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Static Behaviour of Artificially Cemented Sand

by

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Introduction

Whenever soils with unsatisfactory properties are encountered on engineering projects, some form of soil stabilization is required. Many engineering projects, such as improvements of subgrades under highways and airport runways, stabilizing slopes in cuts and embankments, increasing soil bearing capacity under foundations etc. need stabilization.

There are many methods of soil stabilization. The description, merits and demerits of commonly used stabilization techniques are discussed in detail elsewhere (MIT, 1952; USNAEC, 1969, ASCE, 1982, etc.) and are not reported here. To stabilize the weak sandy deposits, the method of artificial cementation is becoming increasingly popular. The addition of a small amount of cementing material such as portland cement substantially improves engineering properties of sands.

The studies of cemented sands were performed at M.I.T. (Wissa, et al., 1964, 1965) involving only static triaxial tests. Bachus et al, (1981), Shafi-Rad and Clough (1982) and Sitar et al. (1980) investigated the behaviour of weakly cemented sands for static loading conditions and for soil slopes under earthquake conditions. Saxena and Lastrico (1978) and Dupas and Pecker (1979) studied the static properties of naturally cemented sands.

The present study is limited to artificially cemented sand with the following objectives :

(*i*) to briefly review the previous studies

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(*ii*) to increase the data base for tensile strength and to correlate it with unconfined compressive strength.

(*iii*) to analyze the results of triaxial drained tests with cemented-stabilized Monterey No. θ sand to obtain an understanding about

(*a*) Strength generation

(*b*) Initial tangent modulus

(*c*) Stress-strain characteristics

Previous Studies

Wissa and Ladd (1964, 65) were the first to study the properties of compacted stabilized soils (artificially cemented). They used two types of coarse soils; one coarse Ottawa uniform sand which entirely passed sieve #20 and 95 percent retained on sieve #30. The second was a medium Ottawa sand obtained by sieving well graded Ottawa sand and using the portion passed through sieve #40 and retained on sieve #60. The first type of soil has maximum and minimum dry densities of 1.78 and 1.541 g/cc respectively, while the second type had maximum and minimum dry densities of 1.716 and 1.44 g/cc respectively. These dry densities of sand portion were obtained by air pluviation technique. The relative density of the samples tested were; Coarse Ottawa sand—43 percent and medium Ottawa sand—62, 64 and 75 percent. The samples tested were 8 cm in length and 3.57 cm in diameter. The stabilizer used was Portland cement Type 1. For coarse sand 5 percent stabilizer by dry weight of sand was used and for medium sand two proportions 5 and 10 percents by dry weight were used. The exact weight of any sand for one sample was handblended with the appropriate amount of cement. The mixing water was then added and mixed thoroughly by hand. The samples were compacted in two part split mould in 10-15 layers using 10 soft tamping per layer applied with 12.5 mm diameter tamper. The samples were first humid cured in desicant jars for three days and then were completely immersed in water for at least 24 hours before testing.

The samples were saturated under a back pressure of 10 kg/cm^2 for two hours and saturation was considered 100 percent when Skempton's B parameter was at least 0.90. Deaired water was used to saturate the samples. The samples were subject to consolidated undrained and consolidated drained triaxial tests. All tests were strain controlled with strain rate of 6 percent per hour. Final water content was determined at the end of the test. In the drained tests volume changes during shear were measured under the back pressure of 10 kg/cm^2 to the nearest 0.01 percent. The total number of tests conducted were 27 on sands and 107 on clays.

The influence of cementation was studied based on CID triaxial compression tests with Ottawa sand stabilized with different cement content and curing periods. Unlike uncemented sand, the cemented sand was found to cause Mohr's envelope with cohesion intercept and appreciable curved at lower confining pressures due to premature brittle fracture caused by inadequate confining pressure to close the submicroscopic shrinkage cracks due to hydration during curing. For example, a consolidation pressure of 10 kg/cm^2 was sufficient to avoid brittle fracture for the case of medium dense sand with 5 percent cement content. This value, however was greater for higher cement contents.

Axial strain contours, indicate that at low strains the shearing resistance was due to the cementation between grains and no appreciable friction is mobilized. After about 0.6 percent axial strain the frictional resistance continued to increase and the cementation gradually broke down. On further straining ultimate conditions were reached at which time the continuous cementation between grains in the failure zone was completely destroyed and the effective stress-strength curve converged towards the origin on a p vs q plot. The maximum principal strength difference was found to occur when the sum of the shearing resistance due to friction and cementation reached a maximum. At this time, the slopes of the volumetric strain versus axial strain curve did not reach a maximum.

The second study appeared in the literature in 1978 by Saxena and Lastrico. In this investigation the naturally cemented sand of Vincetown Formation in the New Jersey coast were studied. The Vincetown formation is composed of a variably cemented fine to medium greenish gray sand, with fine content ranging from 10-40 percent by weight. The D_{50} values of the sand range from 0.15—0.49 mm with natural water content varying from 20-40 percent. The material passing #200 sieve had liquid limits and plastic limits of 23-47 percent and 16-33 percent respectively. The material had specific gravity ranging from 2.66 to 2.76 and the dry density ranging from 1.20 to 1.60 g/cc. The sample had length to diameter ratio of greater than 2. The stabilizer was calcite cement and only samples with least cement content were tested. The samples were saturated under back pressure of 20.97 kg/cm^2 and saturation was assumed 100 percent when B parameter had a value equal to or greater than 0.96. Isotropically consolidated triaxial tests were conducted on samples under various confining pressures. Pore pressures were measured in the tests. The rate of shear was 0.025 cm/min and the failure was assumed when the post shear behaviour was observed or until 20 percent axial strain was reached. In total 92 triaxial tests were conducted.

The test results indicated no clear relation between initial porosity and friction angles and also between density and strength because of variation in cementation. Besides, no correlation was observed between strain at failure or maximum deviator stress and confining stresses, thereby confirming the fact that the natural cemented sands possess inherent variation in strength. In general, the stress-strain curves were observed similar to that of a dilating or dense material, even though the tested samples were not dense enough (the cementation creates an "apparent high density"). It was also noted that cemented sands exhibit higher undrained shear strength at lower confining pressures and lower strain levels; however at higher strains behaviour was like uncemented sands. The axial strain contours on p-q plots indicated breakage of cementation and increase in frictional resistance after certain strain levels.

Dupas and Pecker (1979) described static consolidated drained triaxial tests and dynamic triaxial tests to assess the static and dynamic behaviour of cement treated sands. Samples were prepared by compaction with cement content of 5, 7 and 9 percent of the dry weight of sand and at two different dry weights corresponding to 100 and 95 percent of maximum standard proctor density cured for 7 days. A consistent decrease in permeability was obtained with the increase of cement content and dry density. Based on static CID triaxial tests with small range of confining pressure (0.1 — 0.5 MPa) and assuming straight line envelopes, it was concluded that the angle of shearing resistance does not change significantly whereas cohesion intercept increases considerably with the increase in dry density, curing period and cement content. They found that the stress-strain data can be expressed as below :

$$E_t = E_i\left(1-\frac{R_f\,(1-\sin\phi)\,(\sigma_1-\sigma_3)}{2c\cos\phi+2\sigma_3\sin\phi}\right)^2 \qquad \ldots (1)$$

$$E_i = kp_a\left(\frac{\sigma_3+c\tan^{-1}\phi}{P_a}\right)^n \qquad \ldots (2)$$

in which c, ϕ = drained strength parameters, σ_1, σ_3 = principal stresses, E_i = initial Young modulus, p_a = atmospheric pressure and R_f, k and n are the parameters determined from test results. It was observed that k value decreases and n value increases as cement content increases, however R_f value was found constant.

A study on behaviour of natural weakly cemented sands and artificially cemented sands from Stanford Linear Accelerator site and along the Pacific coast was undertaken at Stanford University by Sitar, Clough and Bachus (1980, 1981). Samples from the above two sites (intact and reconstituted) were tested for unconfined compression and drained triaxial compression. About 50 tests were conducted on intact samples and nine on reconstituted ones. The intact samples were tested at natural water content, after soaking for two days, after soaking for four days and in oven-dry conditions. The reconstituted samples were tested at natural water content and oven-dry conditions.

The tests on artificially cemented soils used 50 percent of Monterey sand #0 and 50 percent of Monterey sand #20. The sample dimensions were 7 cm in diameter and 13.8 cm in height. The stabilizer used was Portland cement (2 and 4 percent by dry weight of sand). The samples tested had relative densities of 60, 74 and 90 percent respectively. The samples were compacted in layers of constant thickness to assure uniform density and humid curing was used. Samples were cured for 3 to 28 days and a total of 28 unconfined compression tests were performed to determine the variations of strength with time. The results indicated that 80 percent of 28 day strength had occurred during the first ten days of curing. Therefore, all samples later were cured for 14 days only. The tests performed on artificially cemented sands consisted of four types of static tests : (*a*) unconfined compression tests (*b*) consolidated drained triaxial tests, (*c*) unconfined simple shear tests and (*d*) Brazilian tests. The dynamic tests were cyclic compression triaxial tests

Studies by Rad and Clough (1982) were directed to understand the behaviour of cemented sands subjected to static and dynamic loading under undrained conditions. The investigations involved more than 300 static drained and undrained strain controlled traixial tests. Both naturally and artificially cemented sands, as well uncemented sands were tested. For the artificially cemented samples 1, 2 and 4 percent cement was used and the relative density ranged from 25 to 90 percent. The results of tests on uncemented sand samples formed a basis of comparison to the artificially cemented ones. The samples were prepared by a new method which involves application of an initial vacuum to the specimen, which in turn facilitates saturation under back pressure. The volume change was measured by a new device developed during the research which measures the volume change automatically.

The conventional field tests, such as SPT, CPT and Self boring pressuremeter tests were also conducted in the areas of naturally cemented soils. It was found that the SPT and CPT are of limited use whereas self boring pressuremeter is the best in-situ testing tool to determine the parameters of a weakly cemented sand.

Experimental Investigations

An experimental research programme was initiated at the Illinois Institute of Technology (IIT) from 1983 through 1985 in order to understand the behaviour of artificially cemented sand at different strain levels. The static tests conducted include permeability tests and consolidated drained triaxial compression tests. The parameters considered were : relative density, cement content, curing period, and effective confining pressure. All the details of test results and conclusions have been reported in the interim report (Avramidis and Saxena, 1985) submitted to the National Science Foundation.

In this section a brief background of selected materials, the methods of preparing samples, testing procedures and salient results of above investigation is presented. Results of further studies by newly conducted brazilian and unconfined compression tests are described. Finally, the large body of available data is thoroughly analyzed to fulfil the objectives mentioned earlier.

Materials Used

Monterey No. 0 sand and Portland cement type I (commercial grade) were used. The grain size distribution curve and index properties of Monterey No. 0 sand are given in Fig. 1 and Table 1 respectively.

Sample preparation :

Specimens used in previous and current research were reconstituted by the method of undercompaction, proposed by Ladd (1978). To obtain the desired relative density with this method, a predetermined mass of sand must occupy certain volume inside the sample preparation mould.

The whole sample is made in layers. The lower layers are placed in a

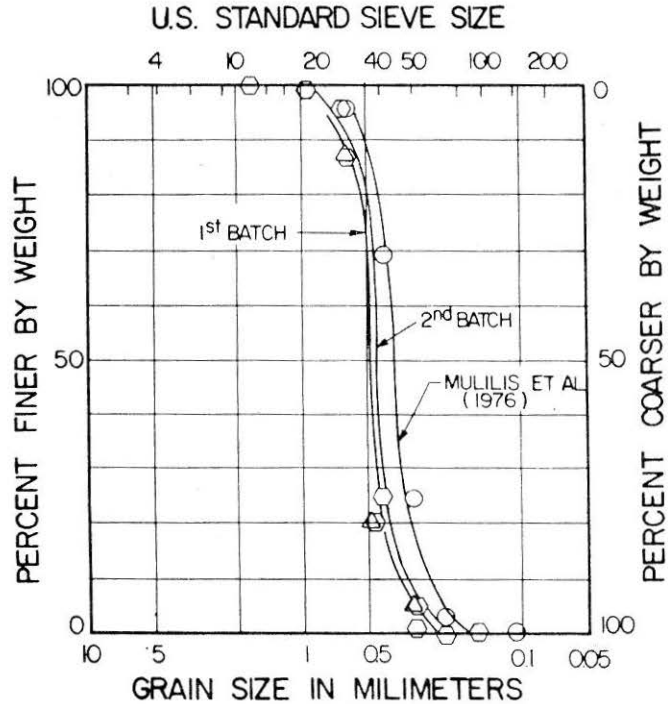

FIGURE 1. Grain Size Distribution for Montery No. O Sand

TABLE 1

Index Properties of Montery Sand

Properties	Values
U.S.C.S. Group Symbol	SP
Mean Specific Gravity	2.65
Particle Size Distribution Data :	
Coefficient of Curvature, C_c	1.02
Coefficient of Uniformity, C_u	1.35
Mean Grain Size Diameter, D_{10}	0.44 mm
Maximum void Ratio	0.85
Minimum Void Ratio	0.56

relatively loose condition (undercompacted) so as the compaction due to the subsequent layers above them will densify these to the desired relative density. Samples prepared with this method are more reproducible than those made with vibration or pluviation techniques (Ladd, 1978). particle segregation is also minimized during preparation and a wide range of uniform relative densities can be achieved.

All the uncemented specimens for static triaxial testing were prepared in the testing apparatus inside an aluminium split mould. However, all the cemented specimens for triaxial testing, brazilian tests and unconfined compression tests were prepared on a stand inside a plastic mould made out of PVC tubing. The details of this sample preparation set up are given in Avramidis and Saxena (1985).

All the specimens were prepared in six layers. In preparing the specimens, first the cement for desired percentage based on the dry weight of Monterey sand per layer was weighed in a porcelain dish. Then proper weighted amount of dry sand, per layer, was added and the two materials were thoroughly mixed by hand without adding water until a mixture of uniform colour appeared. The material was emptied in a larger porcelain dish where 8 percent water, based on the dry weight of the sand-cement mixture was added. The resulting sand-cement-water mixture was re-mixed thoroughly using a steel rod 0.635 cm in diameter. The wet homogeneous mixture was then placed inside the mould using a spoon, leveled and subsequently campacted with a tamper. The degree of compaction used was 6 percent and the procedure was repeated for the rest of the layers. The cemented samples were then cured below water for different days. The height to diameter ratio for the uncemented and cemented samples was between 2.0 and 3.0.

Tests conducted

The different static tests conducted previously and during current research are as follows :

(*i*) Static Drained Triaxial Tests

(*ii*) Brazilian Tests (or Splitting Tension Tests)

(*iii*) Unconfined Compression Tests

The materials and method of sample preparation are same for all these tests.

Static Triaxial Tests : A total of 152 static strain controlled, isotropically consolidated drained triaxial compression tests were conducted on uncemented and cemented sands during the first phase. The following test variables were considered :

Loading Strain Rate, percent per minute = 0.186

Effective Consolidation Pressuie, kPa = 49, 245 and 490

Relative Density, percent = 43, 60 and 80

Cement Content, percent = 0, 2, 5 and 8.

Curing period, days = 15, 30, 60 and 180

After the specific curing period was completed, measurements of height and diameter of the specimen were made. The sample surrounded by two membranes, each having thickness of 0.317 mm, was placed between the pedestals of the triaxial cell and top and bottom were sealed off using two rubber O-rings. Uncemented specimens were prepared in a split mould which was placed on the triaxial cell pedestal and surrounded by two membranes. The space between the specimen and the cell chamber was filled with fresh deaired water.

To facilitate saturation process, the specimens were first flushed with carbon dioxide and then with fresh deaired water under a back pressure of 192 kPa. The effective confining pressure during saturation was 25 kPa. Some samples were also saturated under vacuum. No effect of carbon dioxide on the strength of samples was found. The saturation was considered adequate when Skempton's pore pressure parameter, B was equal or larger than 0.96. With the triaxial testing set up used in this investigation, the coefficient of permeability was also determined. Subsequently the specimen were consolidated under specific effective confining pressure. The volume change during consolidation was obtained from water levels in the burette from which the preshear data such as void ratio etc. were obtained. Then the specimen was axially loaded to shear. During testing, the following parameters were monitored and recorded : (1) time, (2) axial deformation, (3) axial load, (4) volumetric change, (5) back pressure and (6) cell pressure.

Brazilian Tests : Splitting Tension Tests : A total of 16 specimens were tested with the test set-up schematically shown in Fig. 2. The following variables were considered in the testing :

Loading Strain Rate, percent per minute = 0.186

Relative Density, percent = 43, 60 and 80

Cement Content, percent = 2, 5 and 8

Curing period, days = 15

Length/Diameter Ratio = 2.0

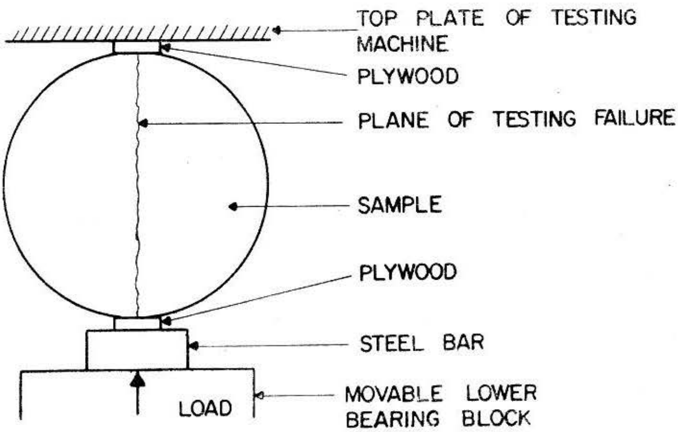

FIGURE 2. Schematic Diagram of Test Set-up for Brazilian Tests

The samples tested were prepared by method of undercompaction in the same way as for static triaxial tests. Circular steel plates with diameter slightly larger than the length of the samples were fixed to the top plate of testing machine and the lower bearing block of triaxial testing machine in such a manner that the load applied is distributed over the entire length of the specimen. Two bearing strips of 2.5 cm wide and 0.5 cm in thickness of smooth plywood of a length equal to length of specimen were prepared. One of the plywood strips was placed in the center of the lower bearing block. After precise measurement of length and diameter, the specimen was placed on this lower plywood strip. The upper plywood strip was then placed lengthwise on top of the specimen. The movable lower bearing block was raised slowly until the sample and the plywood strips were gripped by the top plate. Load was then increased until the specimen failed. The loads at which first crack appeared and the sample failed, were recorded. No measurements for strains were made. The tensile strength of specimen was calculated using the following expression;

$$\sigma_t = \frac{2P}{\pi LD} \tag{3}$$

where σ_t = tensile strength in pounds per square inch, P = maximum applied load, in pounds, L = length in inches and D = diameter in inches.

Unconfined Compression Tests : In order to relate tensile strength of cemented sands with unconfined compressive strength of cemented sand with identical parameters, a total of 16 specimens were tested with similar parameters considered for tensile tests. The samples were prepared by the method of undercompaction described earlier and then tested without membranes in triaxial cell.

Aanalysis of Test Results

Based on the previous studies, the behavior of cemented sand is found to be influenced by factors such as strain level, type of cement, cement content, density, time, effective consolidation pressure grain size distribution, structure, method of sample preparation, water content, degree of satuation, etc.

The objective of this paper is to present analysis of results obtained from tests mentioned in the previous sections to comprehend the strength-deformation characteristics of cemented sand.

Tensile strength versus unconfined compressive strength

Unlike sands, the cemented sands possess some tensile strength. Therefore to reveal the complete constitutive behavior of cemented sand, the results of tensile tests are essential in addition to compression tests. The tensile strength of cemented sand is not given much attention because of lack of practical and reliable testing technique. Clough et al. (1980 and 1981) reported very few brazilian tests on cemented sand and stated that the tensile strength is about 10 to 12 percent of the unconfined compressive strength and also the cohesion intercept is about twice the tensile strength. In the absence of sufficient experimental data, the present practice is to assume a parabolic stress-strain variation in the tensile region. There is a great need of new research to establish some data base for tensile strength of cemented sands by adopting similar techniques that provide fairly good data for concrete, rocks, clay, etc. in tension.

In this study, several brazilian (or splitting tension) tests were conducted. The results of brazilian tests and unconfined compression tests are summarized in Table II along with the results reported by other investigators on similar studies which provide a preliminary data base. The tensile strength

TABLE II

Summary of Results from Brazilian & Unconfined Compression Tests

Cement Content (CC) %	Relative Density (Dr) %	Tensile Strength kN/m²	Unconfined Compressive Strength, q_f kN/m²		
			Rad & Clough 1982	Acar & Tahir 1986	Present Study
	25	1	7	10	12
	35	—	—	15	—
1	43	1.5	—	—	17
	50	—	20	19	—
	60	1.8	—	—	25
	80	2.2	30	28	33
	25	5.3	25	22	24
	35	—	—	33	—
2	43	8.8	—	—	43
	50	—	42	41	—
	80	11	55	54	58
	25	—	—	48	—
	35	—	—	51	—
	50	—	—	59	—
4	60	—	203	63	—
	75	—	275	69	—
	80	—	—	71	—
	90	—	350	77	—
	25	24.4	—	—	181
5	43	31	—	—	218
	60	39	—	—	247
	80	45	—	—	282
	25	67	—	—	476
	43	72	—	—	495
8	60	84	—	—	527
	80	90	—	—	564

obtained from these tests are used in the Lade's model to find the parameter 'a'. More details have been provided by the authors elsewhere (Saxena and Reddy, 1987).

A statistical analysis on the available experimental data with cemented sand (Table II) provided the following correlations between unconfined compressive strength q_u and shear strength parameter 'c'.

For low cementation

$$q_u = 2.1\ c' \tag{4}$$

For high cementation

$$q_u = 1.4c' \tag{5}$$

It may be noted that the Eq. 4 was also suggested by Acar and Tahir (1986), however they use this equation to predict strength for all degrees of cementation. The experimental results obtained during this study clearly indicate that at high cementation levels Eq. 4 overpredicts q_u and Eq. 5 provides better results.

Similar investigation with brazilian test results and unconfined compression test results resulted the following correlation.

$$\sigma_t = -0.15\, q_u \tag{6}$$

This relation has been found valid at all cementation levels and leads to the discussion about the applicability of Griffith's theory of failure (1920) for cemented sand.

According to Griffith's theory, if $\sigma_1 > \sigma_2$ and $\sigma_1 + \sigma_3 < 0$ the failure envelope is expressed as;

$$(\sigma_1 - \sigma_3)^2 = -8\sigma_t(\sigma_1 + \sigma_3) \tag{7}$$

For uniaxial compression condition $(\sigma_3 = 0, \sigma_1 = q_u)$ one gets

$$q_u = -8\sigma_t \tag{8}$$

However from Eq. 6, one obtains

$$q_u = -6.7\sigma_t \tag{9}$$

Maclintock and Walsh (1962) and Brace (1963) suggested modifications to Griffith's theory and derived the following expression :

$$\frac{q_u}{\sigma_t} = \frac{4}{(1+\mu^2)^{1/2} - \mu} \quad \ldots (10)$$

in which μ = coefficient of friction for crack surface. Considering an average angle of shearing resistance of 37 degrees for cemented sands (Table III), and assuming $\mu = tan\phi$, the above expression reduces to Eq.8. The comparison of Eqs. 8 and 9 however, suggest a need for further investigations regarding the determination of coefficient of friction for crack surface (μ) and also the validity of parabolic strength envelope of Griffith's theory in the tensile stress region.

Shear Strength

Angle of shearing resistance and cohesion intercept are the two important shear strength parameters of soils. But in general static loads on soils are carried by the five components of their shear resistance, nomely cohesion,

TABLE III(a)

Values of Angle of Shearing Resistance and Cohesion for Peak and Residual Stages for Cemented Monterey #O Sand

C.C. %	Dr %		C.P. 15 days Peak	C.P. 15 days Resid.	C.P. 30 days Peak	C.P. 30 days Resid.	C.P. 60-days Peak	C.P. 60-days Resid.	C.P. 180 days Peak	C.P. 180 days Resid.
2	43	$\bar{\phi}$*	34.1	31.8	33.0	31.4	33.1	32.8	34.3	33.8
		$\bar{c}$**	43	22	53	15	55	13	51	6
2	60	$\bar{\phi}$	34.9	33.7	33.2	32.0	35.2	34.3	35.6	34.4
		$\bar{c}$	49	0	66	16	71	9	60	19
2	80	$\bar{\phi}$	36.9	32.9	36.3	32.0	35.3	34	37.4	24.9
		$\bar{c}$	50	8	53	11	58	4	64	5
5	43	$\bar{\phi}$	35.6	35.3	35.4	34.9	35.9	32.9	36.1	35.2
		$\bar{c}$	146	21	150	17	157	15	159	13
5	60	$\bar{\phi}$	37.3	36.9	36.9	36.9	37.8	35.2	36.9	36.3
		$\bar{c}$	153	19	177	22	190	20	210	14
5	80	$\bar{\phi}$	38.7	34.6	39.2	36.9	38.5	37.8	38.0	38.0
		$\bar{c}$	150	6	221	0	230	0	223	3
8	43	$\bar{\phi}$	36.3	36.3	36.9	35.8	37.8	37.1	37.8	37.8
		$\bar{c}$	347	20	360	20	368	0	372	11
8	60	$\bar{\phi}$	39.8	39.6	40.3	39.9	39.8	39.8	39.4	36.6
		$\bar{c}$	358	0	367	0	369	18	371	20
8	80	$\bar{\phi}$	40.9	40.9	42.0	40.5	42.4	38.0	43.4	39.8
		$\bar{c}$	366	0	417	0	383	15	420	0

*The angle of shearing resistance is measured in degrees.

**The cohesion, c, is measured in kPa.

TABLE III(b)

Values of Angle of Shearing Resistance and Cohesion for Peak and Residual Stages, for Uncemented Monterey #0 Sand

Dr %	Peak $\bar{\phi}$ (deg)	Peak $\bar{c}$ (kPa)	Residual $\bar{\phi}$ (deg)	Residual $\bar{c}$ (kPa)
43	33.7	0	32.9	0
60	35.3	4	33.6	4
80	37.1	6	34.9	0

basic mineral friction, dilatancy, particle crushing and particle rearrangement. However, basic mineral friction, dilatancy, particle crushing and particle rearrangement, are usually considered to constitute the frictional resistance of soils.

The gross shearing resistance of soils is increased greatly when they are mixed with small amounts of cementing agents such as portland cement, lime, etc., as it was shown by Wissa and Ladd (1965). Avramidis and Saxena (1985) based on the previously mentioned static triaxial test results, explained and reconfirmed the conclusions of study by Wissa and Ladd (1965). Following are the brief conclusions of Avramidis and Saxena (1985) which formed the strong background for the present study. :

1. The stress-strain response was greatly influenced by effective confining pressure ($\bar{\sigma}_c$) and cement content (CC) and to a smaller degree by curing period (CP) and relative density (D_r). Even a loose specimen stabilized with a small amount of cement could exhibit brittle behavior (Fig. 3).

2. In order to quantify the brittleness of cemented sands, the brittleness coefficient (Bc) was introduced. Bc was defined as the ratio of peak shear strength (S_{peak}) over its residual shear strength (S_{resid}). Brittle behavior was demonstrated more at low $\bar{\sigma}_c$ and large CC conditions (Fig. 4).

3. The residual strength of the uncemented sands was slightly lower than corresponding one for the cemented sands at the same D_r and $\bar{\sigma}_c$ values.

4. An increase in the angle of shearing resistance and the cohesion intercept with increase in cement content was observed consistently (Fig. 5). Strength ratio, defined as the ratio of cemented peak strength

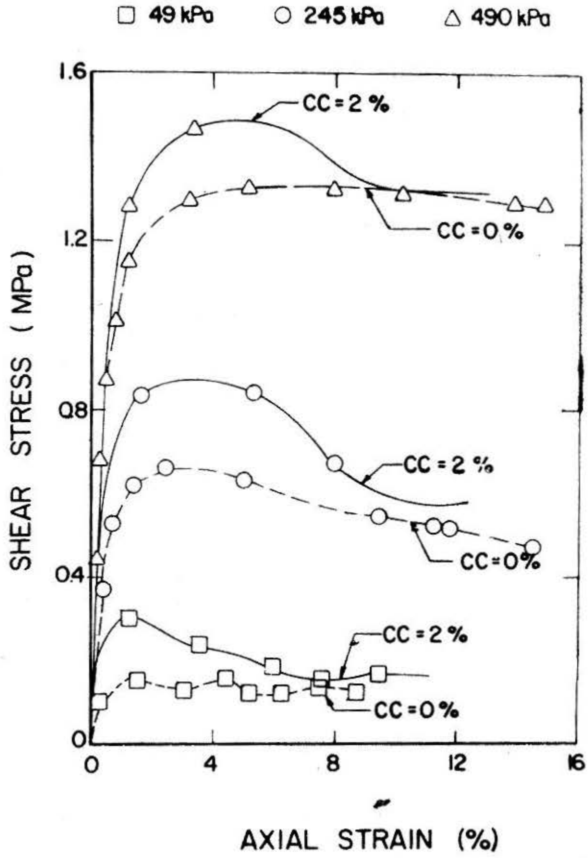

FIGURE 3 (a) Stress-Strain Response of Uncemented and Cemented Sands for $D_r = 43\,\%$

to uncemented peak strength, decreases as $\bar{\sigma}_c$ increases and CC decreases (Fig. 6).

5. *For the uncemented sand Mohr* envelope at peak strength represents a condition where the maximum rate of volumetric expansion occurs. Whereas for the cemented sand it represents a condition where the summation of all strength components become maximum.

6. All the values of angle of shearing resistance and cohesion for peak and residual stages for the range of variable considered are given in Table III.

Though the above conclusions created the static quantitative behavioral basis, need for further study to investigate the strength generation (i.e. the variation of c and ϕ with strain), the initial Young's modulus and the stress-strain characteristics was felt.

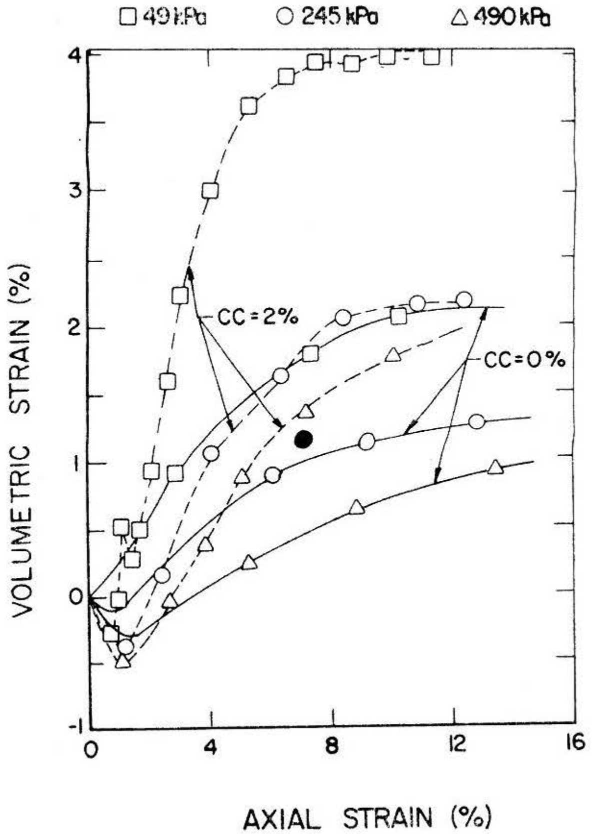

FIGURE 3 (b) Volumetric Strain versus Axial Strain Curves of Uncemented and Cemented Sands for $D_r = 43\%$

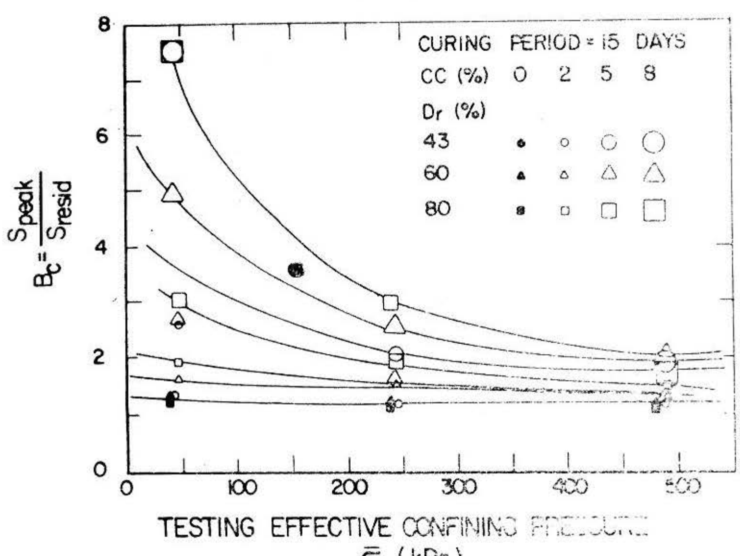

FIGURE 4 Coefficient of Brittleness versus σ_c at Various D_r and CC values

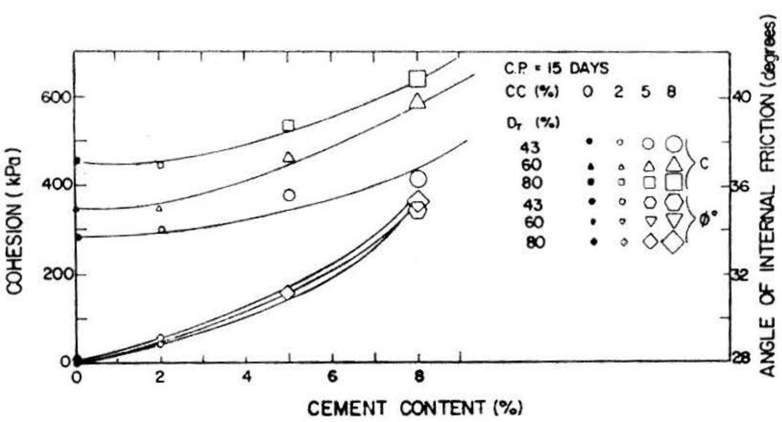

FIGURE 5 Cohesion and Angle of Shearing Resistance at Peak Strength versus Cement Content after 15 days of Curing

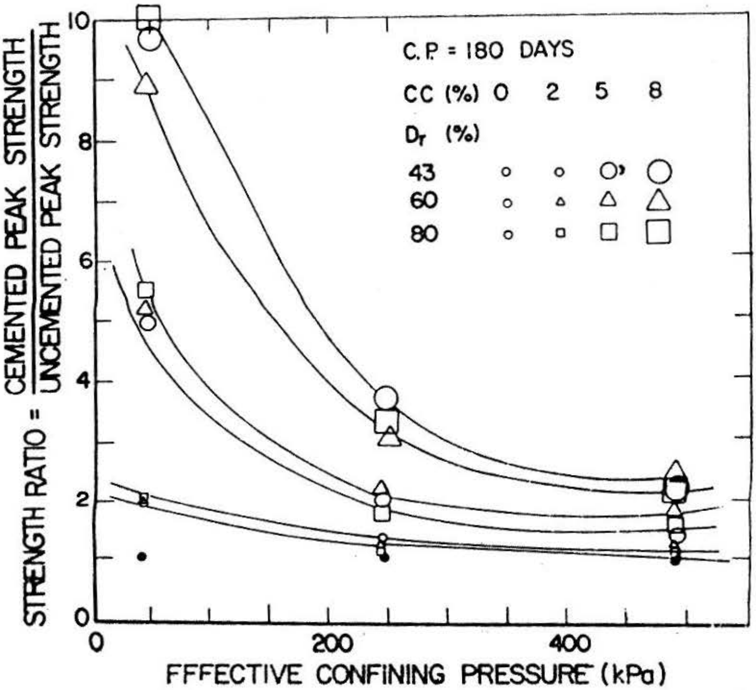

FIGURE 6 Strength Ratio versus Effective Confining Pressure

Strength Generation

The shearing resistance of uncemented sand has been very well understood because of significant research efforts by Hvorslev, Ladanyi, Rowe, Koerner and others. The mineral soils were noted to be nonplastic in the grain sizes tested and therefore the effective cohesion was considered as zero. Therefore the entire attention has to be focused on the effective angle of shearing resistance parameter ϕ. It has also been suggested by many investigators that the ϕ found from drained tests could be expressed as :

$$\phi = \phi_{mf} + \phi_{pc} + \phi_{pr} + \phi_d \tag{11}$$

where ϕ_{mf} = angle of basic mineral friction

ϕ_{pc} = angle of degradation or particle crushing

ϕ_{pr} = angle of reorientation or particle rearrangement

ϕ_d = angle of dilatancy

The above expression can be rewritten as

$$\phi = \phi_f + \phi_d \tag{12}$$

where ϕ_f angle of interparticle friction or angle of internal friction can also be termed as effective angle of shearing resistance. Therefore in order to find effective angle of shearing resistance ϕ_f the ϕ_d has to be separated from measured angle of friction ϕ.

Uncemented Monterey No. 0 sand used in this study exhibited curved effective stress envelope in drained tests under dense conditions as shown in Fig. 7. The slope of the envelope decreases with increasing consolidation

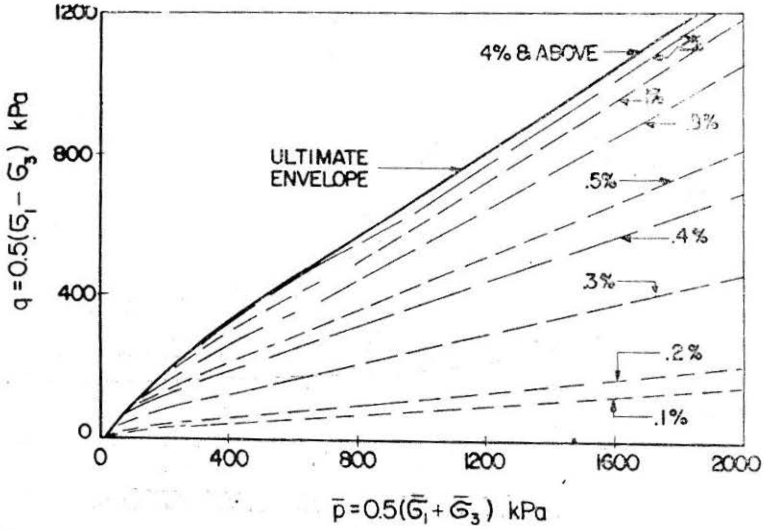

FIGURE 7 Strength Envelope and Strain Contours for Dense Uncemented Monterey sand ($D_r = 80\%$)

pressure. Due to curvature, the Mohr envelope shows on increase in 'apparent' cohesion intercept with increasing consolidation pressure. However at high consolidation pressure, the curvature of the drained Mohr envelope and the apparent cohesion disappeared. Rowe stress-dilatancy equation as given below can be used to modify the stress difference (deviator stress) to account the influence of volume changes on the work done during shear.

$$(\sigma_1 - \sigma_3)_{modi} = \frac{\bar{\sigma}_1}{(1 + \frac{d\varepsilon_v}{d\varepsilon})} - \bar{\sigma}_3 \qquad \ldots (13)$$

For sands the Mohr envelope using the modified stress difference results in a straight line.

In case of loose sand, the effective stress envelope in drained tests was straight line as shown in Fig. 8. Therefore the cohesion is always zero and no need of Rowe stress-dilatancy correction arises.

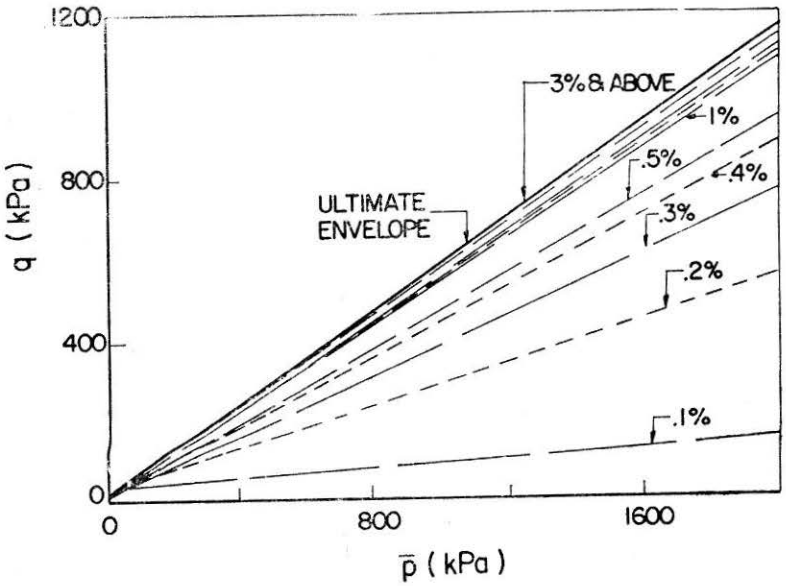

FIGURE 8 Strength Envelope and Strain Contours for Loose Uncemented Monterey Sand ($D_r = 43\%$)

Axial strain contours have been drawn on the drained p versus q plot as shown in figures 7 and 8. It may be noticed that these results are similar to those reported by Wissa and Ladd (1965). The slopes initially increased with increasing axial strain in loose conditions mainly because of the mineral to mineral friction between grains until failure. Whereas in case of dense sands when the volume of the samples reached a minimum, on further straining the samples started to dialate at a gradually increasing rate, causing an increase in the slopes of the strain contours until maximum rate of dilatancy occurs. On further straining, the rate of dilatancy decreases and therefore, the slopes decreased until ultimate conditions were reached.

Figures 9 and 10 show the variation of gross cohesion (c) and gross angle of internal friction ϕ with axial strain. The reported values of the

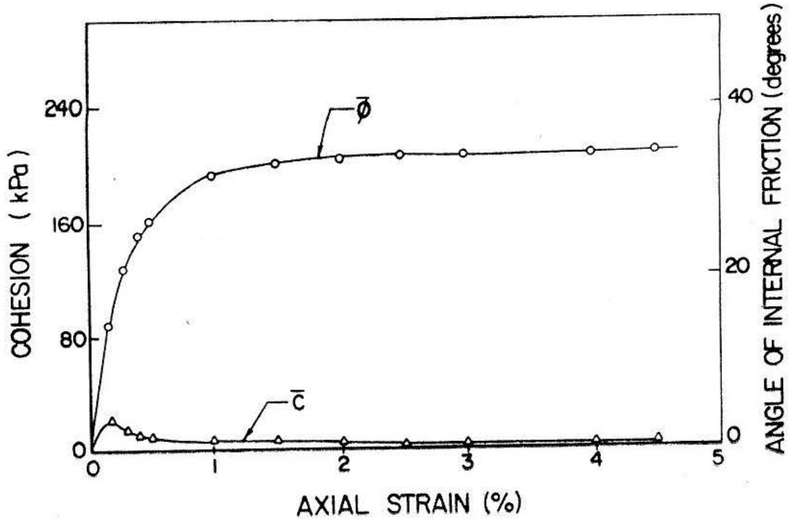

FIGURE 9 Variation of c and ϕ with Axial Strain for Dense Uncemented Monterey sand (D_r=80)

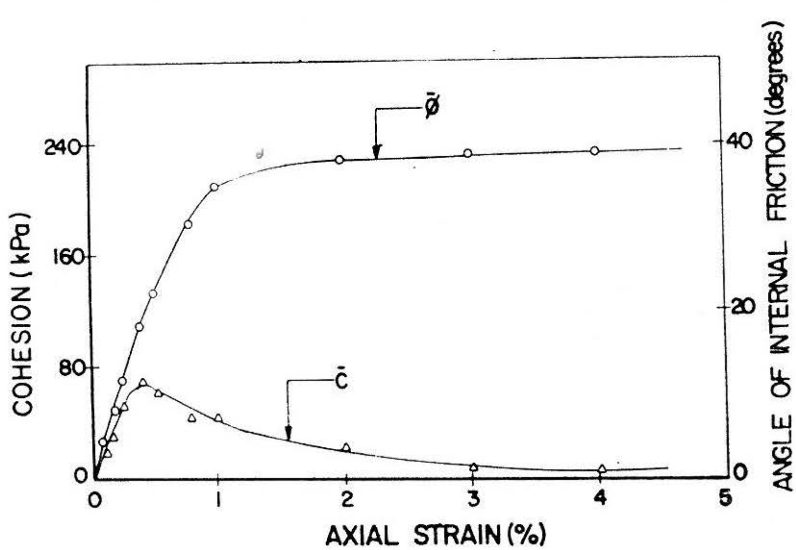

FIGURE 10 Variation of c and ϕ with Axial Strain for Loose Uncemented Monterey Sand (D_r=43%)

strength parameters (c and ϕ) are deduced indirectly from p − q envelopes, using the following relationships.

$$\phi = \sin^{-1}(\tan \alpha) \quad (14)$$

$$c = \frac{a}{\cos \phi} \quad \dots (15)$$

where α is the inclination of the p − q envelope and a is the q intercept of the p − q envelope. It can be concluded that at relatively small strain, the mobilization of friction occurs and remains same on further straining.

Only few investigators studied the effect of artificial cementation of sand (Wissa and Ladd, 1965; Dupas and Pecker, 1979.) In all the studies, Mohr Coulomb strength criterion was used. The strength was represented by two components namely cohesion and friction. Avramidis and Saxena (1985) demonstrated that the shear strength of artificially cemented sands is influenced by cement content and curing period because the cement tends to bond the sand grains together. Besides all these, the present investigation revealed that the shear strength of cemented sand is strongly dependent on strain level. This strain dependent behaviour was not adequately studied quantitatively by previous researchers.

The use of strain contours to separate the frictional and cohesive resistance is open to question when it is applied to fine-grained soils since large decrease in void ratio occurs with increasing consolidation pressure, and/or applied load during shear of the sample, therefore, the fabric changes with these loads. As the void ratio decreases, the number of mineral to mineral contacts increase which will increase the frictional resistance during shear.

The strain contours of cemented sand with different ratios for a specific $D_r = 43$ percent are shown in Figs. 11 through 13. The rates at which the cohesive and frictional resistance change with increasing axial strain as obtained from intercept and slope of strain contours (Eqs. 14 and 15), are shown in Fig. 14 through 16.

From these results, it may be concluded that at small axial strains, most of the shear strength is contributed from cohesion and with increasing axial strain, frictional resistance increases. After the cohesive resistance approaches its maximum value around 0.25 − 0.85 percent strain, the contribution of cohesion drops fast and the mobilization of frictional resistance increases relatively quickly. The frictional resistance becomes maximum as the stress path touches the Mohr-Coulomb envelope.

Deformation Modulus

In contrary to other soils, the stress-strain response of cemented sands is predominantly elastic during the initial stages of loading. Sitar and Clough

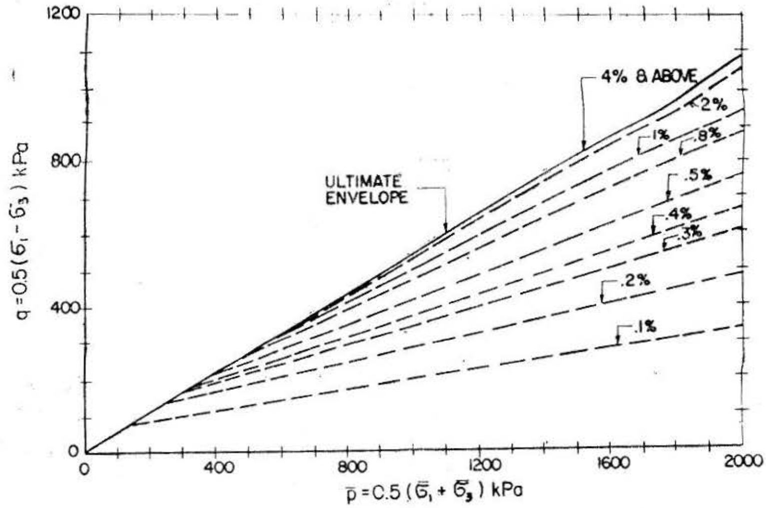

FIGURE 11 **Strength Envelope and Strain Contours for Cemented Monterey Sand with $D_r = 43\%$ and CC=2%**

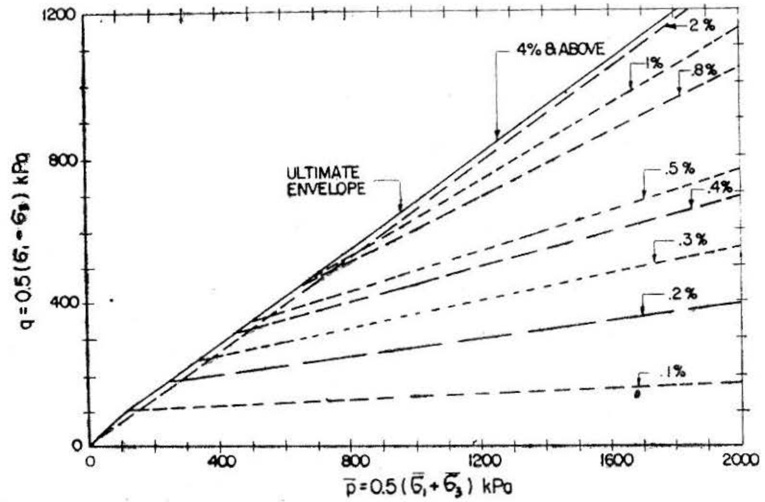

FIGURE 12 **Strength Envelope and Strain Contours for Cemented Monterey Sand with $D_r = 43\%$ and CC=5%**

(1983) assumed stress-strain variation of cemented soil as linear upto peak strength in a finite element analysis to study the behaviour of cemented soil slopes. The post failure constitutive behavior was not considered in their study perhaps because the post failure deformations of the sliding mass are irrelevant as it collapses and disintegrates. However, they recognized that after the peak, the brittle nature of the failure and subsequent softening, makes it extremely difficult to duplicate the exact behavior by simple models.

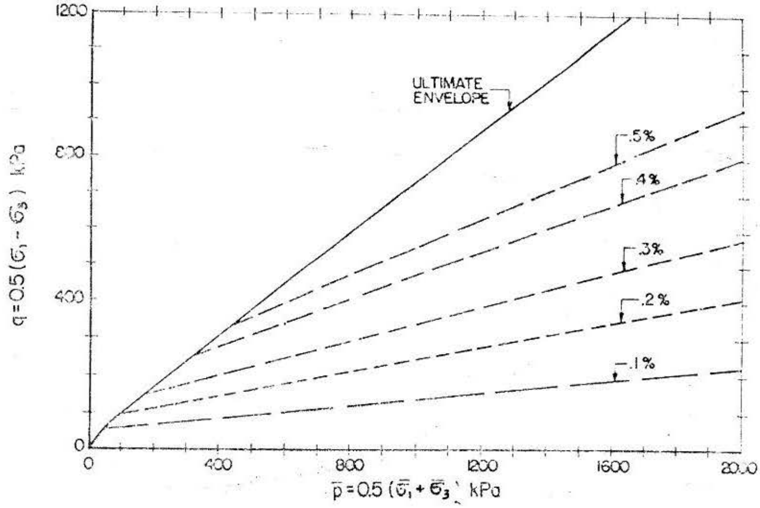

FIGURE 13 Strength Envelope and Strain Contours for Cemented Monterey Sand with $D_r = 43\%$ and $CC = 8\%$

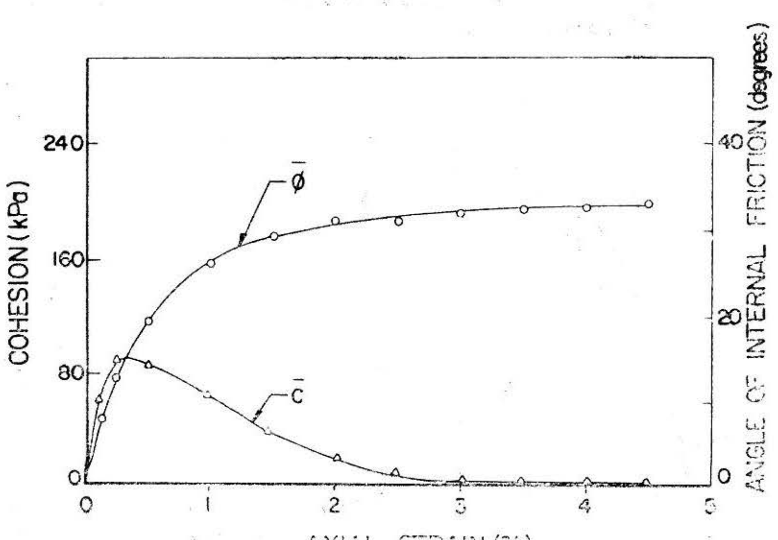

FIGURE 14 Variation of C and ϕ with Axial Strain for Cemented Monterey Sand with $D_r = 43\%$ and $CC = 2\%$

Yielding, therefore, occurs just before the peak and well before peak in cases of strongly and weakly cemented sands respectively. Therefore, it should be understood that the consideration of elastic response upto peak stress state is a crude approximation for weakly cemented sands and may be a reasonable approximation for strongly cemented sands. Finally, the success of elastic model for cemented sand for prefailure conditions greatly

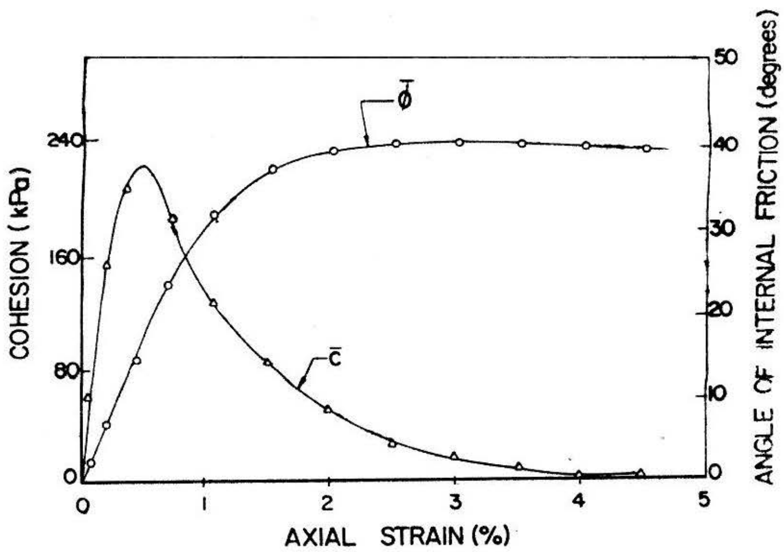

FIGURE 15 Variation of c and ϕ with Axial Strain for Cemented Monterey Sand with $D_r = 43\%$ and $CC = 5\%$

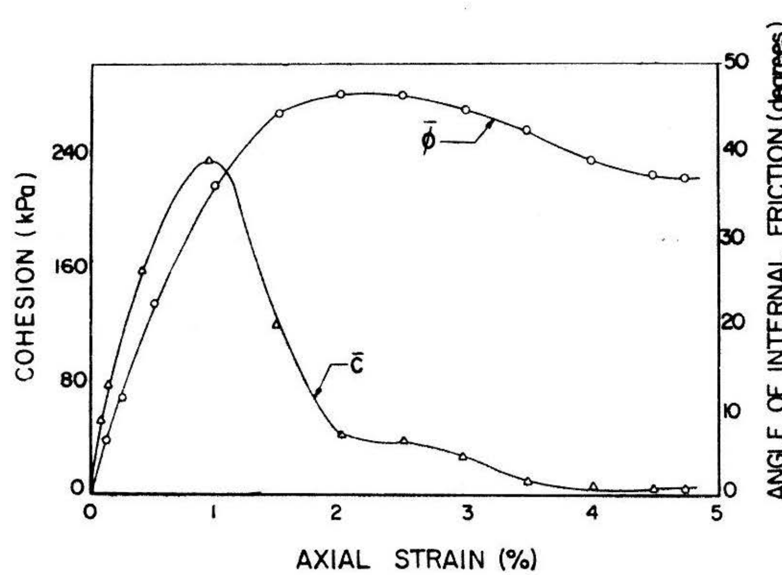

FIGURE 16 Variation of c and ϕ with Axial Strain for Cemented Monterey Sand with $D_r = 43\%$ and $CC = 8\%$

depends on the method of selecting elastic modulus. Even the advanced constitututive models require initial or elastic modulus as input. In this study, the results of triaxial (drained) tests are used to investigate the affecting factors and the selection methods of elastic modulus for cemented sands.

Elastic modulus or deformation modulus is considered equal to initial tangent modulus from stress-strain curves of drained triaxial tests. The parameters that affect the modulus are confining pressure, cement content, curing period and density. All these variables tend to increase the initial tangent modulus values, however, the effects of cement content and confining pressure are significant.

All the previous investigators found that $(\bar{\sigma}_3/p_a)$ versus (E_i/p_a) represents "reasonably" a straight line on the log-log scale for cemented sands. Such an idea is adopted based on Janbu (1963) and Wong and Duncan (1974), who first found such relationship for uncemented sands and the equation of this straight line was given as below :

$$E_i = kp_a\left(\frac{\bar{\sigma}_3}{p_a}\right)^n \qquad \dots (16)$$

where $\bar{\sigma}_3$ = effective confining pressure, E_i = initial tangent modulus and p_a = atmospheric pressure. $\bar{\sigma}_3$, E_i and p_a are expressed in the same units. k and n are parameters which depend on soil condition and are determined from tests (k is the intercept at $(\bar{\sigma}_3/p_a) = 1$ and n is the slope of the line). In case of sands, k is directly related to stiffness and n exhibits the effect of confining pressure and relates the frictional component of strength.

Obviously for cemented sand, the initial tangent modulus not only depends on $\bar{\sigma}_3$ but also on density, cement content, curing time, etc. In order to account the effect of cement content drained shear strength parameters c and ϕ were included in the above equation and the modified equation was given by Dupas and Pecker as below :

$$E_i = kp_a\left(\frac{\sigma_3 + c\ \tan^{-1}\phi}{p_a}\right)^n \qquad \dots (17)$$

Dupas and Pecker (1979) observed that the influence of density, cement content and curing time are indirectly accounted by the term c $\tan^{-1}\phi$. It is concluded that k values decrease and the n values increase as the cement content increases. On the other hand, researchers at Stanford University (Sitar et al., 1980, Bachus et al. 1981) adopted equation 16 because, in their view, it is most conveniently used by geotechnical engineers. They found that k values increase and the n values decrease as the cement content increases. This variation of k and n in equation 16 is just contrary to the observation of Dupas and Pecker (1979). The results of present study are therefore useful to evaluate these conflicting conclusions. Undoubtedly, all straight line relationships are liked by practicing engineers, however accuracy should not be sacrificed for simplicity when using such relations.

The values of initial tangent modulus are obtained for different effective confining pressures based on stress-strain data from triaxial (drained) tests with wide range of variables such as density, cement content, curing time, etc. A set of k and n values are obtained by plotting $(\bar{\sigma}_3/p_a)$ versus (E_i/p_a) on log-log scale. Also a set of k and n values are obtained by plotting $(\bar{\sigma}_3 + c\,tan^{-1}\phi)/p_a$ versus (E_i/p_a). All the values of k and n are summarized in Table IV for different values of the variables considered in the present experimental investigations.

TABLE IV

Values of Elastic Modulus Parameters

Cement content %	Relative Density %	Elastic Modulus Parameters: Eqn. 16 (text) k	Eqn. 16 (text) n	Eqn. 17 or Eqn. 18 (text) k	Eqn. 17 or Eqn. 18 (text) n
	43	675.0	0.88	675	0.88
0	60	749.0	0.85	713.2	0.875
	80	877.0	0.81	820.2	0.845
	43	1082.8	0.57	660.57	0.815
2	60	1252.0	0.61	683.34	0.90
	80	1598.5	0.67	923.02	0.94
	43	1613.2	0.42	555.31	0.90
5	60	2003.0	0.47	525.76	1.06
	80	2781.6	0.52	584.68	1.20
	43	2170.7	0.27	396.87	0.95
8	60	2549.2	0.31	436.51	1.02
	80	3685.5	0.38	449.61	1.23

The present investigation revealed that the following expression for E_i (obtained by translation of axes) also gives the same results as obtained from Eq. 17;

$$E_i = kp_a \left(\frac{\bar{\sigma}_3 + c \cot \phi}{p_a} \right)^n \quad \ldots (18)$$

Inspite of inclusion of c and ϕ in the expression for E_i (Eqs. 17 & 18), the values of k and n still remained dependent on the cementation level.

An alternate expression for initial Youngs modulus for cemented sand (E_i^*) can be expressed in terms of initial Youngs modulus of uncemented sand (E_i) as below;

$$E_i^* = RE_i \quad \ldots (19)$$

where R can be called modulus ratio. The value of R, in general depends on cement content, density, curing period and effective confining pressure. A statistical analysis with the experimentally determined modulus values provided the following relationship for R;

$$\log R = \log\,(1+C-eC)+(0.71-1.3e)(C)^{(2.2-2.4e)}\log\left(\frac{\bar{\sigma}_3}{p_a}\right) \quad \dots (20)$$

Where C is the cement content (CC) expressed in percentage and e is the void ratio. The effect of curing period is to increase the modulus. However after initial few days of curing the increase in modulus is not very significant therefore it has not been incorporated in the above expression for R. The modulus ratio (R) can also be related to unconfined compressive strength (q_u) as follows :

$$R = a\left(\frac{\bar{\sigma}_3}{p_a}\right)^b \quad \dots (21)$$

$$a = 2\left(\frac{q_u}{p_a}\right)^{0.29} \quad \dots (22)$$

$$b = \left(\frac{q_u}{p_a}\right) - 0.40 \dots \text{ for } \frac{q_u}{p_a} < 0.25 \quad \dots (23a)$$

$$b = 0.7\left(\frac{q_u}{p_a}\right) - 0.57 \dots \text{ for } \frac{q_u}{p_a} < 0.50 \quad \dots (23b)$$

$$b = 0.2\left(\frac{q_u}{p_a}\right) - 0.86 \dots \text{ for } \frac{q_u}{p_a} < 3.00 \quad \dots (23c)$$

$$b = 0.22\left(\frac{q_u}{p_a}\right) - 1.70 \dots \text{ for } \frac{q_u}{p_a} < 6.00 \quad \dots (23d)$$

In summary, the elastic modulus for artificially cemented sands can be found using any of the above mentioned relationships. If unconfined compressive strength is known, the modulus can be easily computed from *Eq. 19 by knowing modulus ratio from Eqs. 21, 22 and 23.*

Stress-Strain Characteristics or Constitutive Behaviour

Unlike metals, the stress-strain response of soils is complicated. The artificially cemented sands tend to possess dilatant brittle nature. Based on the results of drained triaxial test results of Avramidis and Saxena (1985), the present study is devoted to describe the constitutive behaviour of cemented sands.

The typical stress-strain variation of strongly cemented sands is depicted qualitatively in Fig 17. During the initial stages of loading (OA), a linear elastic response (almost upto the peak) can be observed because of cementation which prevents intergranular movement. The initiation of cement

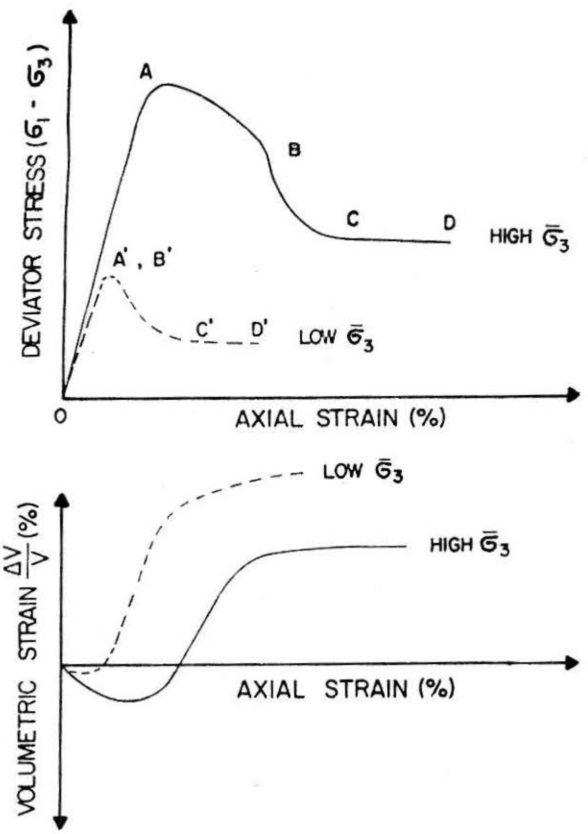

FIGURE 17 Stress-Strain Response of Strongly Cemented Sands

bond breaking starts just before peak (A), afterwhich gradual nonlinear softening occurs due to progressive breaking of cement bonds (A to B). The gradual softening behaviour is mainly because of high confining pressure which offers resistance to dialation. When complete breakdown of cementation occurs (B), a rapid nonlinear softening is exhibited (BC). After the residual state (C), the strength remains constant and is mainly due to the frictional resistance of sand. As can be seen from the same figure, at low confining pressures because of insufficient resistance for dilation, gradual softening (AB) does not occur.

The behaviour of weakly cemented sands is different from that of strongly cemented sands (Fig. 18). The elastic range (OA) is very small and the yielding of cement bonds start well before the peak. The complete brakdown of cementation occurs almost near the peak. Further straining causes gradual and sudden nonlinear softening under high and low confining pressures respectively and finally residual strength is attained. It may be pointed out that the behaviour of weakly cemented sands is almost similar to that of uncemented dense sand.

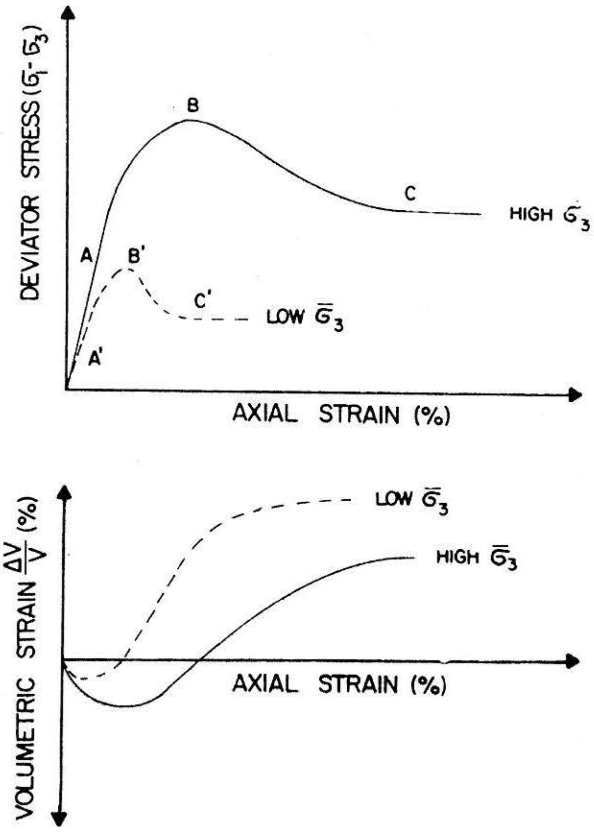

FIGURE 18 Stress-Strain Response of Weakly Cemented Sands

The results also indicate decrease in volume during the initial stages of loading afterwhich continuous increase in volume occurs. It has been concluded by previous investigators that the peak strength represents the maximum rate of volumetric expansion in cases of uncemented sands, whereas, in case of cemented sands it represents the culmination of the contribution of cementation and dilation followed later by the residual strength. The peak and residual strengths indicate the degree of brittleness of soil. Brittleness coefficient (Bc) may be used to quantify the brittleness. The brittleness of cemented sands is found more at low confining pressures and at high cement contents.

In order to quantify the above described stress-strain behaviour, the four popular types of constitutive models namely (1) Hyperbolic model (Duncan & Chang model), (2) Elasto-plastic model (Lade's model), (3) cap models and (4) Endochronic model are under investigation by the authors for applicability and duplicating the behaviour of cemented sands.

Conclusion

The extensive experimental programme undertaken in this study increased the data base for cemented sand under static conditions. The different types of tests such as triaxial tests, unconfined compression tests etc. were conducted on the specimens prepared identically with one-to-one correspondence between the involved variables therefore the relationships developed among q_u, c and σ_t are free from sample preparation effects. The static triaxial test results helped to quantify the beneficial effects of artificial cementation of sand. The strain dependent behaviour (or strength generation) is adequately studied and it is found that at small axial strains (0.25—0.85%) most of the shear strength is contributed from cohesion and with increasing axial strain, frictional resistance increases until the stress path touches the failure envelope. Also, the selection methods of deformation modulus are reviewed and an alternate new relationship is proposed. Finally a qualitative description of constitutive behaviour is given in order to examine the validity of existing constitutive models for cemented sand in future research.

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