Behaviour of Coarse-Grained Soils under High Stresses

by

T. Ramamurthy*

V.K. Kanitkar**

Kamlesh Prakash***

Introduction

IN the recent past, some attention has been paid to study the behaviour of soils under high confining stresses with the use of heavy compaction equipment in the construction of earth and rockfill dams. Tropical soils undergo considerable amount of degradation under high stresses resulting complete change in their behaviour. Extrapolation of test results obtained at low stresses for predicting the behaviour under high stresses becomes intolerable and necessitates testing of soils under stress ranges anticipated in design and construction of earth and rockfill dams for evaluating appropriate design data.

The various factors contributing to crushing, nature of crushing and its occurrence have been presented elsewhere (Ramamurthy 1969). In the present study two coarse, one medium and one fine Badarpur sands have been tested in axisymmetrical stress conditions in triaxial under various confining stresses ranging up to 127 kg/cm^2. In addition to the study of stress-strain, axial and volumetric strains, strength and magnitude of crushing, the influence of axial strain, tensile strength of particles, particle size, strain rate on the magnitude of crushing have been presented.

Notations

σ'_1 & σ'_3 = Effective major and minor principal stresses

ϵ_1 & v = Axial and volumetric strains during shear

e_{min} & e_{max} = Minimum and maximum void ratios

v_c = Volumetric strain due to isotropic consolidation only

ϕ' = Effective angle of shearing resistance

d_{50} = Diameter corresponding to 50 percent finer by weight

C_u = Coefficient of uniformity

τ_o = Tensile strength of particle

K_o = Coefficient of earth pressure at rest.

* Asstt. Professor, Civil Engineering Department, Indian Institute of Technology, New Delhi-110029.

** SO2 (Designs), Chief Engineer's Office, Central Zone, Jabalpur.

*** SO2 (Designs), Chief Engineer's Office, Delhi Cantt.

This paper (modified) was received on 29 September 1973. It is open for discussion up to July 1974.

Literature Review

Soils ranging from boulders to clays have been tested by various investigators to study their behaviour under high confining stresses. A comprehensive review was made by Ramamurthy (1969) and Ramamurthy and Lall (1970). Only the recent relevant literature on the behaviour of coarse-grained soils is presented briefly with special reference to shear strength, axial and volumetric strains at failure, magnitude of crushing and shape of Mohr rupture envelope on silts, sands, gravels and rockfills.

Silts

Hirschfeld and Poulos (1963) tested undisturbed specimens of silt up to a maximum confining stress of about 42 kg/cm². The triaxial compression tests showed maximum compressional volumetric strain of the order of 3 percent and maximum axial strain at failure of 12 percent. Very little crushing was reported. The Mohr rupture envelope was curved showing a total decrease of 9° in the effective angle of shearing resistance (ϕ') from low to high stress range. The curved nature of the rupture envelope was attributed to brake down of the initial particulate structure of the undisturbed specimens.

Sands

Most investigators have tested sands under high cofining stresses ranging up to 70 kg/cm² (Hirschfeld and Poulos 1963, Hall and Gordon 1963, Marsal 1963, Bishop *et al.* 1965, Bishop 1966, Ramamurthy and Lall 1970), up to 281 kg/cm² by Skinner (Bishop 1966) and up to 633 kg/cm² by Vesic and Barksdale (1963) and Vesic and Clough (1968). The maximum compressional volumetric strains varied from 10 to 15 percent and the axial strains at failure from 20 to 30 percent. Significant crushing was reported during triaxial compression shear with major crushing occurring during shear. Crushing was more in uniformly graded sands. Curved rupture envelope was observed in all cases with the envelope becoming straight at high stresses (Vesic and Clough 1968, Ramamurthy and Lall 1970). The values of the effective angle of shearing resistance at high confining stresses were lower by 10° to 20° than those obtained at low confining stresses. The effects of initial porosity disappeared when sheared under high stresses.

Gravels

Hall and Gordon (1963) conducted tests on soils predominantly gravels up to a maximum confining stress of 38·7 kg/cm². Maximum compressional volumetric strains varied from 2 to 4 percent and axial strains at failure from 15 to 20 percent. Significant crushing was mentioned. Curved rupture envelops were observed with effective angle of shearing resistance at high stress 5° to 6° lower than that obtained at low stresses.

Rockfills

Large scale triaxial tests (115 cm diameter $\times$ 250 cm height) up to confining stress of 25 kg/cm² were conducted by Marsal (1967) on three different rockfill materials. The maximum compressional volumetric strains varied from 6 to 10 percent when the axial strains at failure were about 13 percent. The gradation curves before and after shear revealed significant amount of crushing. This was predominant in uniformly graded rockfill materials. The rupture envelops were curved in all the cases. The effective angle of shearing resistance at high confining stresses decreased by 5° to 6°.

Leps (1970) review of shear strength of rockfills showed an average reduction of the effective angle of shearing resistance from 55° to 38° in the range of confining stresses of 0·07 kg/cm^2 to 35 kg/cm^2. Rockfill material (granite gneiss) which was initially proposed for Mica Dam (244 m) had to be abandoned since it crushed extensively into micaceous sand and indicated very low effective angle of shearing resistance when tested under appropriate stress range anticipated during construction; the design of the dam had to be changed to a gravel-soil embankment (Thompson 1971). Marachi *et al.* (1972) suggested modelling of the rockfill material (i.e., adopting reduced grain size of the test material retaining the shape of the grain size distribution curve. Even if shape, surface roughness and fissures present in rockfill material are not reproduced in the modelling material, the behaviour of the modelled material may not be significantly different from the rockfill material due to crushing.

Available experimental data indicates that sands have been subjected to much higher confining stresses than any other coarse grained soil. Consequently, sands have exhibited larger volume contraction, larger axial strains at failure, larger crushing and larger reduction in the effective angle of shearing resistance.

Literature reveals that very little is known of the influences of particle size and particle strength on the magnitude of crushing, axial strain in mobilizing shear strength and producing quality and quantity of crushing, strain rate on the magnitude of crushing; magnitude of crushing and particle strength on elastic and consolidation volumetric strains during isotropic consolidation and also energy imparted during shear in changing surface area of the material. These aspects have been investigated in the persent study. Information on the influence of high stresses on elastic and consolidation volumetric strains on four fractions of Badarpur sand which experienced considerable crushing, were reported elsewhere (Ramamurthy and Kanitkar 1972).

Material

White Badarpur sand, quarried from Fatepur, Delhi, was collected from Badarpur site. It is a poorly graded coarse sand of weathered quartzite. The grain size distribution curves for the portion of the sand passing through B.S. sieve No. 7 and for the four fractions obtained for this sand are shown in Figure 1. By sieving the original sand through successive sieves, samples having mean particle diameter (diameter at 50 percent finer by weight) of 1·7 mm, 1·0 mm and 0·5 mm were obtained. By grinding the original sand, sample having mean particle diameter of 0·096 mm was obtained. The ratio of the mean diameters of the coarser to the fine sand was about 17. The properties of these four fractions of sand are given in Table I. Hardness of particles was determined by scratching test using Moh's Scale of hardness. Individual particles of each size was seen under microscope to determine roundness and sphericity from the charts prepared by Wadell (1935) and Rittenhouse (1943) respectively. The particles of sand having 1·7 mm and 1·0 mm mean diameter were more angular than those of the other two sands.

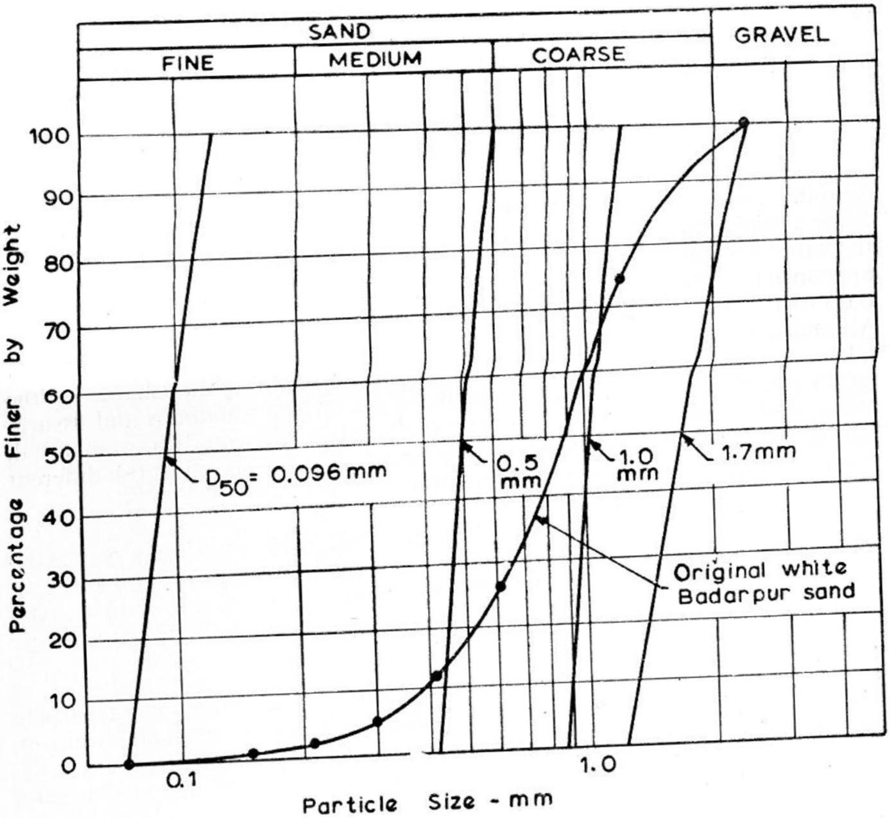

FIGURE 1 : Grain size distribution for sands.

TABLE I

	100% passing B.S. sieve	100% retained B.S. sieve	Mean diameter mm	C_u	Sp. Gr.	Hard-ness	Round-ness	Sphericity
Coarser sand	7	14	1·7	1·3	2·66	6	0·1	0·69
Coarse sand	14	18	1·0	1·2	2·66	6	0·1	0·65
Medium sand	25	36	0·5	1·2	2·66	6	0·3	0·79
Fine sand	120	200	0·096	1·3	2·66	6	0·5	0·83

Experimental Investigations

More than 350 tests were conducted in conventional and high pressure triaxial apparatuses for isotropic consolidation and drained shear tests on the four fractions of Badarpur sand. In addition, a total of 80 tests were conducted to determine tensile strength of individual particles of Badarpur sand.

The procedure for the preparation of specimens for isotropic consolidation and drained shear was similar for the tests conducted in the conventional and high pressure triaxial apparatuses. The test procedure described by Bishop and Henkel (1957) was followed for consolidated drained shear tests. The sands were washed with water to eliminate dust particles and then boiled to remove trapped and dissolved air. The base of the triaxial cell was deaired and a burrette filled with water was connected to the base for measuring volume changes of a specimen during consolidation and shear. A thin rubber membrane was slipped on to the pedestal and sealed with rubber 0-rings. After a sample former was fixed in position, the membrane was filled with deaired water and saturated sand was deposited under water with a spoon. To achieve as far as possible dense specimens, uniform tamping of the former was adopted. To provide stability to the specimen, suction of about 0·1 kg/cm^2 was applied before removing the sample former. Additional membranes whenever required were slipped over the specimen and sealed with rubber 0-rings at the pedestal and cap. After measuring the diameter and height of the specimen, it was subjected to one of the following three series of tests :

Series I : Specimens were subjected to isotropic consolidation under various confining stresses. After consolidation each specimen was sieved by hand to determine changes in grain size distribution.

Series II : Specimens were isotropically consolidated and sheared continuously under drained condition beyond failure or up to a maximum axial strain of 30 percent and then sieved.

Series III: Specimens were isotropically consolidated and sheared under drained condition up to various axial strains of 2, 4, 6, 8, 10, 15, 20, 25 and 30 percent and then sieved.

For all the tests conducted in conventional triaxial cell up to a confining stress of about 11·5 kg/cm^2, a strain rate of 0·254 mm/mt. was used and beyond this stress in the high pressure triaxial apparatus a strain rate of 0·127 mm/mt. was used since the faster rate was not available in the high pressure machine. By using the faster rate in the conventional triaxial apparatus the behaviour of sand would not have been different. For studying the effect of rate of strain under high confining stresses on the magnitude of crushing, strain rates of 0.127 mm/mt. and 0·0127 mm/mt. were adopted.

Individual particles of White Badarpur sand which were nearly spherical in shape having average diameters of 3 mm, 2·2 mm, 1·6 mm and 1·0 mm were selected and one particle at a time was tested in a loading machine using a sensitive proving ring. Twenty similar tests were conducted for each size. In the early stage of loading some flaking was observed but the particles remained intact. At higher load particles shattered into smaller particles with clear audible noise. The load corresponding to shattering of particles was considered as the ultimate load causing crushing. In spite of different initial particle sizes, the process of crushing was similar.

Test Results and Discussions

Mostly, results of coarser and fine sands with mean particle diameters of 1·7 mm and 0·096 mm are presented for comparison, but the observations are based on the behaviour of all the four sands.

Particle Size Distribution

Changes in particle size distribution due to isotropic consolidation and drained shear tests of coarser and fine sands tested at confining stresses ranging from 1·4 kg/cm² to 127 kg/cm² are shown in Figures 2, 3 and 4. Original gradation curves for these sands are also given. All the four sands were initially uniformly graded. Changes in uniformity coefficients (C_u) for the four sands during consolidation and shear are presented in Figure 5. Only coarser sand became well graded ($C_u > 4$) during isotropic consolidation due to particle crushing for confining stress greater than 112·5 kg/cm². When sheared under consolidated drained condition the coarser sand became well graded under confining stresses (σ'_3) greater than 18 kg/cm² ; the other coarse sand under $\sigma'_3 >$ 30 kg/cm² and medium sand under $\sigma'_3 >$ 85 kg/cm². The fine sand remained uniformly graded even when sheared under $\sigma'_3 = 105{\cdot}5$ kg/cm². The coarser sand attained a uniformly coefficient greater than 10 when sheared in drained condition at $\sigma'_3 = 105{\cdot}5$ kg/cm² ; its original value was only 1·30. The uniformity coefficient of fine sand changed from 1·3 to 3·0 when sheared under similar confining stress.

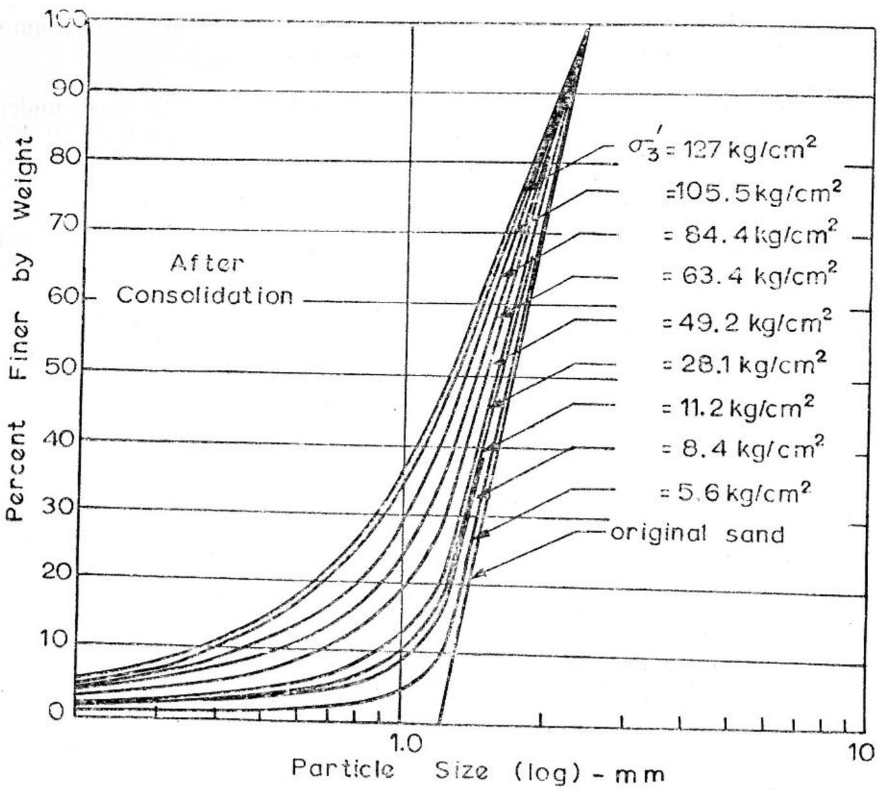

FIGURE 2 : Changes in gradation due to crushing in isotropic consolidation for 1·7 mm size.

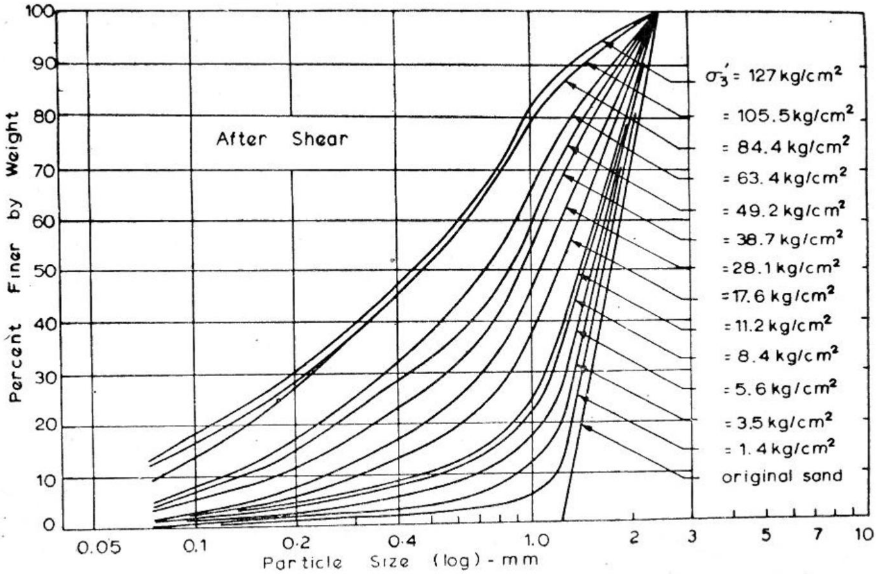

FIGURE 3 : Changes in gradation due to crushing in triaxial compression shear for 1·7 mm size.

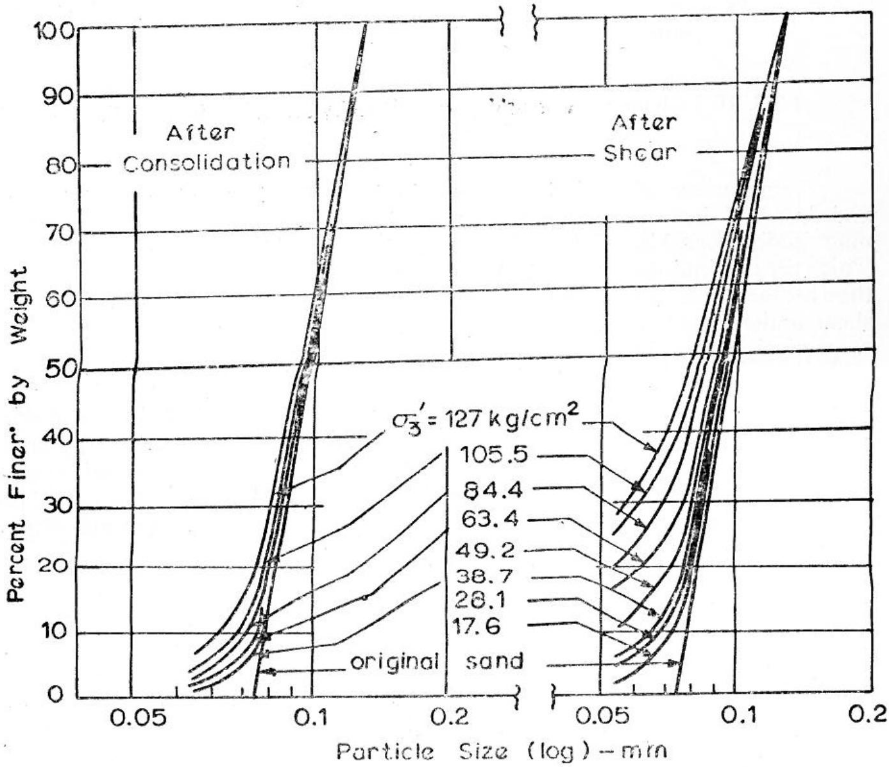

FIGURE 4 : Changes in gradation due to crushing in consolidation and shear for 0·096 mm size.

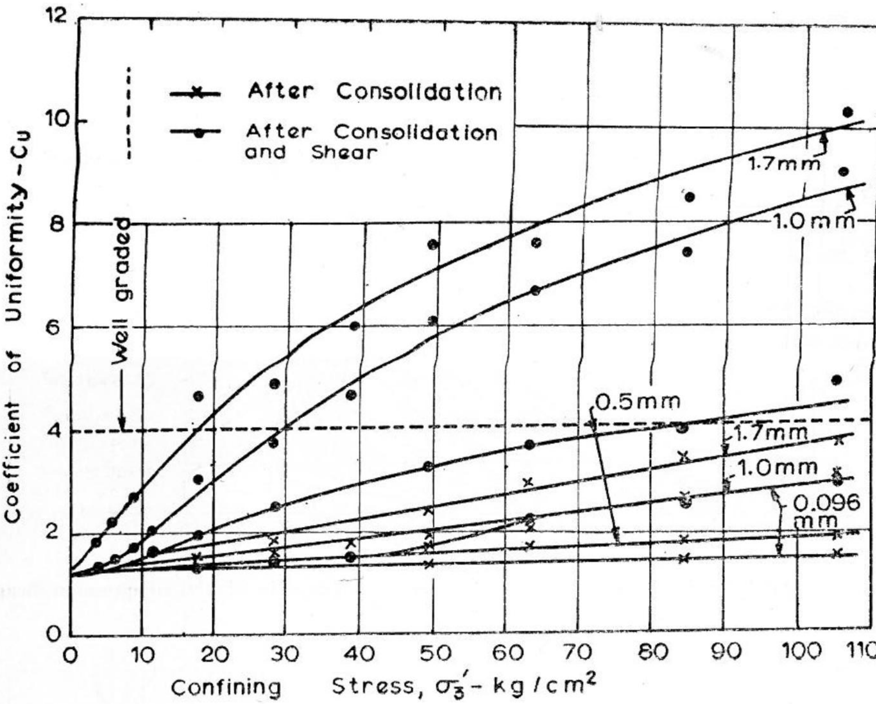

FIGURE 5 : Influence of crushing on uniformity coefficient in consolidation and shear.

Comparison of gradation curves obtained after isotropic consolidation and drained shear suggested greater crushing occurring during shear. The magnitude of crushing, defined by the percentage passing through the sieve on which the original sample was 100 percent retained, was larger in coarser sand than in fine sand. The magnitudes of crushing after consolidation and after shear under $\sigma'_3 = 127$ kg/cm^2 in coarser sand were 48 percent and 90 percent respectively. For fine sand the corresponding values were about 15 percent and 45 percent. Incidentally, the gradation curves for all the sands after isotropic consolidation at 105·5 kg/cm^2 were identical with their gradations after consolidated drained shear tests conducted under $\sigma'_3 = 17{\cdot}6$ kg/cm^2. For producing desired changes in gradation of a coarse grained soil, application of shear stresses will be more desirable than isotropic stresses. The gradation achieved due to crushing after shear under any confining stress is likely to be stable to a large extent since the fragments formed during shear are likely to be harder and have experienced the possible stresses during shear. If a specimen is prepared with a similar gradation without subjecting the particles to prior crushing, it is likely that the grading of the sample may undergo some change.

Figure 6 shows that the magnitude of crushing increases with particle size but the differences in crushing in the coarse sands are negligible. The relation between tensile strength of particles and particle size is also shown. The tensile strength of particles was calculated using

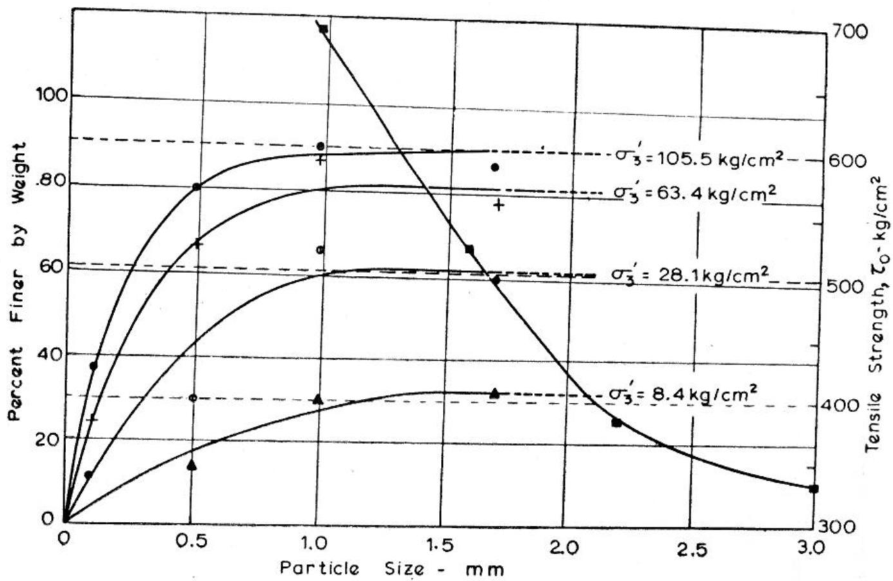

FIGURE 6 : Variation of tensile strength and magnitude of crushing with particle size.

the expression

$$\tau_o = \frac{KW}{r^2} \quad \text{(Jaeger and Cook 1969)}$$

where, K = constant varying from 0·25—0·4 ; lower value (0·25) was used in the calculation of tensile strength (τ_o) of the particles.

W = load causing crushing.

r = radius of particle.

The tensile strength of particles decreased with the increasing size of particles and consequently larger particles experienced greater crushing. Since the original Badarpur sand is weathered, the coarser particles appear to have more fissures and planes of weakness than finer fractions ; as a result coarser fractions indicated lower tensile strength and larger crushing during consolidation and shear.

Results of the magnitude of crushing in coarser and fine sands sheared for different axial strains up to confining stress of 127 kg/cm² are presented in Figures 7 and 8. The magnitude of crushing increases continuously with axial strain. Up to about 10 percent axial strain crushing takes place rapidly and becomes slower under larger strains. Figures 9 and 10 show percentages passing through finer sieves for coarser sand after it had been shear under various confining stresses. The fines formed during consolidation are considerably smaller than during shearing. During shear larger particles appear to undergo continuous crushing producing finer and

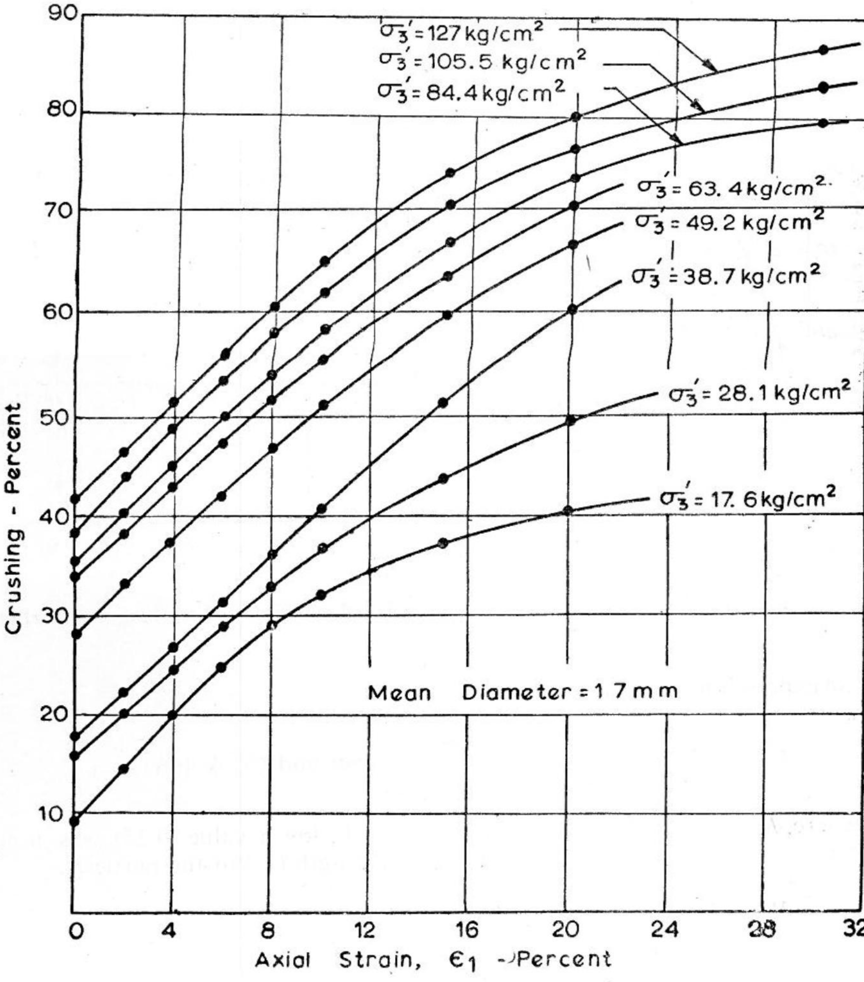

FIGURE 7 : Influence of axial strain on crushing for 1·7 mm size.

finer fraction indicating a change in the "quality" of crushing. Similar variations were observed in other sands. In all the four sands crushing during consolidation was found to be ranging between 35 to 48 percent of the total crushing; lower crushing was found in fine sand. Generally, coarser fractions were formed during consolidation and finer fractions were mostly the results of shearing. The process of fragmentation appears to be continuous with axial strain.

The changes in specific surface area produced during crushing will be the result of the magnitude of energy involved in crushing. The total energy imparted to a specimen is utilized in producing shear at inter-particle contacts, in crushing the particles and in changing the boundaries of the specimen. The energy involved in changing the boundaries of the specimen could be easily separated but the estimation of energies involved in crushing

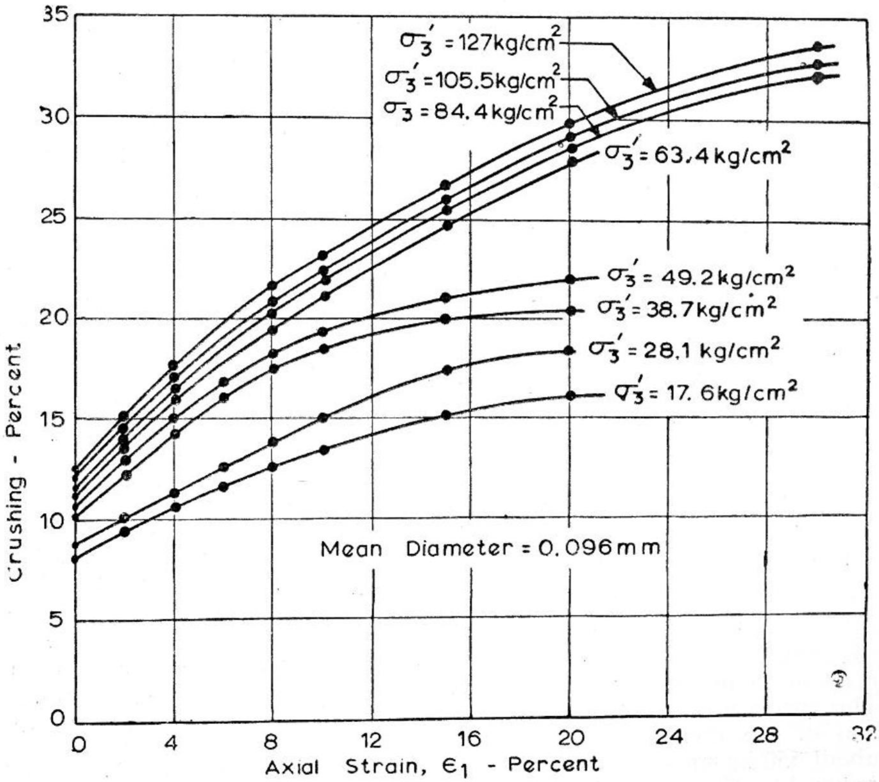

FIGURE 8 : Influence of axial strain on crushing for 0·096 mm size.

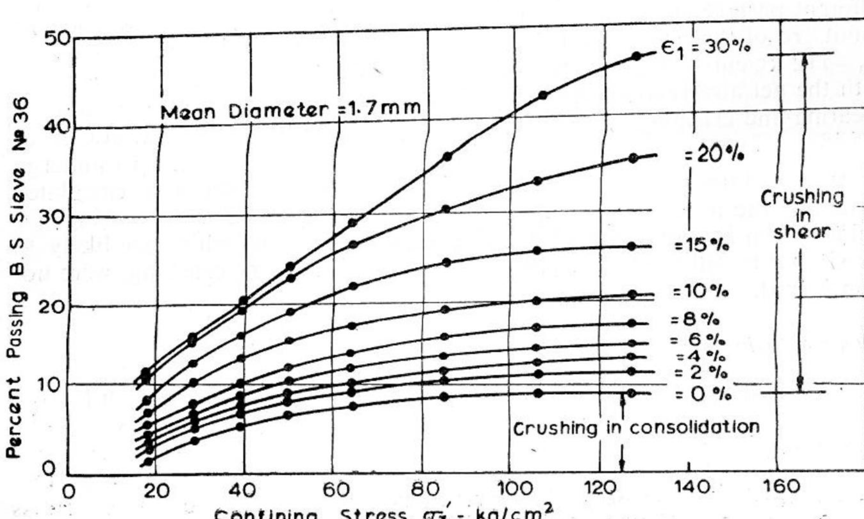

FIGURE 9 : Comparison of fines formed during consolidation and shear.

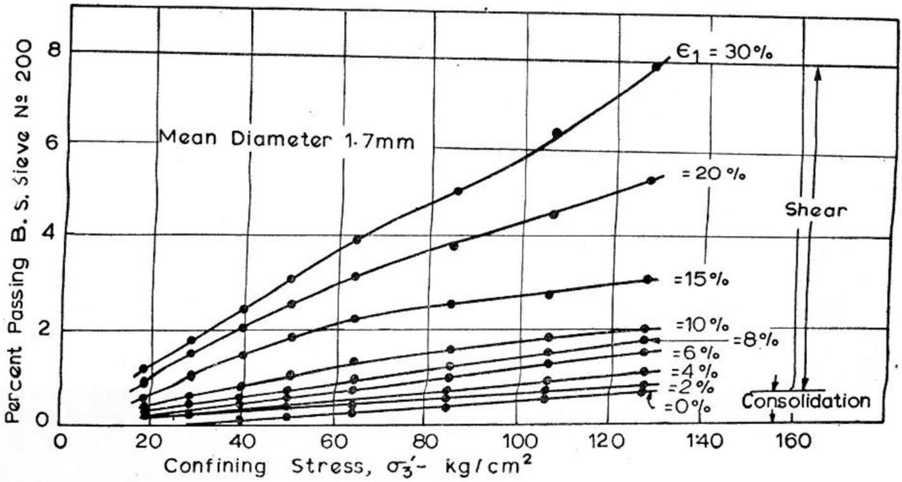

FIGURE 10 : Comparison of fi1es f3rm3d d1ring co1solid1tion and sh3ar.

and shearing separately, could not be done with the present theories. The energy absorbed (net) in shearing and crushing was calculated for relating with the changes in diameter corresponding to 50 percent finer by weight (d_{50}) and with specific surface area. Figure 11(*a*) shows the variation of d_{50} with the net energy imparted to the specimens of all the four sands. This energy was calculated from the stresses and strains developed at the end of the shear. When the sands were sheared under a net energy of about 550 kg.-cm/cm² (up to 30 percent axial strain), d_{50} of the coarser sand reduced to 27 percent of its original value, while that of the other coarse sand to 40 percent, medium sand to 50 percent and fine sand to 83 percent of their original d_{50}. For similar energies adsorbed coarser sand experienced considerable reduction in mean diameter than the fine sand. Lo (1969) observed from his tests on materials of similar particle size but having different particle strength that the geometric mean diameter reduced to about ⅓rd of the original values when the specimens were sheared under $\sigma'_3=112$ kg/cm². Figure 11(*b*) shows the changes in specific surface area with the net absorbed energy. For similar values of energy involved in shearing and crushing all the four sands undergo similar variations in specific surface area. This would mean that coarser sands have to undergo greater crushing than fine sand. The specific surface area was calculated assuming the particles to be spherical between successive sieves using the values given by Loudon (1952). The angularity factors which are likely to be similar for all the four sands undergoing considerable crushing, were not considered.

Stress-Strain Curves

The stress-strain curves for coarser and fine sands are given in Figures 12 and 13. The stress-strain curves become steeper with confining stress suggesting increasing initial tangent modulus. The specimens which showed axial strain at failure varying from 6 percent to 7 percent at $\sigma'_3=1{\cdot}4$ kg/cm² did not reach failure even at 30 percent axial strain under confining stress of 105·5 kg/cm². Figure 14 shows the variation of axial strain at failure with confining stress. Points corresponding to axial strain at failure for

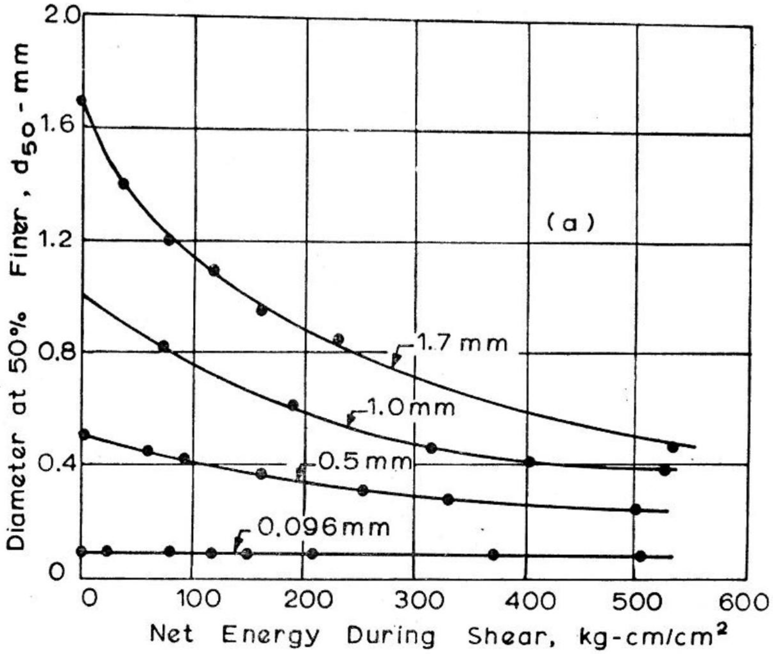

FIGURE 11 (a) : Changes in mean diameter due to net energy imparted during shear.

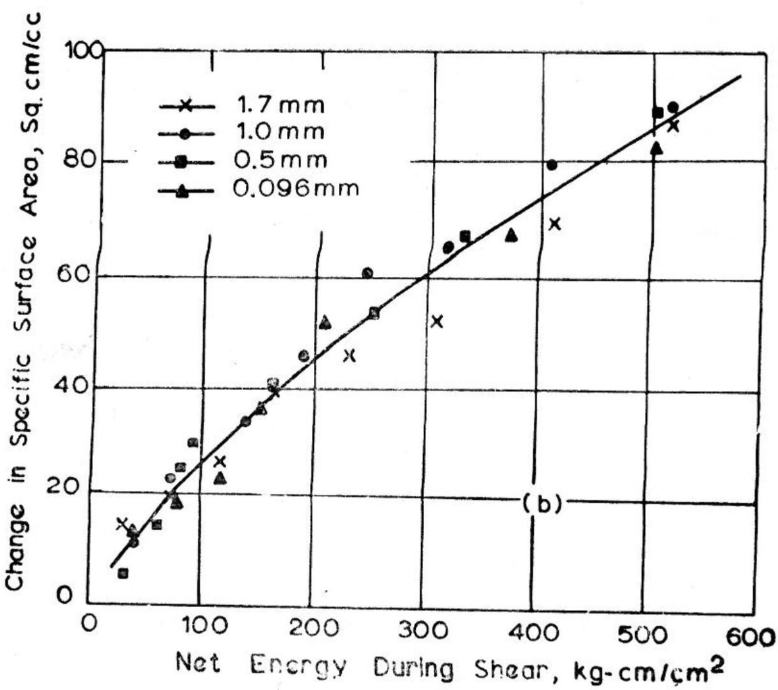

FIGURE 11 (b): Changes in specific surface area due to net energy imparted during shear

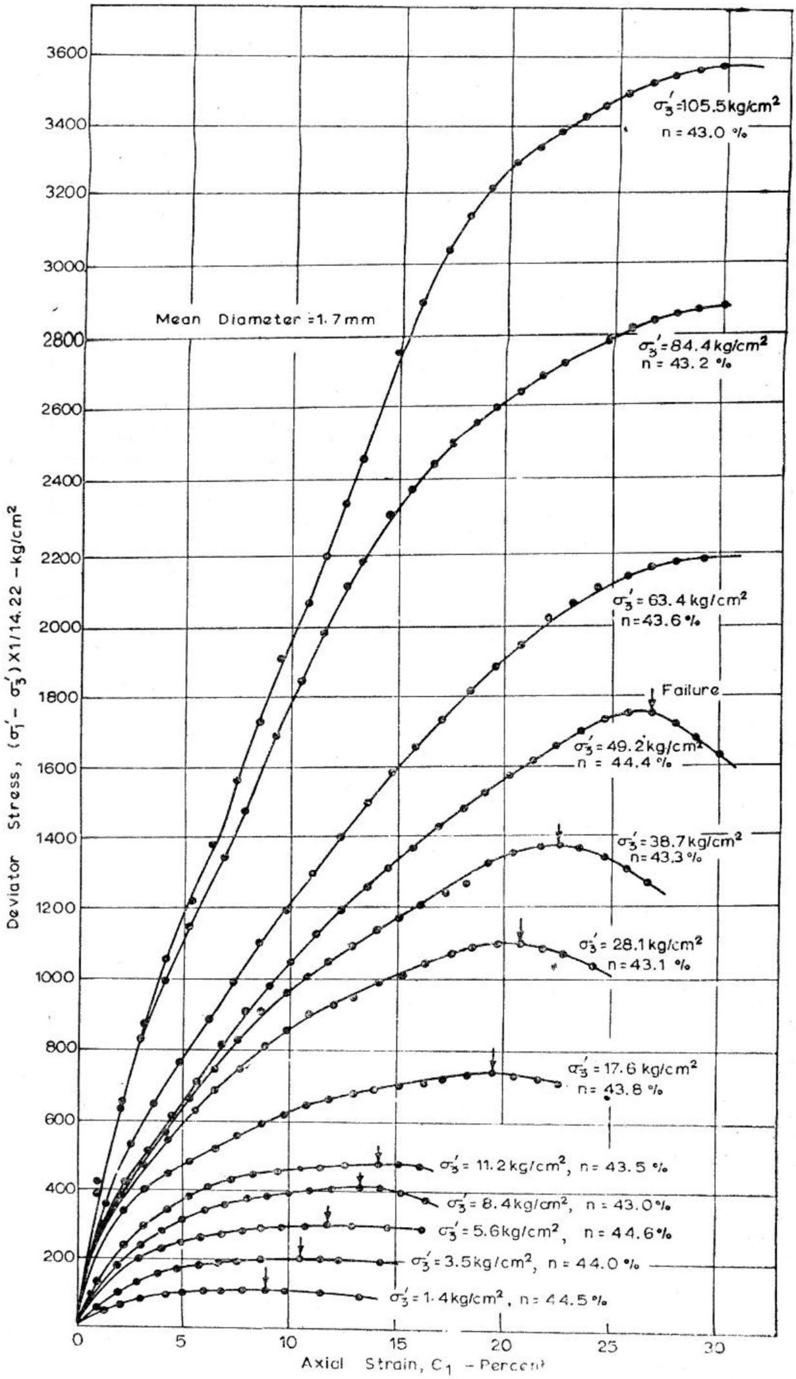

FIGURE 12 : Stress-strain curves for 1·7 mm size.

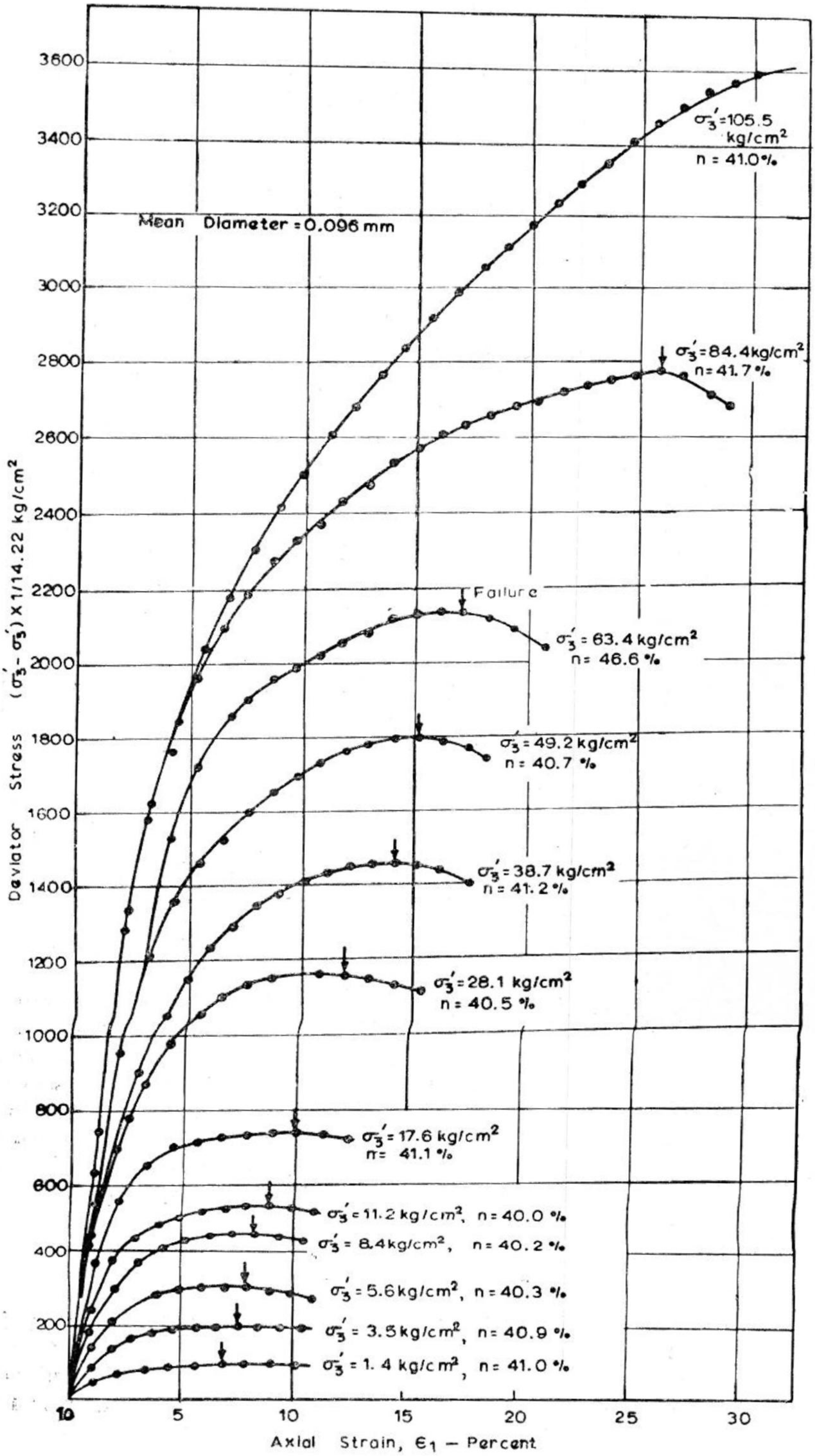

FIGURE 13 : Stress-strain curves for 0·096 mm size.

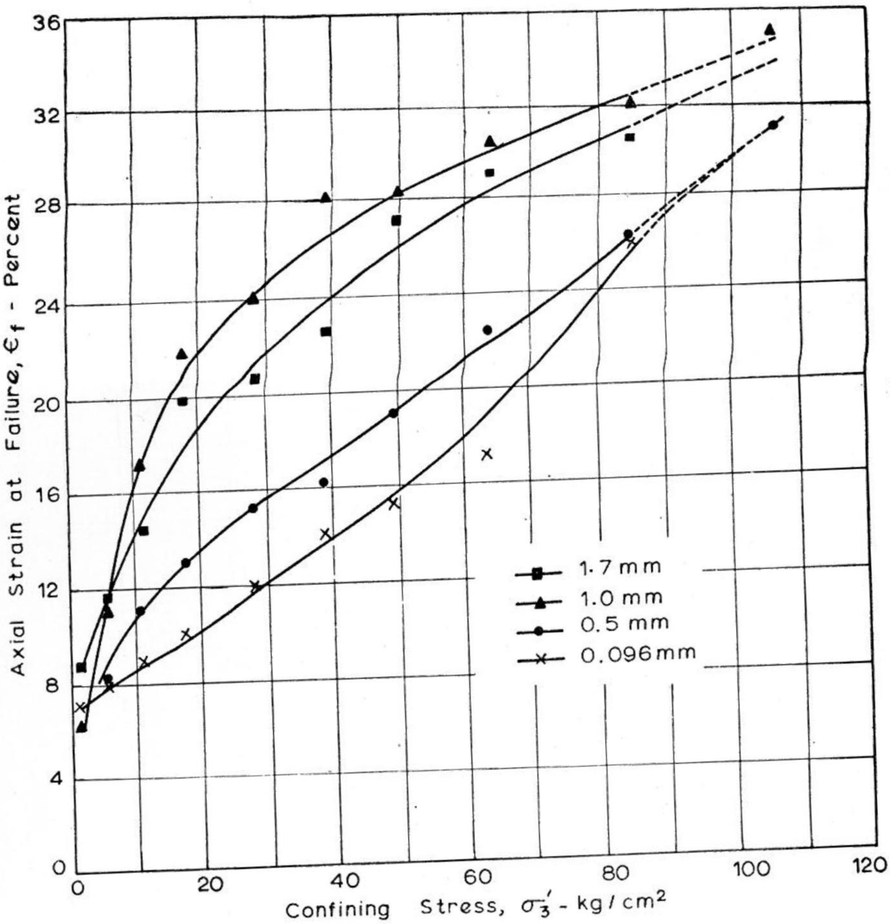

FIGURE 14 : Influence of crushing on axial strain at failure.

$\sigma'_3=84{\cdot}4$ kg/cm² and $\sigma'_3=105{\cdot}5$ kg/cm² were extrapolated from stress-strain curves. The coarser sand showed lower failure strains than the other coarse sand. Fine and medium sands failed at lower axial strains than coarse sand since they experienced lower magnitude of crushing. The crushed material, which by remaining at the inter-particle contacts behaves as a weaker and more compressible material or may be displaced into voids during shear, produced larger strain indicating increasing plastic nature of failure with increasing crushing under higher confining stresses.

Volumetric Strain

The results of volumetric strain during shear are presented in Figures 15 and 16 for coarser and fine sands. The coarser sand exhibited a net volume expansion of 4.5 percent at an axial strain of the order of 12 percent when the sand was sheared under a confining stress of 1·4 kg/cm². When a specimen of this sand sheared under 105·5 kg/cm² it exhibited a net volume contraction of about 19 percent at an axial strain

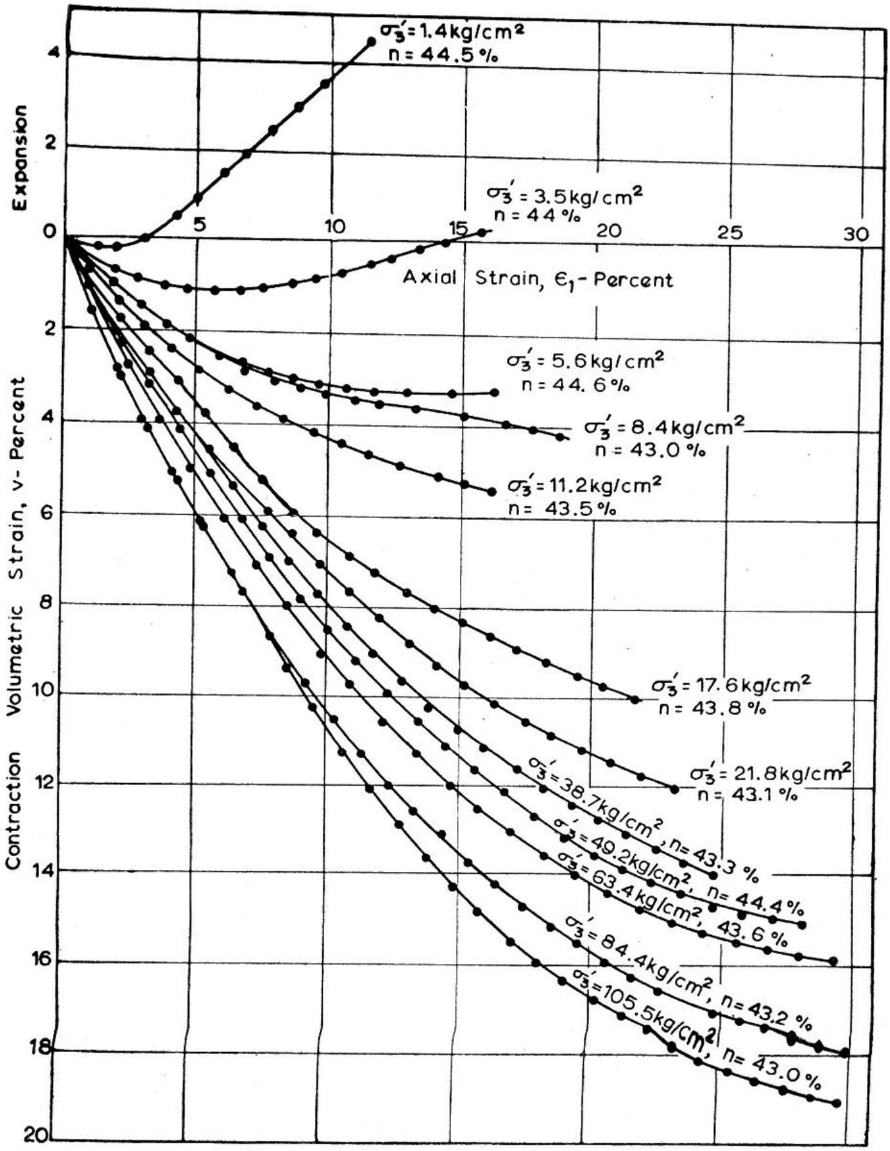

FIGURE 15 : Influence of crushing on volumetric strain during shear for 1·7 mm size.

of 30 percent. The specimen did not indicate any tendency to dilate. The fine sand showed a net volume expansion of about 6 percent at an axial strain of 12 percent when sheared under $\sigma'_3 = 1\cdot4$ kg/cm^2. A similar specimen when sheared under $\sigma'_3 = 105\cdot5$ kg/cm^2 underwent 12 percent volume contraction at 30 percent axial strain. The volume contraction in fine sand under high confining stress was less than that experienced by the coarser sand due to lower crushing. Due to crushing, the dilatant behaviour of specimen tested at low confining stresses disappeared at high stresses and significant volume contraction of specimens occurred during shear. Negative Poisson's ratio was observed for coarse sand during

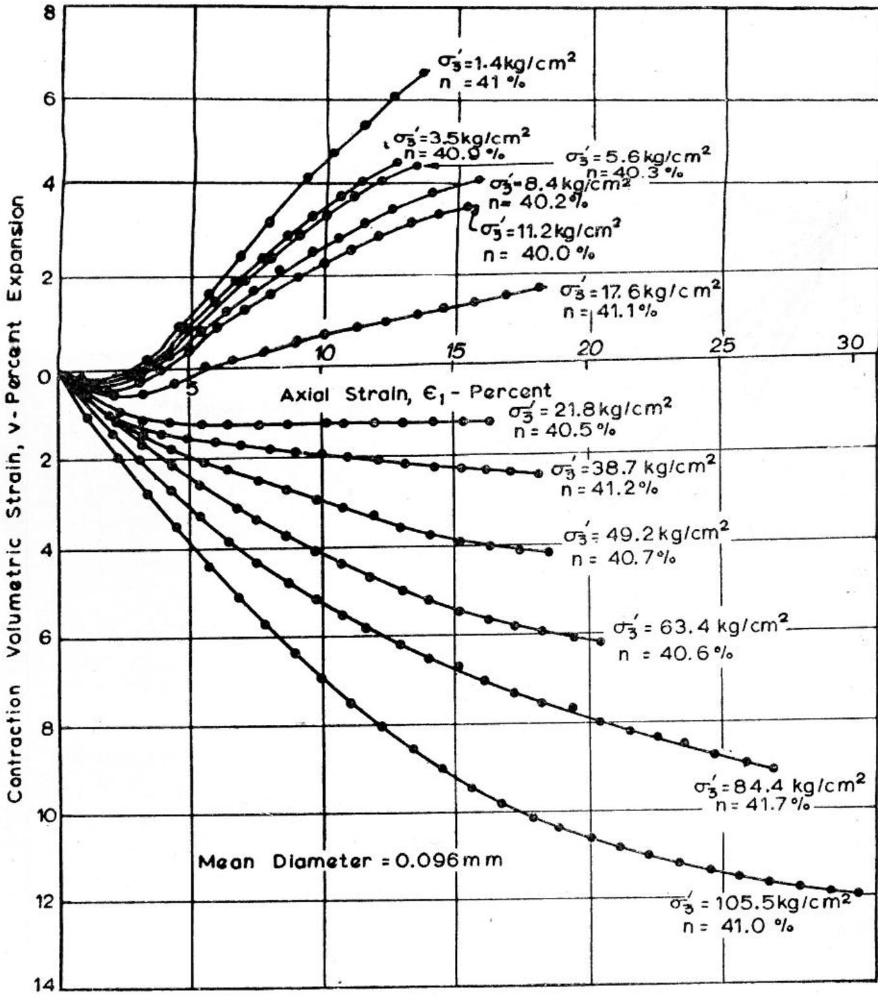

FIGURE 15 : Influence of crushing on volumetric strain during shear for 0·096 mm size.

early stages of shearing under confining stresses greater than about 60 kg/cm².

Volumetric strain during isotropic consolidation for all the four sands after separating the elastic compression of sand and membrane penetration (Ramamurthy and Kanitkar, 1972) are shown in Figure 17. The fine sand which showed lower crushing during isotropic consolidation showed volumetric contraction of 0·3 percent and 4·5 percent for confining stress of 1·4 kg/cm² and 84·4 kg/cm² respectively. The corresponding values for coarser sand were 0·8 percent and 16 percent.

Shear Strength

Mohr rupture envelops and envelops to stress circles for various axial strains for coarser and fine sands are presented in Figures 18 and 19.

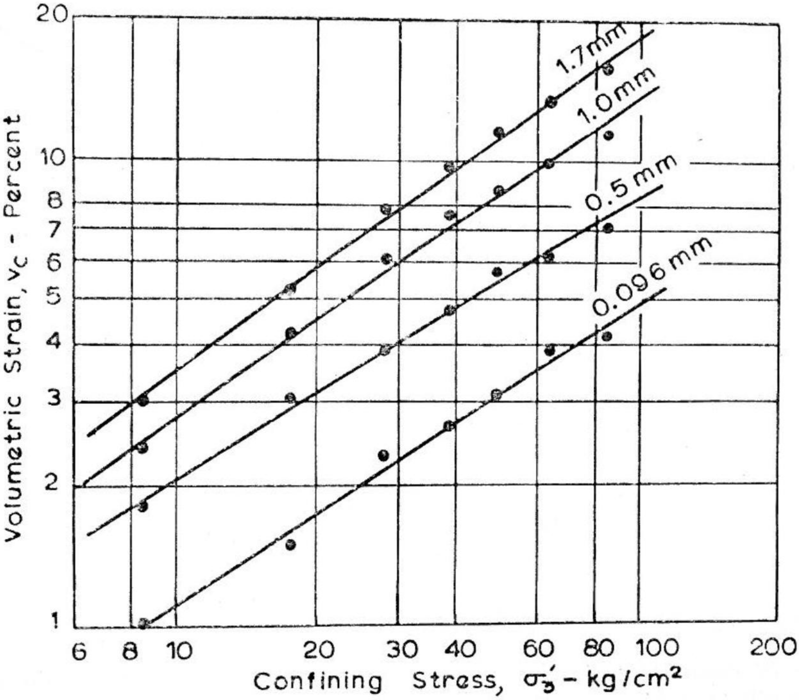

FIGURE 17 : Volumetric strain due to consolidation alone.

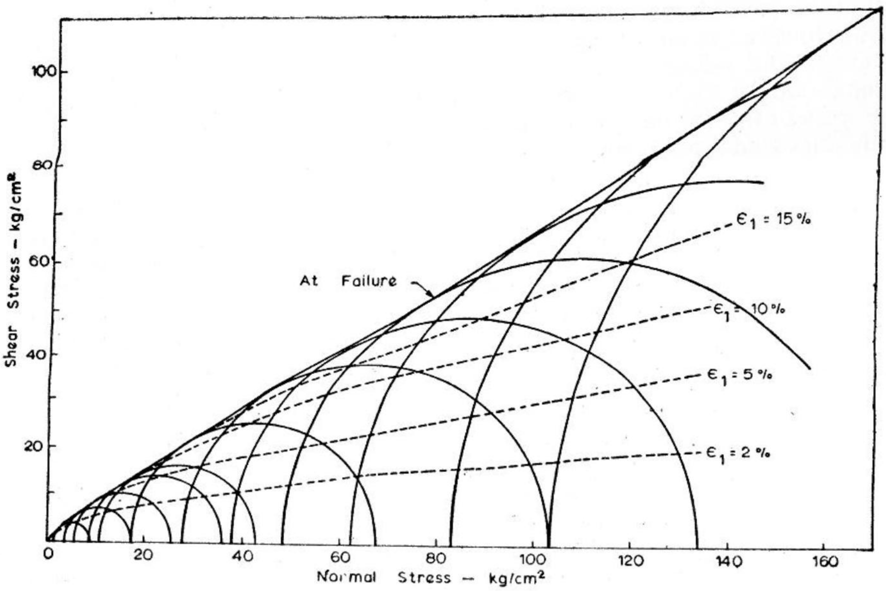

FIGURE 18 : Mohr rupture envelope for 1·7 mm size.

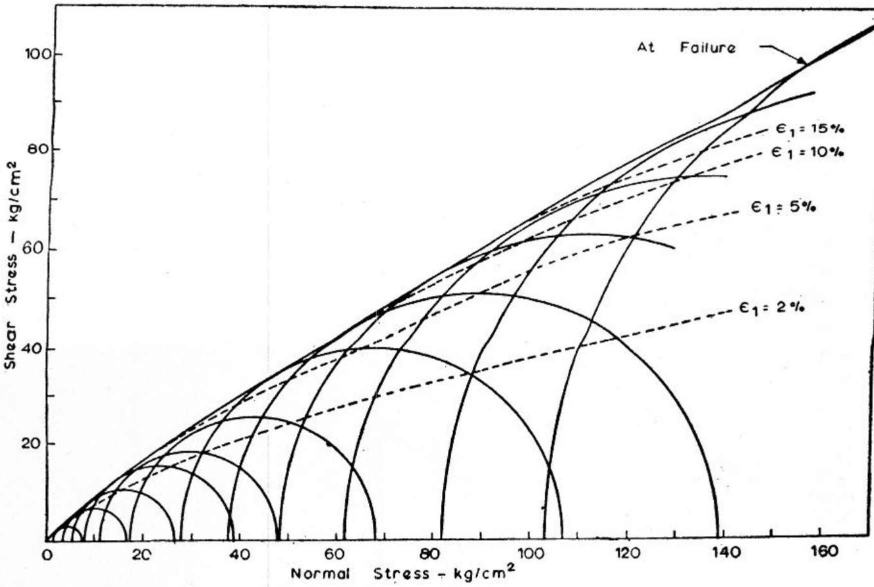

FIGURE 19 : Mohr rupture envelope for 0·096 mm size.

The envelops corresponding to 2, 5, 10 and 15 percent axial strains are curved throughout. By comparing these curves, it was observed that in fine sand shear strength was generated at a faster rate with strain than in coarser sand. The rupture envelops are curved up to 28·0 kg/cm² confining stress and bccome straight afterwards. The values of the effective angle of shearing resistance (ϕ') mobilized at 2 and 5 percent axial strain for coarser sand during shear under 105·5 kg/cm² confining stress were only 10° and 16° respectively. Similar values for fine sand were 16·6° and 23°. However, at larger axial strains, these differences were small. For example, the values of ϕ' for coarser sand at 10 and 15 percent axial strain for σ'_3 = 105·5 kg/cm² were 23·3° and 28·6°. The corresponding values of ϕ' for fine sand were 27.7° and 29.3°. But at failure, all the four sands indicated very nearly similar values of ϕ' for σ'_3 greater than 28·0 km/cm², even though these sands had different particle strength and size. Results of Hirschfeld and Poulos (1963), Hall and Gordon (1963), Bishop, Webb and Skinner (1965) suggested curved rupture envelops throughout the stress range investigated. The curved rupture envelops were observed by Vesic and Barksdale (1963) up to σ'_3=63·3 kg/cm². The four fractions of Badarpur sand seem to have lower particle strength than those tested by Vesic and Barksdale and underwent significant crushing up to a confining stress of 28.0 kg/cm² and as a consequence of this they have exhibited marked reduction in effective angle of shearing resistance. The values of ϕ' at failure under σ'_3=1.4 kg/cm² dropped from a value of 47° to 34° under σ'_3=28.0 kg/cm². The maximum reduction in ϕ' between 28·0 kg/cm² and 105·5 kg/cm² was only of the order of 2°. The particle size distribution of the sands sheared under σ'_3=28·1 kg/cm² suggested that the formation of about 2 percent fines (passing through 200 mesh) for coarse sands, about 4 percent fines in medium sand and about 8 percent fines in fine sand was sufficient to bring about marked changes in the angle of shearing resistance. Formation of more fines for confining stresses greater than

28·0 kg/cm² did not contribute in decreasing ϕ'. The values of ϕ' for the four sands were between 44° and 46°, at low confining stresses but under high stresses these values were between 32·4 to 33°, in spite of the initial differences in particle strength, particle size and the magnitude of crushing. Even the differences in initial porosities of these sands did not influence the values of ϕ' for σ'_3 greater than 28.0 kg/cm².

Stress-Ratio and Void Ratio Relations

The variations of average void ratio of specimens during shear are represented with effective stress ratio for coarser and fine sands in Figures 20 and 21. The behaviour is unlike that observed at low confining stresses. Specimens tested at minimum void ratio (e_{min}) at low confining stresses expand during shear and the curves shift to the right whereas the specimens tested at maximum void ratio (e_{max}) move towards the left. The peak effective stress ratio decreases as void ratio increases from e_{min} to e_{max} as shown by *AB* in Figure 20(*a*). When a dense specimen tested at e_{min} under high confining stress would contract due to crushing and the effective stress ratio-void ratio curve shifts to left similar to that of a loose specimen and the envelop under high stress would be as shown by *BC* in Figure 20(*a*). The envelop of effective stress ratio and void ratio for low to high confining stresses for a granular soil undergoing crushing may be as indicated by *ABC* in Figure 20(*b*). In the calculation of average void ratio during shear, it was assumed that the specimens deformed as right cylinders. The effective stress ratios at failure for the four sands were

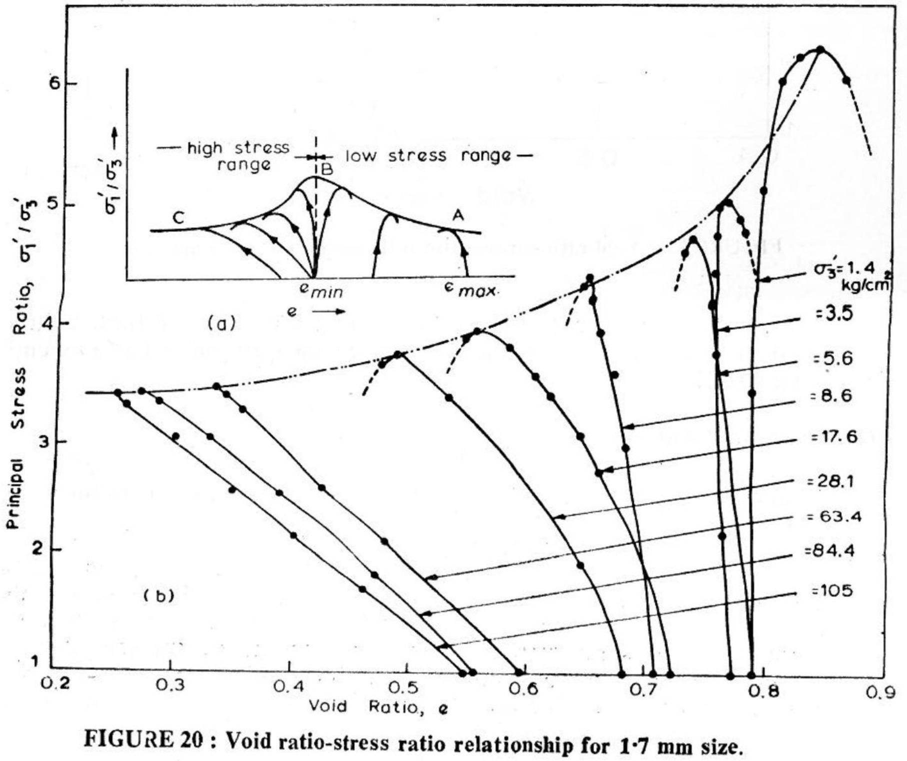

FIGURE 20 : Void ratio-stress ratio relationship for 1·7 mm size.

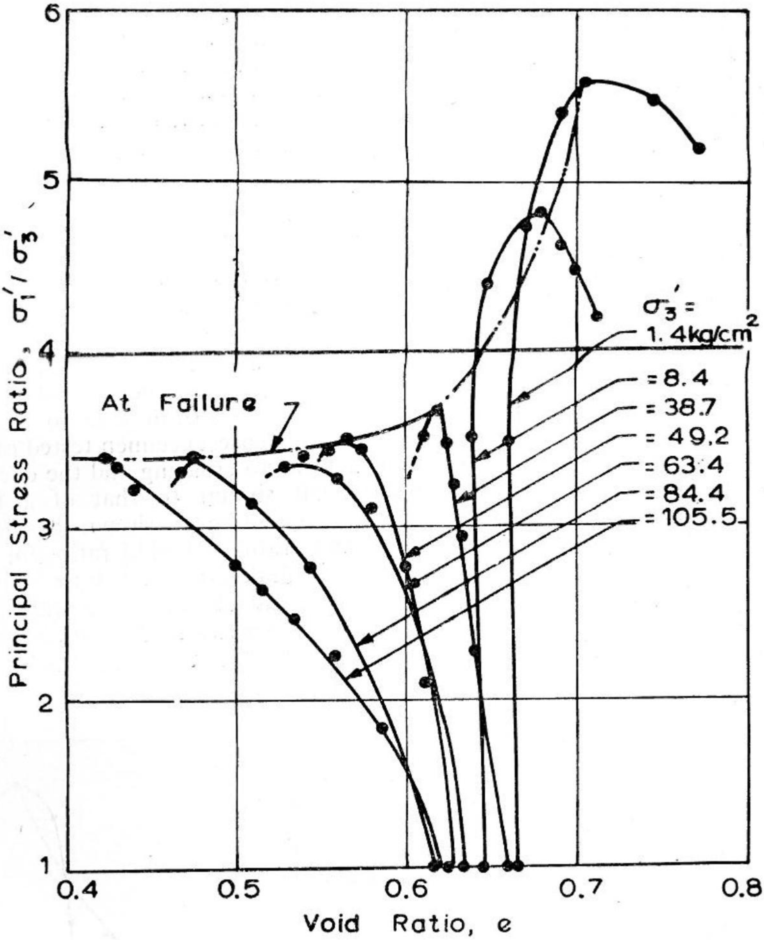

FIGURE 21 : Void ratio-stress ratio relationship for 0·096 mm size.

between 5·6 and 6·3 for confining stress of 1·4 kg/cm² but these values decreased to lie between 3·3 and 3·4 when sheared under 105·5 kg/cm² confining stress.

Dilatancy Rate and K_o

The dilatancy rate, $\left(1+\frac{\delta_v}{\delta\varepsilon_1}\right)$, which was about 1·8 at failure for σ'_3 = 1·4 kg/cm² decreased rapidly up to a confining stress of 28 kg/cm² and thereafter remained almost constant having a value between 0·7 and 0·85 for all the four sands. In spite of the initial differences in size and particle strength these sands have not shown any significant differences in their behaviour of particulate structure. Possibly the particles formed due to crushing in coarse sands were stronger than the parent particles and the fractions common to both coarse and fine sands appear to control the overall behaviour of the sands.

The coefficient of earth pressure at rest (K_o) was calculated using the effective angle of shearing resistance at failure for all the sands from Jáky's expression (1944). Its values increased rapidly from $\sigma'_3 = 1{\cdot}4$ kg/cm² to $\sigma'_3 = 28$ kg/cm² and beyond this stress they did not vary significantly. At $\sigma'_3 = 1{\cdot}4$ kg/cm² the values of K_o for all the four sands varied from 0·25 to 0.28 but at $\sigma'_3 = 105{\cdot}5$ kg/cm² they were between 0·41 to 0·42.

Strain Rate

A limited number of experiments conducted to study the influence of rate of strain suggested that crushing increased with increasing rate of strain. Figure 22 shows the results of the magnitude of crushing for two rates of

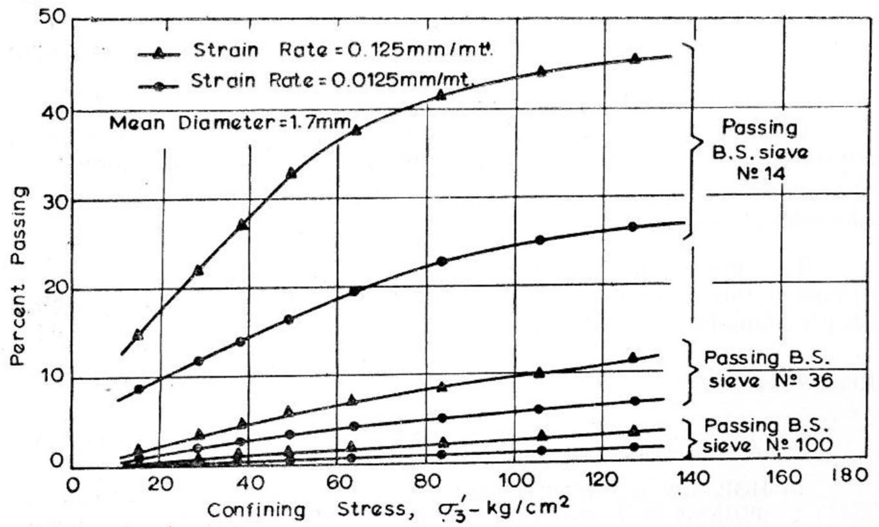

FIGURE 22 : Influence of strain rate on crushing.

strains, namely, 0·125 mm/mt. and 0·0125 mm/mt. for axial strains of 2 percent in coarser sand. One would infer that under sudden application of stresses the magnitude of crushing is likely to be larger than due to the slow application of static stresses. During rolling larger amount of crushing is likely to occur than due to the embankment load which generally increases at comparatively slower rate. To simulate field crushing in the laboratory shear tests, rate of strain may have to be suitably choosen to obtain relevant design data.

Conclusions

Isotropic consolidation and consolidated drained shear tests on four fractions of Badarpur sand having different particle sizes and particle strengths when tested up to confining stress of 127 kg/cm² have revealed that the coarser fractions crush more than the finer fractions. To convert a uniformly graded sand into a well graded one, shearing would bring about greater changes in grading than the application of isotropic stresses. Degradation continues to occur with axial strain. During shear mostly the quality of crushing undergoes change by producing more and more finer fractions. The magnitude of crushing is not only a function of confining stress but also of tensile strength of particles, axial strain and the rate at which the soil is loaded. For similar magnitudes of energies absorbed by specimens coarser sands undergo greater reduction in mean diameter and crush more than fine sand such that the changes in the specific surface area are similar. Due to large magnitude of particle crushing coarser sand mobilizes strength at a slower rate with axial strain than fine sand. But at large axial strains when sufficient degradation has occurred, the strength of all the four sands are almost same, in spite of their initial differences in porosity, particle size and particle strength. Because of particle crushing the stress-strain relationship, mode of failure, dilatancy rate, coefficient of earth pressure at rest, and effective stress ratio at failure undergo considerable change. The Mohr rupture envelope is curved up to a confining stress of 28 kg/cm^2 when significant crushing occurred and beyond this stress the rupture envelope is straight line although crushing was continuous. But the Mohr envelopes drawn for lower axial strains show marked curvature throughout and axial strain influences the shapes of the envelopes.

In the design of filters and earth dams a study of the influence of crushing on the behaviour of granular soils should be a consideration.

Aknowledgement

The authors are grateful to the Director, I.I.T., Delhi for permitting to publish this paper and to the Head of the Civil Engineering Department for providing laboratory facilities.

References

BISHOP, A.W. (1966) : "The Strength of Soil as Engineering Materials". *Geotechnique,* 10 : 2 : 89-128.

BISHOP, A.W. and HENKEL, D.J. (1957) : "The Measurement of Soil Properties in the Triaxial Test". Edward Arnold Ltd., London, 2nd Edition.

BISHOP, A.W.; WEBB, D.L. and SKINNER, A.E. (1965) : "Triaxial Tests on Soil at Elevated Cell Pressures". *Proc. 6th Int. Conf. Soil Mech. and Found. Engg.* 1 : 170-174.

HALL, E.B. and GORDON, B.B. (1953) : "Triaxial Testing with Large Scale High Pressure Equipment". *Sym. Laboratory Shear Testing of Soils,* ASTM, S.T.P. 361, 315-328.

HIRSCHFELD, R.C. and POULOS, S.J. (1963) : "High Pressure Triaxial Tests on a Compacted Sand and Undisturbed Silt". *Sym. Laboratory Shear Testing of Soils,* ASTM, S.T.P. 361, 329-341.

JAEGAR, J.C. and COOK, N.G.W. (1969) : "Fundamentals of Rock Mechanics". Metheum and Co. Ltd. London, 160-167.

JAKY, J. (1944) : "The Coefficient of Earth Pressure at Rest." *Jnl. Soc. of Hungarian Architects and Engineers,* 355-358.

LEPS, T.M. (1970) : "Review of Shear Strength of Rockfill". *Proc. Am. Soc. Civil Engrs.,* 96 : SM 4 1159-1170.

LO, K.L. (1969) : Discussion, *Proc. 7th Int. Conf. Soil Mech. and Found. Engg.,* 3 : 192-193.

MARACHI, N.D., CHAN, C.K. and SEED, H.B. (1972) : "Evaluation of Properties of Rockfill Materials". *Proc. Am. Soc. Civil Engrs.,* 98 : SM 1 : 95-114.

MARSAL, R.J. (1963) : "Progress Report on Triaxial Tests run with Granular Soils and Rockfill Materials". Jan.-May Report.

MALSAL, R.J. (1967) : "Large Scale Testing of Rockfill Materials". *Proc. Am. Soc. Civ. Engrs.*, 93 : SM 2 : 27-43.

RAMAMURTHY, T. (1969) : "Crushing Phenomena in Granular Soils". *Jnl. Ind. Nat. Soc Soil of Mech. and Found. Engg.* 8 : 1 : 67-86.

RAMAMURTHY, T. and KANITKAR, V.K. (1972) : "Elastic Compression of Sands Under High Isotropic Consolidation Stresses". *Sym. on Strength and Deformation Behaviour of Soils*, Bangalore, 1 : 115-120.

RAMAMURTHY, T. and LALL, R.S. (1970) : "Influence of Crushing on the Properties of Badapur Sands". *Jnl. Ind. Nat. Soc. of Soil Mech. and Found. Engg.* 9: 3: 305-322.

RITTENHOUSE, G. (1943): "A Visual Method of Estimating Two-dimensional Sphericity". *Jnl. Sedi. Petro.*, 13 : 2 : 79-81.

THOMPSON, T.F. (1971) : Discussion on "Review of Shear Strengths of Rockfill" by Leps, *Proc. Amer. Soc. Civ. Engrs.* 97 : SM 1 : 277-280.

VESIC, A.S. and BARKSDALE, R.D. (1963) "On Strength of Sand at Very High Pressures". *Sym. Laboratory Shear Testing of Soils*, ASTM, S.T.P. 361, 301-305.

VESIC, A.S. and CLOUGH, W. (1968): "Behaviour of Granular Material Under High Stresses." *Proc. Am. Soc. Civ. Engrs.* 94 : SM 3 : 661-688.

WADELL, H. (1935) : "Volume, Shape and Roundness of Quartz Particles." *Jnl. Geo.* 43 : 3 : 250-280.