective minor principal stress with strain for sand A.

initial peak at about 1 percent strain. This characteristic is most pronounced in sand A and diminishes successively in sands B, C and D; it is not observed at all in sand E. Indications of such initial peaks are also observed in stress-strain curves presented by Lade and Hernandez (1977). In all sands and at all confining pressures, except for sand A at $\bar{\sigma}_c = 64$ kg/cm^2, the deviator stress at the initial peak is less than the maximum deviator stress, hence this peak has not been used as a failure criterion. Table 2(b) shows that the maximum deviator stress occurs at strains of 18 to 20 percent for low cell pressures and normally between 14.5 and 17 percent strain for elevated cell pressures whereas Table 2(a) shows the maximum value of principal effective stress ratio normally occurs at low strains in the range of 2 to 8 percent.

Pore water Pressure-strain behaviour

The pore water pressure-strain behaviour for the four calcareous sands tested shows a change in the pore water pressure response from negative at low cell pressures to positive at elevated cell pressures (see Figure 4 for sand A), even though all the sand samples have an initial relative density of 100 percent. In contrast, Ottawa sand (sand E) developed negative

TABLE 2(a)

Conditions at Failure : Failure Criteria—$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$

Sand	$\bar{\sigma}_c$	$(\bar{\sigma}_1/\bar{\sigma}_3)_p$	ε_p	$(\sigma_1-\sigma_3)_p$	$\triangle U_p$	A_p	$\bar{\sigma}_{3p}$	$\bar{\phi}_p$
A	1	5.60	2.0	6.02	—0.31	—0.05	1.31	44.18
	2	5.40	3.5	8.45	0.08	0.01	1.92	43·43
	4	5.32	4.0	11.26	1.39	0.12	2.61	43.10
	16	5.40	5.5	13.89	12.84	0.92	13.16	43.42
	32	4.84	16.5	18.07	27.29	1.51	4.71	41.10
	64	4.73	14.5	26.78	56.83	2.12	7.17	40.61
B	1	5.06	3.5	11.17	—1.75	—0.16	2.75	42.06
	4	4.87	3.5	15·16	0.08	0.01	3.92	41·22
	16	4·96	3.5	19.67	11.03	0.56	4.97	41.64
	32	4.85	8.0	26.90	25.00	0.93	7.00	41.18
	64	4.71	10.0	35.72	54.37	1.52	9.63	40.52
C	1	5.10	4.5	13.16	—2.21	—0.17	3.21	42.19
	4	5.10	4.5	15.25	0.28	0.02	3.72	42.23
	16	5.24	5.0	20.61	11.12	0.54	4.86	42.81
	32	5.07	7.0	26·30	25.54	0.97	6.46	42.11
	64	4·95	8 0	35.30	55.07	1·56	8.93	41.07
D	1	5.66	3.3	8.86	—0.90	—0.10	1.90	44.40
	4	5.35	4.0	17.65	—0.06	—0.003	4.06	43.23
	16	5.43	4.5	23.15	10.77	0.47	5.23	43.53
	32	4.96	7.0	38.10	22.37	0.59	9.63	41.62
	64	4.95	7.0	46.36	52.26	1.13	11.74	41.59
E	1	4.32	1.5	17.32	—4.21	—0·24	5.21	38.61
	32	3.81	2.0	101.72	—4.22	—0.04	36.22	35.74
	64	—	—	—	—	—	—	—

TABLE 2(b)

Conditions at Failure : Failure Criteria—$(\sigma_1-\sigma_3)_{max}$

Sand	$\bar{\sigma}_c$	$(\sigma_1-\sigma_3)_f$	ε_f	$(\bar{\sigma}_1/\bar{\sigma}_3)_f$	$\triangle U_f$	A_f	$\bar{\sigma}_{3f}$	$\bar{\phi}_f$
A	1	14.50	20.0	4.87	—2.75	—0.19	3.75	41.23
	2	15.00	19.5	4.75	—2.00	—0.13	4.00	40.70
	4	16.02	18.0	4.89	—0.12	—0.01	4.12	41.32
	16	16.57	19.5	4.96	11.82	0.71	4.18	41.65
	32	18.07	16.5	4.84	27.29	1.51	4.71	41.10
	64	26.78	14.5	4.74	56.83	2.12	7.17	40.61
B	1	24.00	19.0	4.52	—5.85	—0.24	6.85	39.62
	4	24.88	18.0	4.55	—3.00	—0.12	7.00	39.78
	16	27.13	18.5	4.52	8.30	0.31	7.70	39.63
	32	28.90	17.0	4.65	24.10	0.83	7.90	40.28
	64	36.68	15.5	4.55	53.67	1.46	10.33	39.72
C	1	23.22	18.5	4.62	—5.41	—0.23	6.41	40.11
	4	25.18	18.0	4.62	—2.94	—0.12	6.95	40.12
	16	25.94	15.5	4.53	8.66	0.33	7.43	39.69
	32	28.96	16.5	4.76	24.30	0.84	7.70	40.76
	64	36.65	14.5	4.67	54.02	1.47	9.98	40.34
D	1	31.90	20.0	4.63	—7.80	—0.25	8.80	40.14
	4	33.98	18.5	4.46	—5.82	—0.17	9.82	39.33
	16	35.50	16.5	4.58	6.08	0.17	9.92	39.90
	32	45.13	17.0	4.63	19.56	0.43	12.44	40.14
	64	51.54	15.5	4.54	49.45	0.95	14.55	39.72
E*	1	78.89	5.0	3.65	—28.82	—0.36	29.82	34.74
	32	146.00	4.5	3.39	—29.17	—0.20	61.17	32.99
	64	161.49	3.5	3.43	—2.44	—0.02	66.44	33.27

*Data from incomplete test which had to be stopped because of large negative pore water pressures induced.

pore water pressures even at a cell pressure at 64 kg/cm² (see Figure 7). Figure 5(b) shows that A factors of greater than unity are obtained at elevated cell pressures for sand A. The A values are highest for sand A and lowest for sand D (see Tables 2(a) and 2(b)). The effect of the induced pore water pressures is to change the effective minor principal stress (see Figure 6 for sand A) which shows that at low confining pressures the induced negative pore water pressure causes an increase in the effective minor principal stress and at elevated confining pressures, the positive pore water pressure induced, drastically reduces the effective minor principal stress with the result that the increase in $\bar{\sigma}_{3f}$ is very small.

Discussion

Failure criteria

Since the maximum principal effective stress ratio is reached at a different strain than the maximum deviator stress, both the above mentioned conditions have been taken to represent failure and the data has been analysed separately for each condition. For failure at $(\sigma_1/\sigma_3)_{max}$ all notations have been subscripted by 'p' and for failure at $(\sigma_1\text{-}\sigma_3)_{max}$ all notations have been subscripted by 'f'. When $(\sigma_1\text{-}\sigma_3)_{max}$ was not reached at 20 percent strain, the value at 20 percent strain was taken as representing failure for the latter criterion. In this paper results are presented for

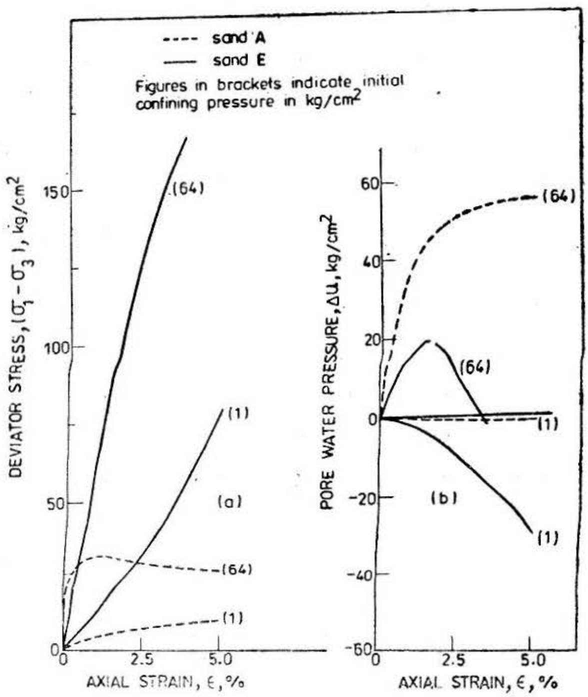

FIGURE 7(a) Stress-strain behaviour for sands A and E.
(b) Pore water pressure-strain behaviour for sands A and E.

both these criteria when necessary. If however the two criteria yielded similar trends, the results are presented for failure at maximum deviator stress only.

Angle of shearing resistance

Figure 8 shows the plot of secant angles of shearing resistance $\bar{\phi}_f$ and $\bar{\phi}_p$ plotted against the confining pressure σ_c. The secant angles were determined using the relation $\sigma_1/\sigma_3 = \tan^2(45+\bar{\phi}/2)$. It is evident from the figure that as the confining pressure increases, the $\bar{\phi}_p$ decreases whereas $\bar{\phi}_f$ remains essentially constant. It appears that with increase in confinement the value of $\bar{\phi}_p$ approaches that of $\bar{\phi}_f$. With the criterion that maximum deviator stress represents failure, one would conclude that crushing, which increases with increase in confining pressure, has no effect on the angle of shearing resistance whereas the failure criterion of maximum $\bar{\sigma}_1/\bar{\sigma}_3$ indicates that crushing causes a small decrease in the value of $\bar{\phi}_p$, the maximum decrease being 3.5 degrees for sand A.

Pore water pressure development/A factors

Figure 9 presents the variation of A factors at failure (A_f) with increasing confining pressure for the four calcareous sands, from which it is apparant that A_f increases significantly with confinement and attains not only positive values but values which are even greater than unity. On the other hand, incomplete tests on Ottawa sand show that *A* factors are negative even for a confining pressure of 64 kg/cm². This unusual behaviour of calcareous sands may well be on account of their high propensity to crush when stressed, data for which has already been presented in Figures 1 and 2. Figure 10 presents a plot of A_f versus C_c which shows that *A*

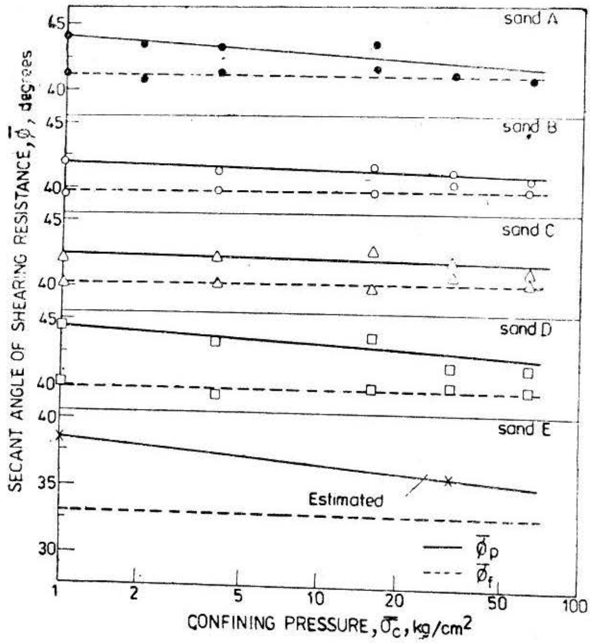

FIGURE 8 Relationship between angle of shearing resistance and confining pressure

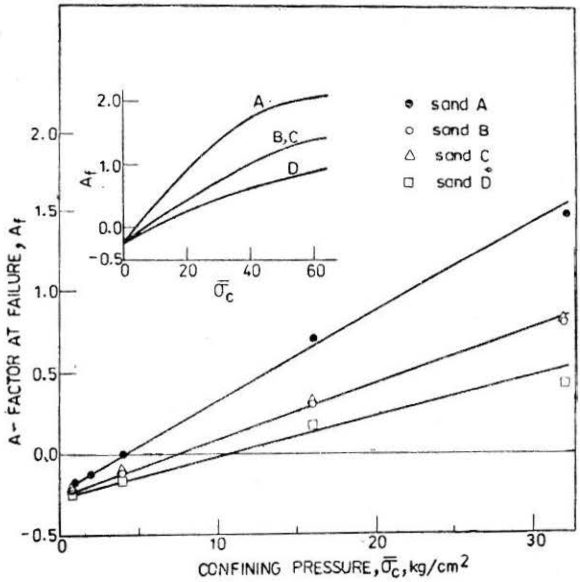

FIGURE 9 Variation of A-factor at failure with confining pressure

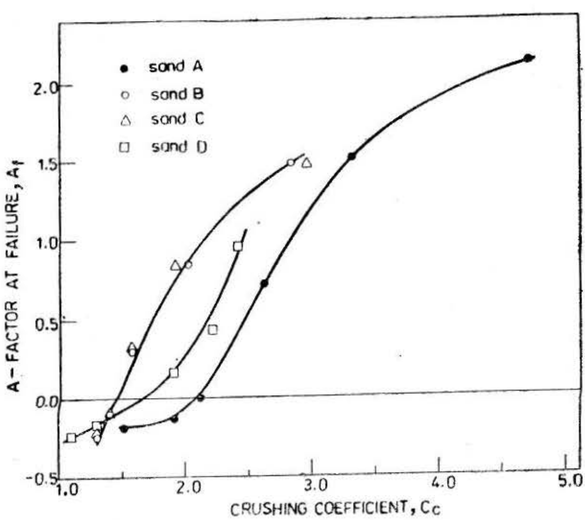

FIGURE 10 Variation of A-factor at failure with crushing coefficient

factors at failure increase as the magnitude of crushing increases. The crushing coefficient at which A_f values become zero may be designated as the critical crushing coefficient, C_{cr}.

The values of $\bar{\sigma}_{3cr}$ for the four sands investigated was determined from Figure 9 by noting the value of $\bar{\sigma}_c$ at zero A_f. (This procedure is slightly different from that of Seed and Lee, 1967). Figure 11 (a) shows the variation of $\bar{\sigma}_{3cr}$ with susceptibility to crushing and Figure 11 (b) shows

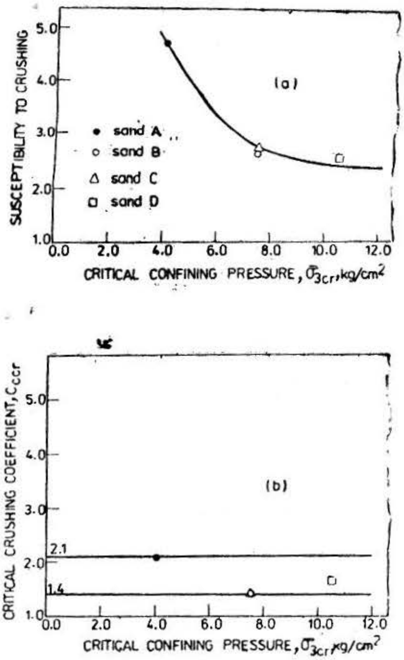

FIGURE 11 (a) Relationship between susceptibility to crushing and critical confining pressure

(b) Relationship between critical crushing coeflicient and critical confining pressure

the plot of C_{Ccr} versus $\bar{\sigma}_{3cr}$. From these figures it is evident that (*i*) sands which are more susceptible to crushing have lower values of $\bar{\sigma}_{3cr}$ and (*ii*) regardless of the magnitude of $\bar{\sigma}_{3cr}$, the values of C_{Ccr} lie within a small range of 1.4 to 2.1, from which one may conclude that a certain amount of crushing is essential for the pore water pressure response to change from negative to positive. For sands which do not crush so readily, e.g. Ottawa sand the critical crushing coefficients are not reached until very high confining stresses are applied. Seed and Lee (1967) have reported that the values of $\bar{\sigma}_{3cr}$ for Ottawa sand from drained tests is between 75 and 100 kg/cm^2.

Stress paths

Figure 12 shows the effective stress paths for sand A at different confining pressures. It is observed that for confining pressures less than $\bar{\sigma}_{3cr}$ the stress paths have a form similar to the form of stress paths of an overconsolidated soil whereas for confining pressures greater than $\bar{\sigma}_{3cr}$, the stress paths are similar to those of a normally consolidated soil. This transformation behaviour is on account of change in induced pore water pressures from negative for $\bar{\sigma}_c$ below $\bar{\sigma}_{3cr}$ to positive for $\bar{\sigma}_c$ greater than $\bar{\sigma}_{3cr}$.

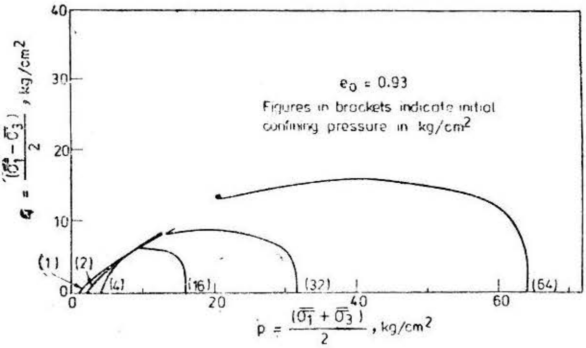

FIGURE 12 Effective stress paths for sand A

Undrained strength

The undrained strength of sand S_u may be expressed in terms of the effective minor principal stress at failure $\bar{\sigma}_{3f}$, and $\bar{\phi}_f$ as given below.

$$S_u = \frac{(\sigma_1 - \sigma_3)_f}{2} = \frac{\bar{\sigma}_{3f}}{2}\left(\frac{\bar{\sigma}_{1f}}{\bar{\sigma}_{3f}} - 1\right) \qquad ...(2)$$

$$= \tfrac{1}{2}\bar{\sigma}_{3f}\left[\tan^2\left(45 + \frac{\bar{\phi}_f}{2}\right) - 1\right] \qquad ...(3)$$

For each of the four sands tested, $\bar{\phi}_f$ was essentially independent of the confining pressures used. Thus the undrained strength of each sand is a function of $\bar{\sigma}_{3f}$. Seed and Lee (1967) showed that the effective minor principal stress at failure tends to become equal to $\bar{\sigma}_{3cr}$. Figure 13 shows a plot of $\bar{\sigma}_{3f}/\bar{\sigma}_{3cr}$ versus $\bar{\sigma}_c$ from which it is apparent that $\bar{\sigma}_{3f}/\bar{\sigma}_{3cr}$ is not equal to unity but close to it and increases slightly with increase in confining pressure. Equation (3) may be rewritten as follows,

$$S_u = \tfrac{1}{2}\bar{\sigma}_{3cr}\left(\frac{\bar{\sigma}_{3f}}{\bar{\sigma}_{3cr}}\right)\left[\tan^2\left(45 + \frac{\bar{\phi}_f}{2}\right) - 1\right] \qquad ...(4)$$

In this equation the expression in square brackets is a constant and the expression in parenthesis is close to unity which implies that the undrained strength is a function of $\bar{\sigma}_{3cr}$. For calcareous sands $\bar{\sigma}_{3cr}$ lies in the range of 4.0 to 10.5 kg/cm² whereas for Ottawa sand it is reported to be between 75 and 100 kg/cm², Even though the $\bar{\phi}_f$ of Ottawa sand is lower than that of calcareous sands studied, the dominating influence in determining the undrained strength is that of $\bar{\sigma}_{3cr}$. This evident from Table 2 (b) and Figure 7 which show that the recorded deviator stresses even at low strains in tests on Ottawa sand are much greater than those at failure for calcareous sands. This observation is of major consequence when one recalls that piles driven in calcareous soils encounter low resistance during driving.

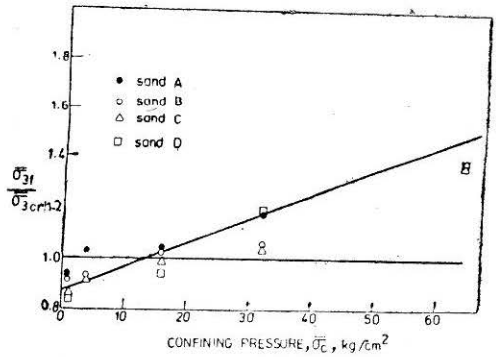

FIGURE 13 Variation of $\bar{\sigma}_{3f}/\bar{\sigma}_{3cr}$ with confining pressure

Membrane penetration

The results reported in this paper are of tests in which no attempt was made to eliminate membrane penetration effects. It may be noted that elimination of membrane penetration would have further increased the positive pore water pressures in the calcareous sands and reduced the undrained strengths. For Ottawa sand elimination of membrane penetration would have yielded more negative pore water pressures and higher undrained strengths. Both these effects would only tend to further reinforce the conclusions of this study.

Conclusions

Dense calcareous sands crush significantly under undrained shear. The crushing of sand particles does not significantly reduce the angle of shearing resistance as has been reported for drained conditions. Increase in the magnitude of crushing causes the pore water pressure response to change from negative to positive when crushing coefficients of 1.4 to 2.1 are achieved. The undrained shear strength is strongly dependent on the critical confining pressure which decreases with increase in susceptibility to crushing. Calcareous sands have very low critical confining pressures as compared to terrigeneous sands and thus have low undrained shear strength.

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