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CYCLIC AND MONOTONIC UNDRAINED SHEAR RESPONSE OF SILTY SAND FROM BHUJ REGION IN INDIA

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ABSTRACT

Recent earthquake case histories have revealed the liquefaction of silty sands during earthquakes. A major challenge is the selection of the appropriate residual strength of liquefied materials for use in analyses to assess the post-liquefaction stability of embankments and other soil structures. During Bhuj earthquake on 26th January 2001, fine sand containing appreciable amount of silt were transported to the ground surface due to liquefaction induced upward ground water flow. Several sand-boils were observed near the region close to epicenter of the earthquake. This paper describes the results of an experimental study on the undrained shear behavior of such silty sands collected from the locations very close to the epicenter of Bhuj earthquake. The present study focuses on the effects of appreciable amount of silt content and confining stress on residual shear strength of silty sands in triaxial compression under monotonic loading and potential for liquefaction under cyclic loading.

KEYWORDS: Earthquake, Liquefaction, Liquefied Soil, Residual Strength

INTRODUCTION

Silty sands are the most common type of soil involved in both static and earthquake-induced liquefaction. This conclusion is based on the extensive literature, where documented historic cases of both static and earthquake liquefaction have been examined. Recent research clearly indicates that sand deposited with significant silt content is much more liquefiable than clean sands (Yamamuro and Lade, 1998) [18]. The high silt content has potential implications for liquefaction of level ground, hydraulic fills and earth dams. Further, the post-earthquake behavior of silty sand and consequently the stability of structures founded on liquefied soil, depend on the post-liquefaction shear strength of soil. The strength of soils mobilized at the Quasi-Steady State has important implications for engineering practice (Ishihara, 1993) [5]. Laboratory research, especially in the last five years, has contributed much to clarify the factors that control residual strength and provides some basis for selecting residual strength for design. However, the selection of appropriate residual strength for design and analysis is still not satisfactory (Finn, 2000) [4].

EXPERIMENTAL INVESTIGATION

The Bhuj earthquake struck the Kutch area in Gujarat State at 8.46 AM (IST) on January 26, 2001 with a magnitude of Mw 7.7. The epicenter of the quake was located at 23.4^o N, 70.28^o E and at a depth of about 25 Km to the north of Bacchau town. The earthquake caused almost a total devastation of several hundred buildings in villages and cities. In many places of the affected area, large masses of silty sands were ejected on to the ground surface. Liquefaction triggered by this earthquake caused major damage to structures resting on loose to medium dense sandy soils.

Selection of Material

Soil samples ejected on to the ground surface from the liquefied layer of the deposit were collected from two locations close to the epicenter of the earthquake. Figure 1 shows a typical location of liquefied ground and sample collection. Figure 2 shows the grain size distribution of the two soil samples and Table 1 gives the summary of the index properties of the same. In the present study, soil sample No.1 which contains 22% fines was used for testing.

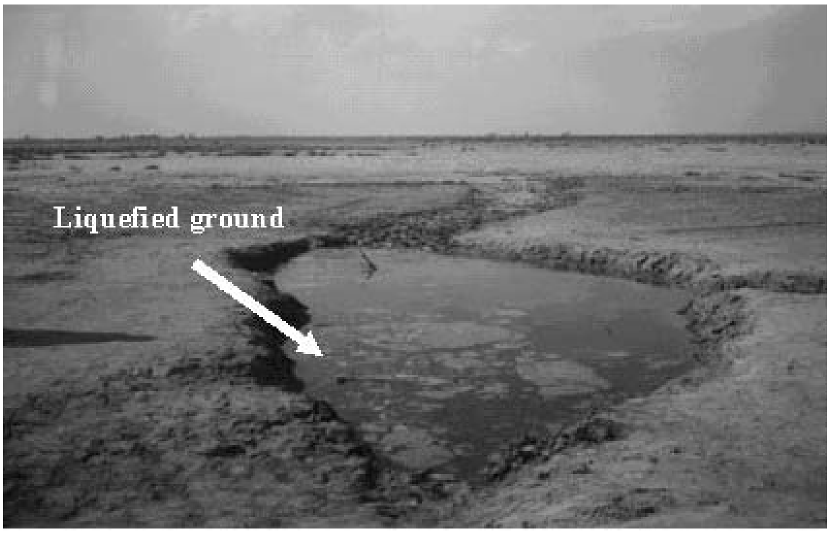

Fig. 1 Location of liquefied ground

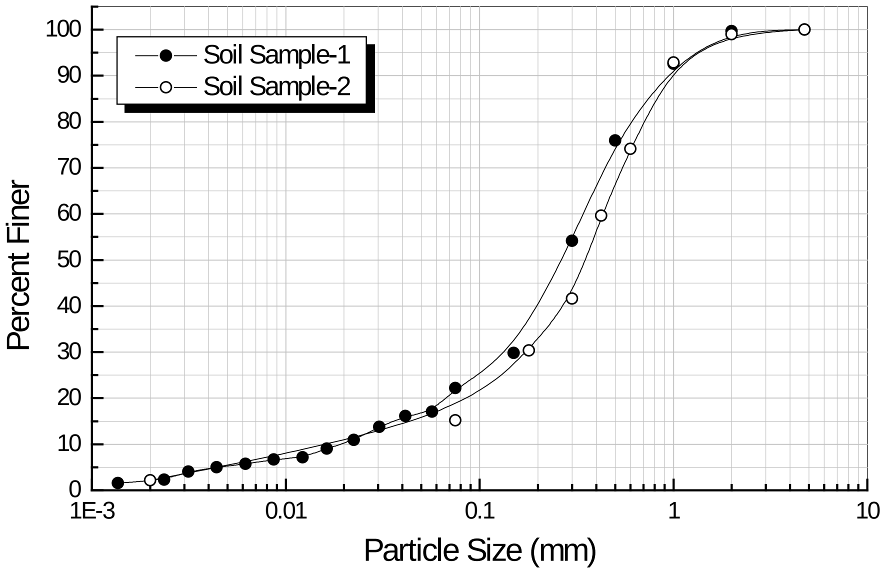

Fig. 2 Grain size distribution of soils

Table 1: Index Properties of Soil Samples

Index Property	Soil Sample No.1	Soil Sample No. 2
Specific Gravity	2.66	2.67
Coarse Sand (%)	-	1.0
Medium Sand (%)	35.0	39.42
Fine Sand (%)	43.0	44.44
Silt Content (%)	20.0	13.0
Clay Content (%)	2.0	2.14
Maximum Void Ratio (e_{max})	0.68	0.71
Minimum Void Ratio (e_{min})	0.42	0.37

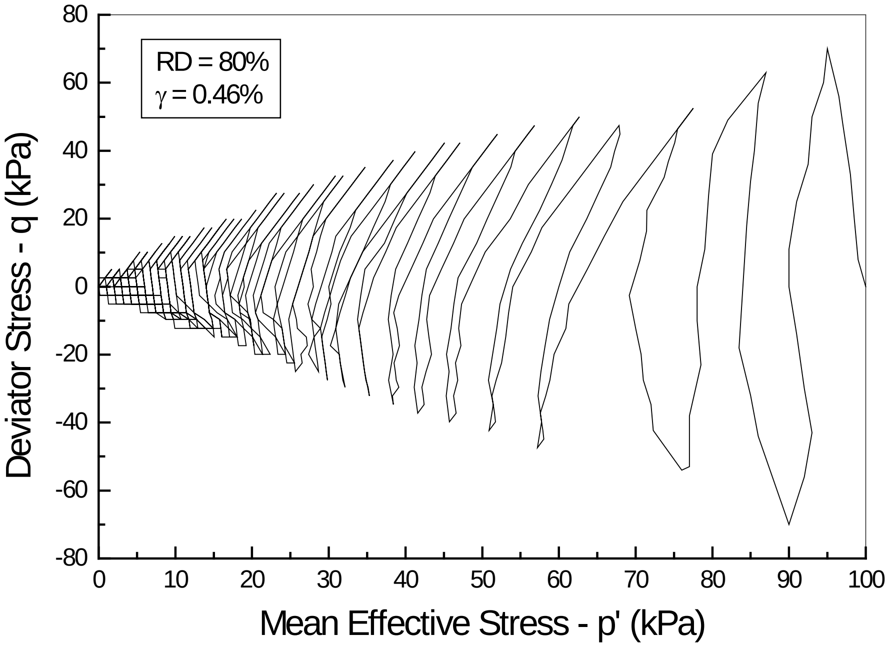

Fig. 3 Variation of mean stress with deviator stress

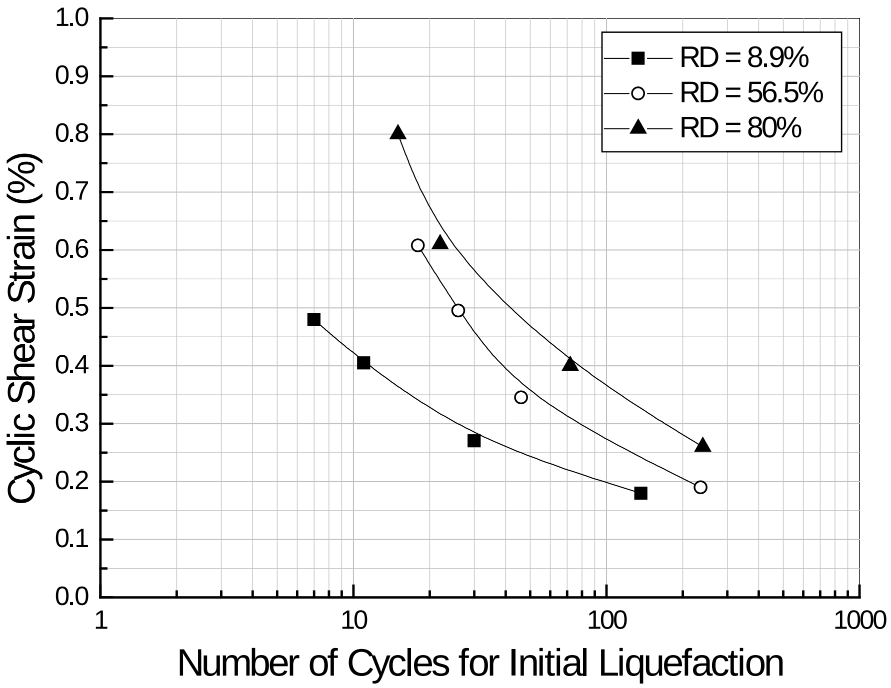

Fig. 4 Cyclic resistance curves

Sample Preparation

Dry pluviation has been shown to create a grain structure similar to that of naturally deposited river sands. Ishihara (1993) [5] has shown from tests on Tia Juana silty sand that the steady state line was nearly the same whether the specimens were prepared by water sedimentation or by dry deposition. Further, many of the water sedimentation depositional methods tend to produce inhomogeneous specimens with the coarser fraction on the bottom and the finer fraction on the top of the specimen (Lade and Yamamuro, 1997) [8]. In view of these observations, dry pluviation method was employed in the present study to prepare the soil samples for cyclic and monotonic testing.

Cylindrical soil specimens of size 50 mm diameter and 100 mm height were created by placing the dry silty sand in to a funnel with a tube attached to the spout. The tube was placed at the bottom of the membrane lined split mould. The tube was slowly raised along the axis of symmetry of the specimen, such that the soil was not allowed any drop height. This procedure was used to achieve the loosest possible density for a specimen prepared in a dry state. While preparing the soil specimens at relatively higher densities, the mould was gently tapped in a symmetrical pattern until the desired density was achieved. Using the above technique, soil specimens with three different target initial relative densities (RD_o) of 8.9%, 56.5% and 80% were prepared. After the specimens were prepared, a small vacuum pressure of 10 kPa was applied to the specimens to reduce disturbance during the removal of split mould and triaxial cell installation. The specimens were then saturated with deaired water using backpressure saturation. Saturation of the specimens was checked by measuring Skempton's pore pressure parameter B. Following saturation, the specimens were then isotropically consolidated to the required confining pressure.

UNDRAINED CYCLIC LOADING TESTS

Strain-controlled, cyclic triaxial tests were carried out on isotropically consolidated soil specimens under undrained conditions to simulate essentially undrained field conditions during earthquake. Cyclic loading was applied on the specimens using hydraulic actuator. The tests were conducted at a constant cyclic axial strain of varying magnitudes. In the entire test program, a frequency of 1Hz with sinusoidal wave and an effective confining pressure of 100 kPa were maintained. The axial deformation, cell pressure, cyclic load and pore water pressure were monitored using a built-in data acquisition system. The axial strain (ε_a) recorded from the cyclic triaxial tests was converted to cyclic shear strain (γ) (i.e. $\gamma = 1.5\ \varepsilon_a$) (Erten and Maher, 1995) [3]. Figure 3 shows a plot of variation of effective mean stress (p') with the deviator stress (q) for the soil at an initial relative density of 80% tested at cyclic shear strain (single amplitude) of 0.46%. Figure 4 represents the cyclic resistance in terms of cyclic shear strain (single amplitude) v/s number of cycles for different relative densities (RD).

Figure 5 shows the variation of cyclic resistance which is a measure of ability of soil to resist liquefaction (specified in terms of the magnitude of single amplitude cyclic shear strain in 20 cycles of uniform strain application) with relative density. The cyclic shear strains of 0.33%, 0.58% and 0.67% were obtained for initial relative densities 8.9%, 56.5% and 80% respectively. It may be seen in Figure 5 that the cyclic resistance increases with increase in relative density. Similar observations were made by Dobry et al. (1982) [2] from tests on Monterey No.0 sand.

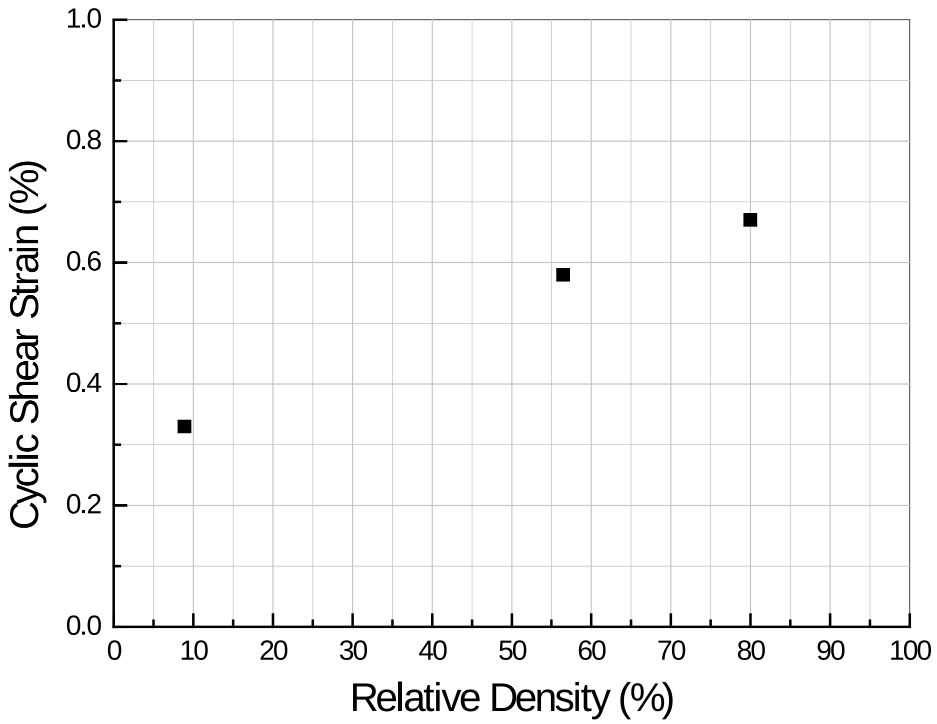

Fig. 5 Variation of cyclic resistance with relative density

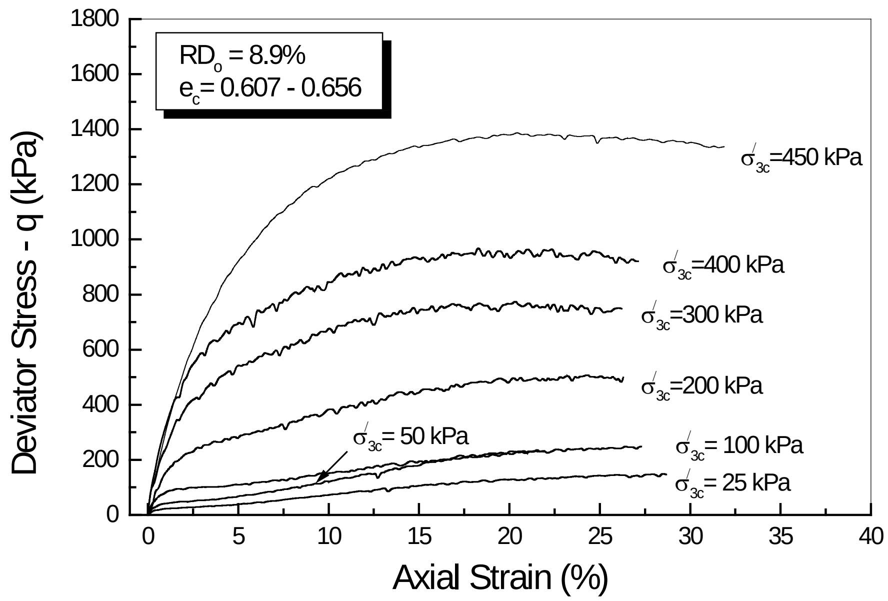

Fig. 6 Stress Strain relationship

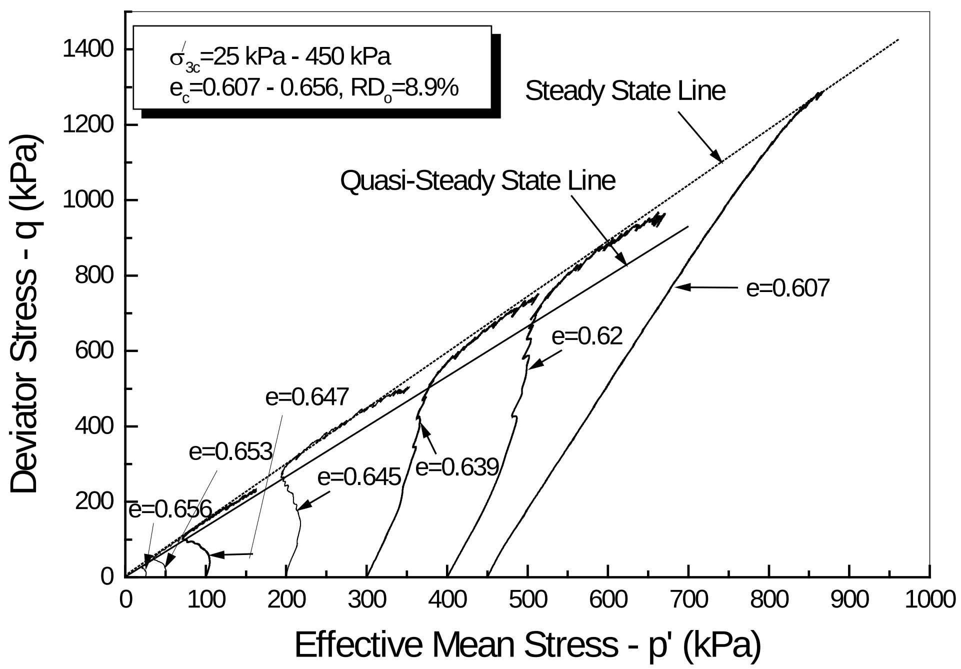

Fig. 7 Stress path diagram

UNDRAINED MONOTONIC LOADING TESTS

Strain controlled undrained triaxial compression tests were carried out on isotropically consolidated soil samples under monotonic loading with an axial strain rate of 0.6mm/minute. The specimens were sheared to large strains to obtain steady state conditions. The steady state is reached when the pore pressures remains constant under continued shearing. The stress-strain, stress-path and stress ratio plots for a soil sample with relative density of 8.9% are presented in figures 6, 7 and 8 respectively. Here, the effective mean stress is defined by $p' = (\sigma'_1 + 2\sigma'_3) / 3$, while the deviator stress by $q = (\sigma'_1 - \sigma'_3)$ and e_c denotes the void ratio of the sample after isotropic consolidation.

From the stress-strain plot (Figure 6), it can be observed that the quasi-steady state is reached at very low levels of shear stress over the axial strain range of 1% to 6%, where as the ultimate steady state is reached at higher shear stresses in the axial strain range in excess of 25%. This behaviour is in close agreement with the experimental results of the studies conducted by Ishihara (1993) [6] on Tia Juana silty sand.

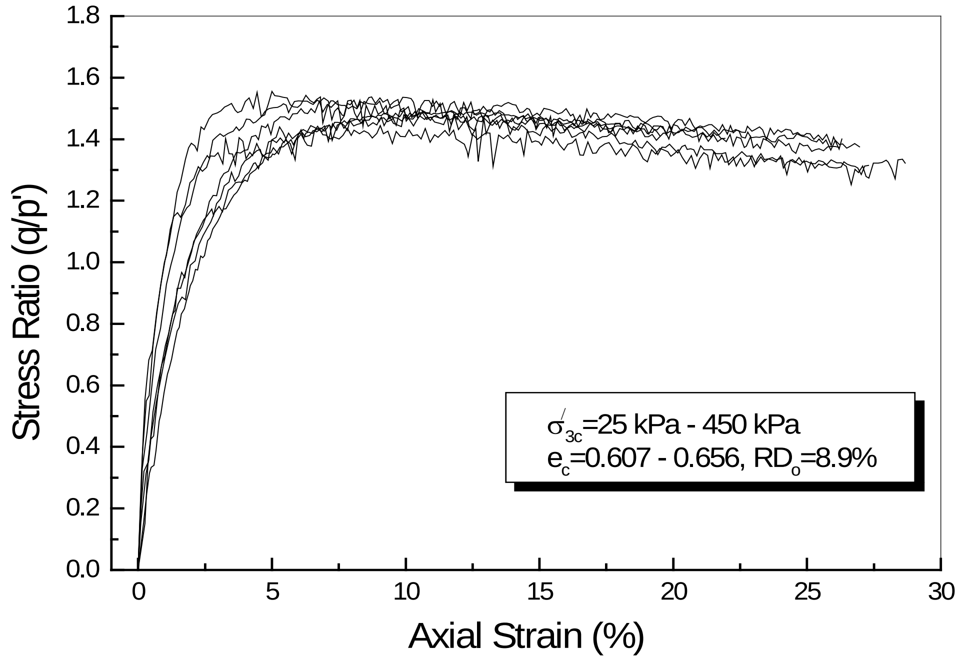

Fig. 8 Variation of stress ratio with axial strain

Undrained compression and extension tests on Brenda mine tailings sand (graded between the Nos.20 and 200 sieve sizes) were conducted at a confining pressure of 350 kPa by Kuerbis et al. (1988) [8] by varying the silt content between 0% and 22.3%. They concluded that the presence of fines in the void spaces makes the soil slightly more dilative. Pitman et al. (1994) [13] also performed undrained triaxial compression test on Ottawa sand (C109) with non-plastic crushed quartz fines and plastic kaolinite fines at an initial confining pressure of 35 kPa. They varied fines content from 0 to 40%. Based on test results, it was concluded that the effect of non-plastic fines was to create a slightly more dilative response.

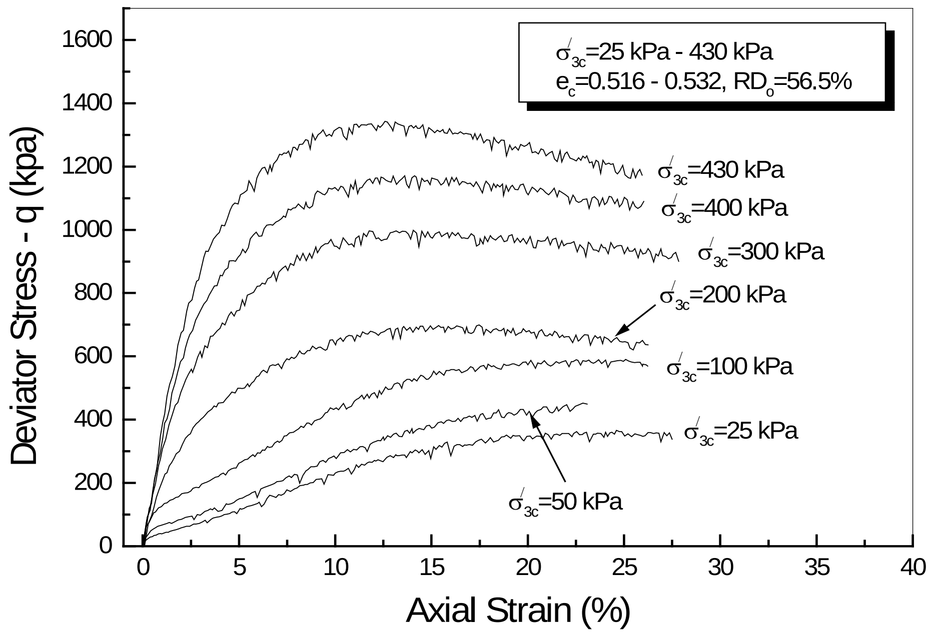

Fig. 9 Stress strain relationship

It can be seen from the Figure 7, that the effective confining stress has major influence in changing the behavior of silty sand from contractive to dilative. Silty sands in the void ratio range of 0.607 to 0.656 showing drastic reduction in strength at lower confining stresses exhibit more dilative behavior at higher confining stresses. This indicates that the silty sands are more stable at higher confining stresses. This represents a different behavior when compared to the behavior of clean sands in loose state. Similar effects were reported by Yamamuro and Lade (1997 &1998) [18] & [19] from tests on Nevada sand with 7% fines at a relative density of 30% and Nevada sand with 6% fines at a relative density of 12%.

Figures 9, 10 and 11 show the stress-strain, stress-path and stress ratio v/s axial strain plots of a specimen prepared initially at a relative density of 56.5%. It is clear from Figure 10 that, as the void ratio decreases (increase in relative density) silty sand exhibits more dilating behavior. This demonstrates the dependency of both void ratio and confining stress on the undrained behavior of silty sands.

Only dilative behavior can be noticed from Figure 13 in the void ratio range of 0.462 to 0.469 at an initial relative density of 80%.

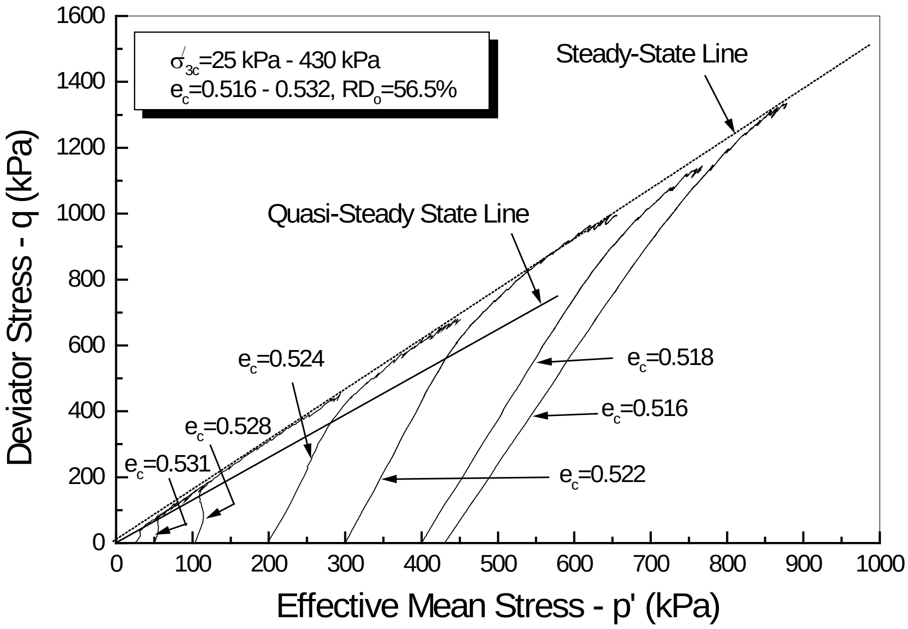

Fig. 10 Stress path diagram

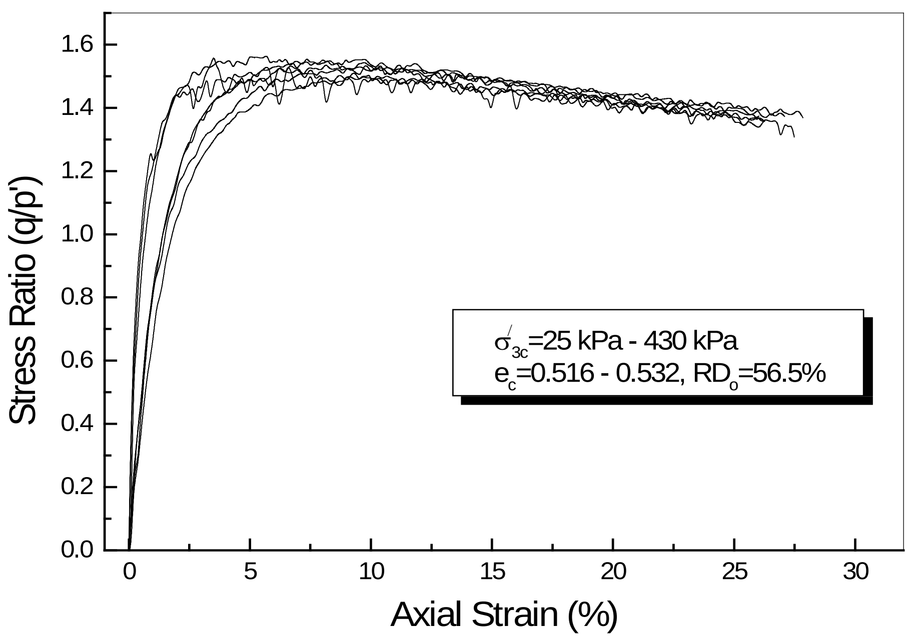

Fig. 11 Variation of stress ratio with axial strain

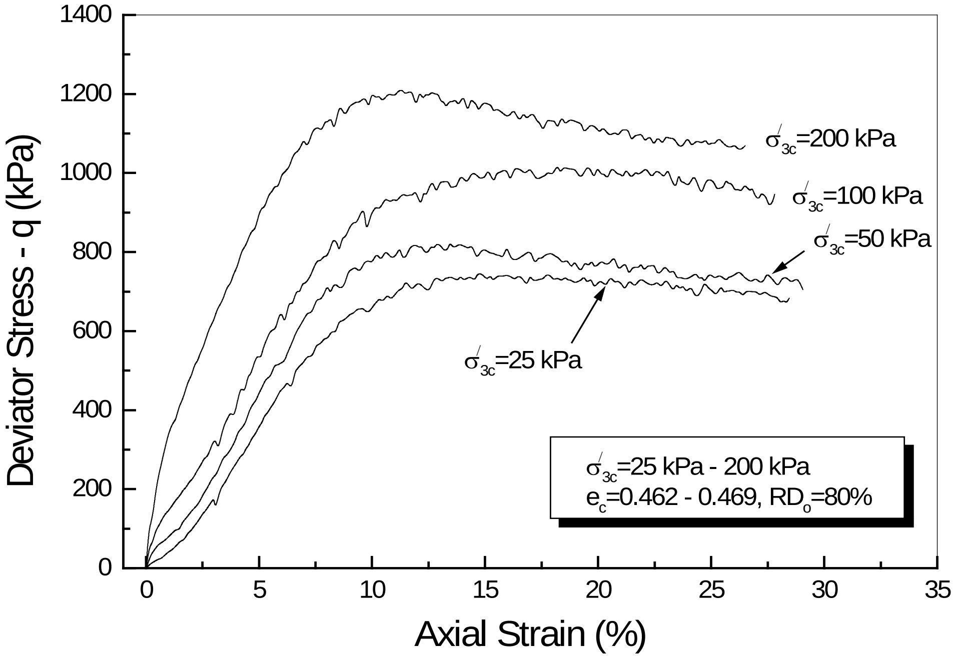

Fig. 12 Stress strain relationship

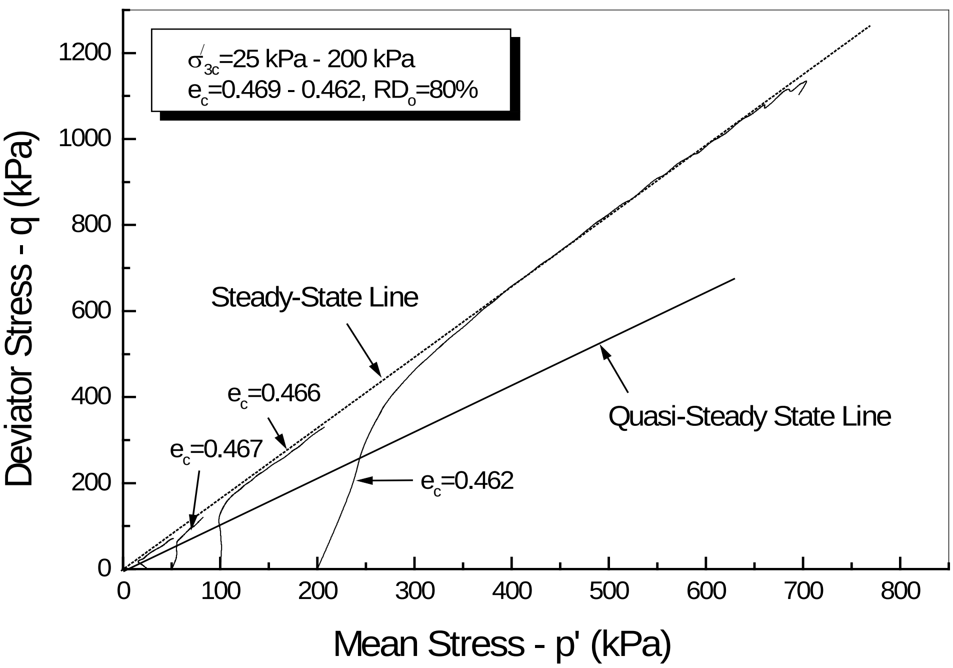

Fig. 13 Stress path diagram

RESIDUAL STRENGTH

When relatively loose sand is strained in undrained shearing beyond the point of peak strength, the undrained strength drops to a near constant value over a large strain. This strength is conventionally called the undrained steady state strength or residual strength. However, if the strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction. Even limited liquefaction may result in significant deformations and associated drop in strength. The residual strength S_{us} may be defined [6] as

$$S_{us} = (q_s /2) \cos\phi_s \qquad (1)$$

where q_s and ϕ_s indicate the deviator stress and the mobilized angle of interparticle friction at the quasi-steady state. For the undrained tests carried out at various confining pressures and the initial state, the deviator stress (q_s) was estimated at quasi-steady state point along with the mobilized friction angle. Further the residual strength was estimated using Equation (1). Figure 15 shows the evaluated residual strength and its variation with void ratio and confining stress. The residual strength decreases with increase in the void ratio of sample. Thevanayagam (1998) [16] observed from experiments on silty sands the similar tendency of decreasing residual strength with increasing void ratio. Further, the residual strength increases with increasing effective confining pressure.

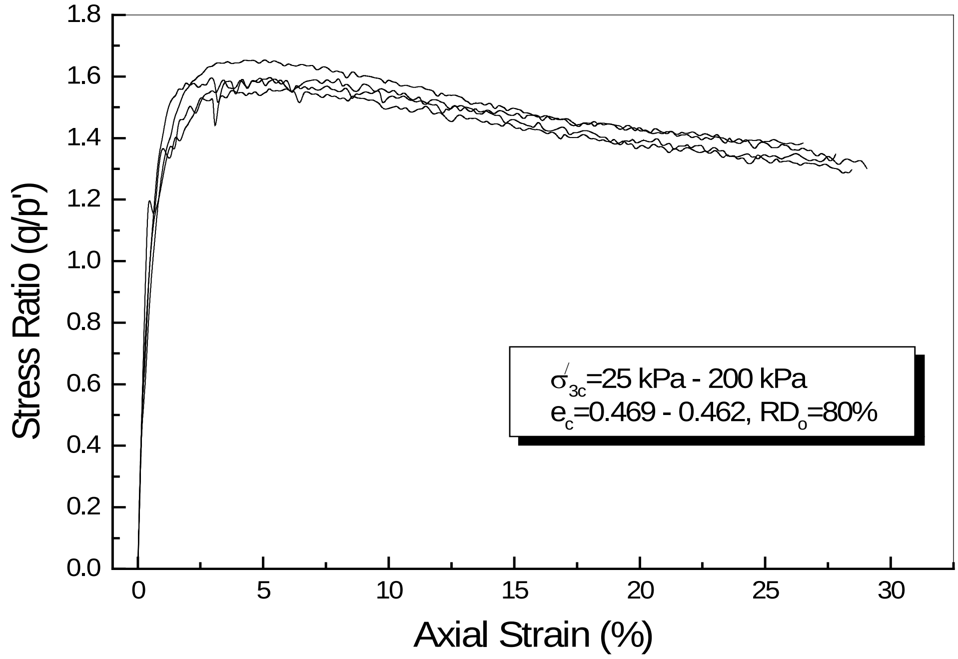

Fig. 14 Variation of stress ratio with axial strain

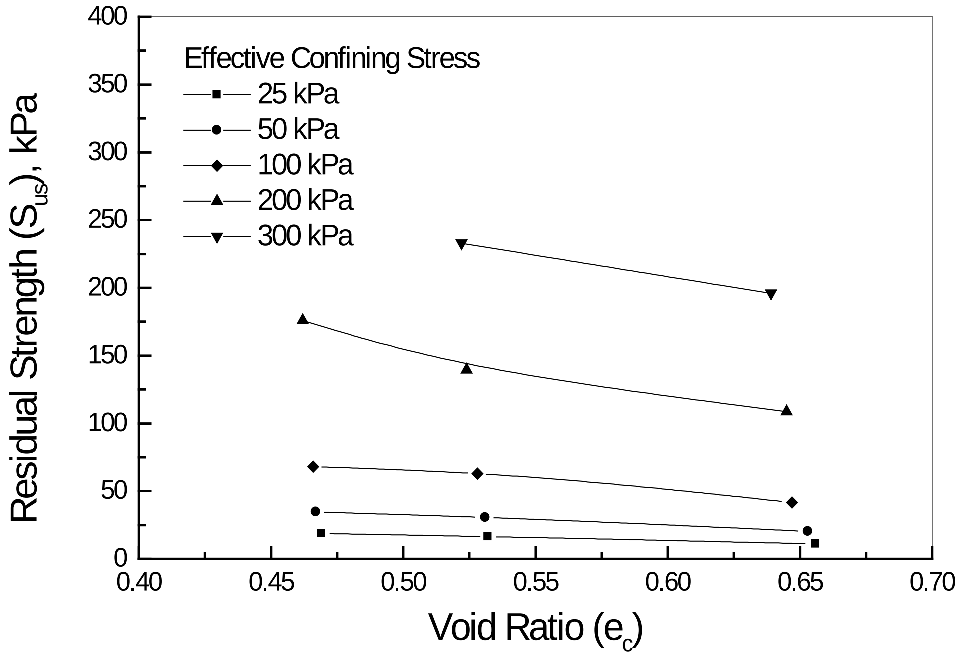

Fig. 15 Variation of residual strength with void ratio and confining stress

Figure 16 shows the normalized residual strength with void ratio. The preliminary results show a greater scatter and it may be attributed to the initial fabric of the sample. Also shown in the Figure 17 is the relationship between relative density (RD_c) and the residual strength (S_{us}) at various confining pressures. It is clear from this figure that an increase in the relative density results in an increase in the residual strength at a given confining pressure. Thevanayagam et al. (1997) [15] from their experimental studies on undrained strength of silty sand (12% to 32% fines) at a confining pressure of 100 kPa, report similar behavior of increasing residual strength with increasing relative density. They also conclude that the possible influence of initial confining stress on the steady state strength was not included in their studies. But the present studies focus on the effect of confining stress on the steady state strength of silty sands at a given initial relative density. It can be noticed from the results of the study that there is a significant increase in the steady state strength with increase in the confining stresses for a given initial relative density. This aspect of the current study is in agreement with the experimental results reported by Ishihara (1993) [6] on Tia Juana silty sand, Baziar & Dobry (1995) [10] on silty sands retrieved from the Lower San Fernando Dam and Naeini & Baziar (2004) [11] on Ardebil sand with different amount of fines content.

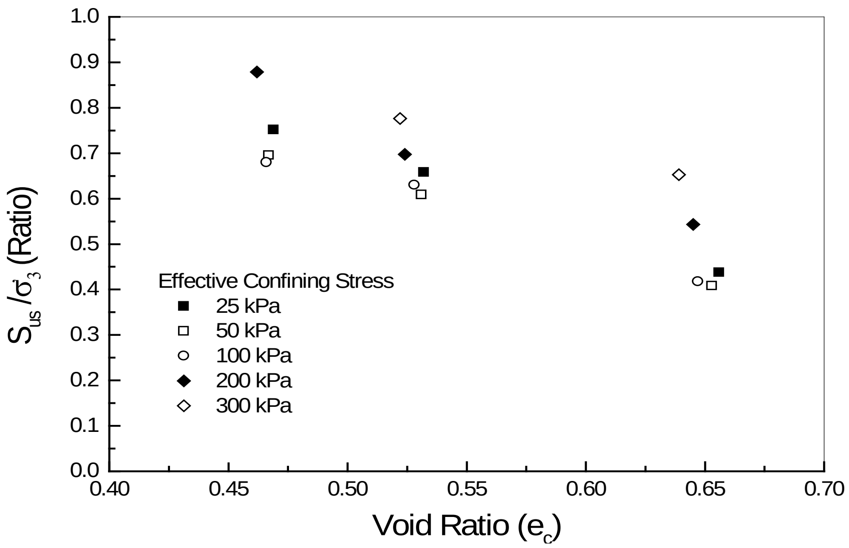

Fig. 16 Variation of normalized residual strength with void ratio

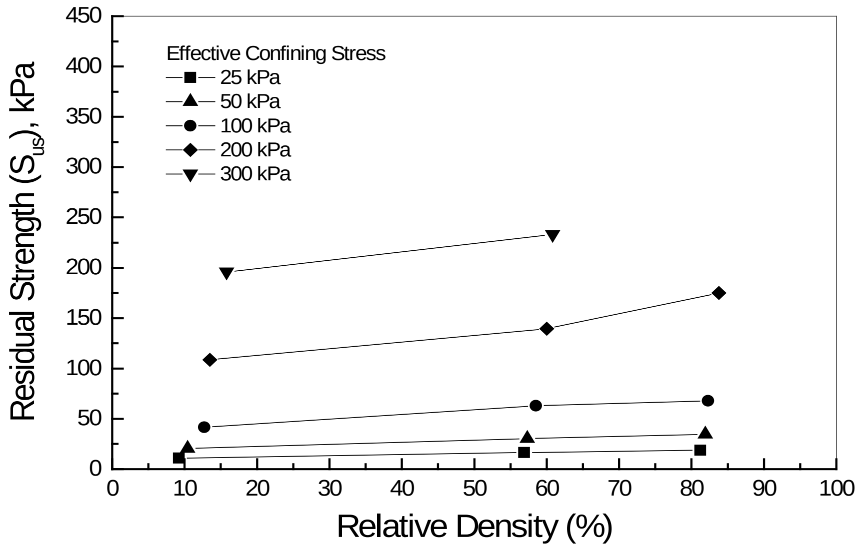

Fig. 17 Variation of residual strength with relative density

POST LIQUEFACTION SETTLEMENT

When a saturated sand deposit is subjected to shaking during an earthquake, pore water pressure builds up leading to liquefaction in the sand deposit. Then the pore water pressure dissipates inducing volume change in the sand deposit reflecting on the ground surface as settlements.

To simulate the field condition, undrained cyclic triaxial tests with two constant strain amplitudes of 0.74% and 1.8% were carried out (on samples with relative density of 8.9%, sine wave load and 1 Hz frequency at an effective confining pressure of 100 kPa) and the tests were continued till the samples liquefied. Further, the drainage line for samples was opened to dissipate the developed excess pore water pressure and the volume changes of the samples were recorded. Table 2 gives the results of post liquefaction volume changes in terms of volumetric strains for the two samples. It is evident that the magnitude of volumetric strain depends on the amplitude of the shear strain. From these limited data it can be inferred that the post liquefaction volumetric strain is influenced by the maximum shear strain that the soil has undergone during the application of seismic load. Similar results have been reported by Tatsuoka et al. (1984) [14], Tokimatsu & Seed (1987) [17], Nagase & Ishihara (1988) [12], Ishihara & Yoshimine (1992) [7], Ishihara (1993) [6] and Hatanaka et al. (1997) [5]. Detailed studies to investigate the effect of relative density and confining pressures along with a wide range of applied shear strain amplitudes are being carried out to understand the post liquefaction behavior of Bhuj sands.

Table 2: Applied Shear Strains and Post Liquefaction Volumetric Strains

Applied Amplitude of Shear Strain during cyclic shear test (γ)	**Volumetric Strain (ε_v)**
0.74 %	1.935 %
1.80 %	2.757 %

CONCLUDING REMARKS

A series of undrained triaxial compression tests in monotonic and cyclic loading conditions were performed on silty sand collected from liquefied sites at Bhuj, India with appreciable amount of silt content (20%). At higher cyclic shear strains, the liquefaction occurs at less number of cycles. An increase in the density results in an increase in the cyclic strength of the soil there by making it less susceptible to liquefaction at lower dynamic shear strain amplitudes. Undrained tests performed with initial confining pressures between 25 kpa and 450 kPa showed contractive behavior at lower confining pressures in the void ratio range tested. An increase in the confining pressure results in an increase in the residual strength at a given void ratio or relative density. Based on the limited tests for the evaluation of post liquefaction settlement in terms of volumetric strains, it is illustrated that the magnitude of post liquefaction volumetric strains are governed by the amplitude of shear strain applied during the cyclic test.

ACKNOWLEDGMENT

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