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Deformation characteristics of sandy soils subjected to cyclic loads

Caractéristiques de déformation de sols sableux soumis a des contraintes cycliques

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ABSTRACT
Large displacements and significant degradation of soil stiffness and strength, compression and liquefaction of loose sands have been observed during and after the earthquakes. The response of soils at medium to large strain levels that produce significant amount of deformation has been one of the major concerns for engineers working in this area. Extensive studies have been carried out on the deformation characteristics of saturated sands during and after earthquakes. However, not much work has been reported on the deformation characteristics of dry and partly saturated sands subjected to seismic loads. This paper summarizes the results of cyclic triaxial element tests on sands collected from the earthquake-affected area of Ahmedabad city. The results from cyclic triaxial testing facility clearly highlight that Ahmedabad sands exhibit higher modulus values at lower saturation levels (up to about 10%) compared to dry and completely saturated sands for the range of shear strain tested. Beyond 10% saturation level, no significant increase in the shear modulus is observed. Dry sands exhibit slightly higher damping ratios in comparison with partly and fully saturated sands. However, the damping values fall in a narrow band in the range of shear strain tested for dry, partly and completely saturated Ahmedabad soils.

RÉSUMÉ
Avant et après les tremblements de terre, on a observe d'importants déplacements de sols, des dégradations significatives des propriétés rhéologiques des sols, ainsi que la compression et la liquéfaction de sables. La réponse des sols à des niveaux de contrainte moyens a forts qui produisent des déformations significatives a été l'une des préoccupations principales des ingénieurs travaillant dans ce domaine. De nombreuses études ont été menées sur les caractéristiques de déformation des sables satures pendant et après les tremblements de terre. Cependant, peu d'études concernent les caractéristiques de déformation des sables secs ou partiellement saturés soumis à des contraintes sismiques. Cet article résume les résultats de tests d'éléments cycliques triaxiaux sur des sables collectés sur des zones affectées par des tremblements de terre dans la ville d'Ahmedabad. Les résultats montrent que les sables d'Ahmedabad présentent une valeur de module plus grande aux faibles saturations (jusqu'à 10%) qu'a l'état sec ou complètement saturé, pour la gamme de contraintes de cisaillement testée. En dessous de 10% de saturation, aucune augmentation significative du module de cisaillement n'est observée. Les sables secs présentent un rapport d'amortissements légèrement plus grands que les sables complètement ou partiellement saturés. Cependant, les valeurs d'amortissement varient peu dans la gamme de contraintes de cisaillement testée pour les sols d'Ahmedabad secs, partiellement ou complètement saturés.

1 INTRODUCTION

Understanding the seismic behaviour of soils, evaluation of dynamic properties of soils and prediction of ground settlements due to seismic shaking has been the major task in the earthquake geotechnical engineering. Sandy soils in the field can exit in dry, partly saturated or fully saturated conditions. Dry sands densify quickly and settlement is usually completed at the end of the earthquake resulting in differential settlements of structures resting on such soils. The settlement of partly saturated soils (up to about 70%) is also completed during the period of earthquake, whereas in the case of loose saturated sand deposits under undrained conditions, buildup of excess pore water pressure during earthquake cause a complete loss in shear strength. Further, the dissipation of excess hydrostatic pressure induced in the soil by the seismic shaking results in large settlements of ground surface known as post liquefaction settlement.

The recent Bhuj earthquake on January 26, 2001 having a magnitude of M_w = 7.7 caused major damages to constructed facilities in Ahmedabad city which is about 300 Kms away from the epicenter. The city is founded over deep deposits of cohesion less sandy soils. A number of medium to high-rise residential apartments collapsed in Ahmedabad city. The random distribution of such damage has been recorded from a number of localities scattered on the left and right banks of Sabarmati river (Bhandari and Sharma, 2001). GovindaRaju et al, (2004) show that the varying degree of such damage to multistory buildings close to Sabarmati river area in Ahmedabad city was essentially due to amplification of the ground and also due to the fact that natural frequencies of the buildings/structures and frequency content of the ground motion are in close proximity. In order to evaluate such ground response in the zone of interest, it becomes essential to evaluate site-specific dynamic properties of such soils in dry, partly saturated and completely saturated conditions. This paper presents the results of an experimental study on the deformation characteristics of soils

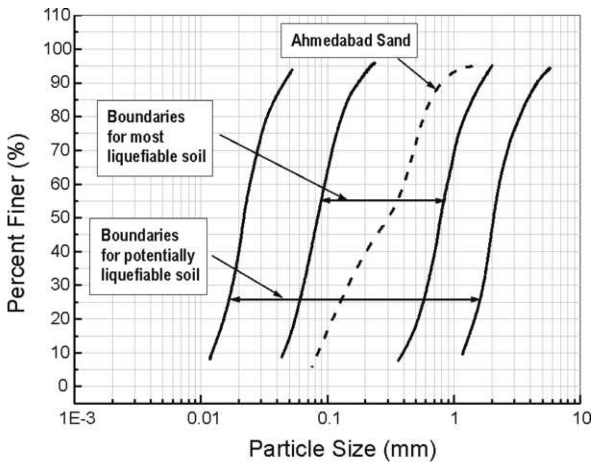

Fig.1 Grain size distribution of soil from Ahmedabad

from Ahmedabad city in dry, partly saturated and completely saturated conditions subjected to cyclic loads.

2 EXPERIMENTAL INVESTIGATION

2.1 *Soil sampling and characterization*

Representative sand samples used in this study were collected from the earthquake-affected area in Ahmedabad city, Gujarat State, India. Figure 1 shows the grain size distribution of the soil sample collected. Also shown in the Fig, the ranges of grain size distribution for liquefaction susceptible soils proposed by Tsuchida (Xenaki and Athanasopoulos, 2003). Fig.1 clearly highlights that Ahmedabad sands fall well with in the range of most liquefiable soils. Table 1 gives the characteristics of the sand sample. It is to be noted that the Ahmedabad sand consists of about 10% of non-cohesive silt size particles.

Table 1. Index properties of soil sample

Specific Gravity	2.66
Medium Sand	37%
Fine Sand	53.4%
Silt Content	9.6%
Maximum Void Ratio	0.67
Specific Gravity	2.66

2.2 *Cyclic Triaxial Testing Equipment*

The cyclic triaxial testing equipment used in this study is an automated triaxial testing system with six sensors, including a load cell to monitor the axial load, an LVDT to measure the vertical displacement, four transducers to detect the chamber pressure, pore pressure, volume change and lateral deformation. The triaxial cell is built with low friction piston rod seal along with a 10 kN servo – controlled submersible load cell. The loading system consists of a load frame and hydraulic actuator capable of performing strain-controlled as well as stress-controlled tests with wide range of frequency of 0.01 Hz to 10 Hz. The specimens of size 38 mm, 50 mm, 75 mm and 100 mm diameter can be tested with confining stresses up to 1000 kPa.

2.3 *Sample Preparation*

Dry soil sample specimens of size 50 mm diameter and 100 mm height with different relative densities were prepared by pouring the pre weighed amount of dry sand through a funnel whose tube was placed at the bottom of the membrane lined split mould. The funnel was slowly raised along the axis of symmetry of the specimen and taping gently to the volume of the mould to achieve the desired density. The partly saturated sand samples with degree of saturation of 10%, 30% and 50% were prepared for a relative density of 30% by adding calculated amount of water and mixing thoroughly. The wet sand was placed in the membrane lined split mould in 3 equal layers by compacting. In case of saturated sand samples, initially dry sand specimens were prepared in a similar manner as above and then saturated with de-aired water using backpressure saturation. The backpressure was increased gradually while maintaining the effective confining pressure at 15 to 20 kPa. This process was continued until the pore pressure parameter B ($=\Delta u/\Delta\sigma_c$, Δu= change in specimen pore pressure, $\Delta\sigma_c$ = change in confining pressure) exceeds 0.95. Following saturation, the sand specimens were consolidated to a required effective isotropic consolidation stress.

2.4 *Cyclic Loading and Data Calculation*

Cyclic loading in the vertical direction was applied directly on the soil samples using strain-controlled method. The tests were carried out with different strain amplitudes and confining pressures. In the test program, a sinusoidal harmonic loading was applied on soil samples with a frequency of 0.1 Hz and 1.0 Hz. A typical hysteresis loop during a cyclic triaxial test is as shown in figure 2 (Sitharam et al, 2004). The slope of the secant line connecting the extreme points on the hysteresis loop is the dynamic Young's modulus, E, which is given by: $E = \sigma_d/\varepsilon$. In this paper the axial strain and the Young's modulus were converted to values of shear strain and shear modulus from the theory of elasticity.

$\gamma = (1+\upsilon)\varepsilon$ and $G = E/2(1+\upsilon)$

where G is the (secant) shear modulus, γ is the shear strain and ν is the Poisson's ratio, which is taken as 0.4 (Marshal and Park, 1975) for dry and partly saturated sands where as for saturated sands it is taken as 0.5 (Sitharam et al, 2004).

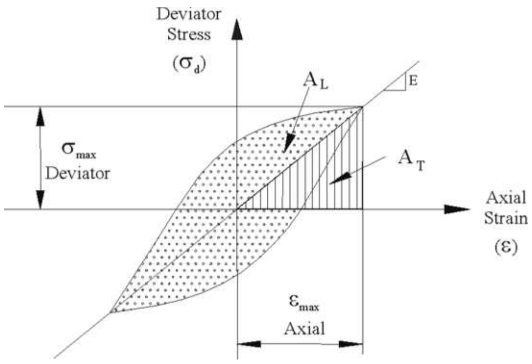

Fig. 2 Typical hysteresis loop

The damping ratio, D (Hysteretic damping) is a measure of dissipated energy versus elastic energy and may be calculated from the equation:

$D = A_L/(4\pi A_T)$

where, A_L is the area enclosed by the hysteresis loop and A_T is the area of the shaded triangle.

2.5 *Post Liquefaction Volumetric Strain*

During an earthquake when a saturated sand deposit is subjected to shaking, pore water pressure builds up leading to liquefaction in the sand deposit. Thereafter the pore water pressure dissipates inducing volume change in the sand deposit and reflecting on the ground surface as settlements. To simulate the field condition, undrained cyclic triaxial tests were continued till the samples liquefied and 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.

3 RESULTS AND DISCUSSIONS

Figure 3 show the relationship between shear modulus and shear strain for dry soil samples prepared at relative densities of 20%, 50% and 70% under a confining pressure of 25 kPa and frequency of 0.1 Hz. It is evident from the figure 3 that the shear modulus increases with increase in relative density at lower strain levels and practically remains constant at large strain values irrespective of relative density. However, the variation in the damping ratios with increase in relative density is not significant in the range of shear strain tested (Fig. 4). Similar behaviour can be noticed from the results of soil sample at 100 kPa confining pressure (Fig. 5 and 6). Here, the increase in the shear modulus values at higher confining pressure is significant at low strain levels when compared to the results at 25 kPa confining pressure.

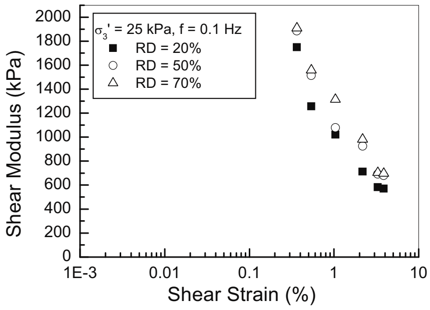

Fig. 3 Variation of shear modulus with shear strain for a confining pressure of 25 kPa-dry sand

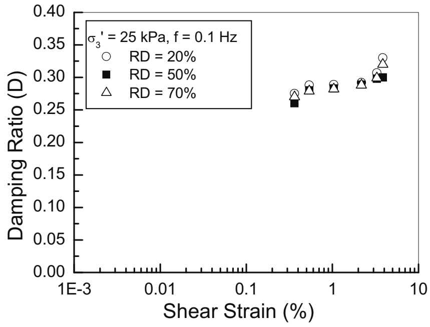

Fig. 4 Variation of damping ratio with shear strain for a confining pressure of 25 kPa – dry sands

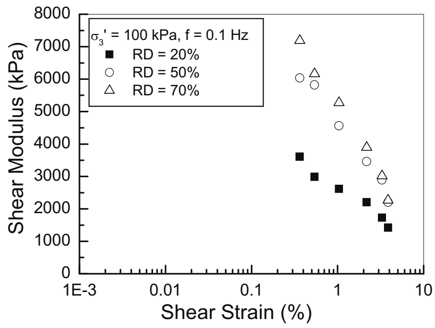

Fig. 5 Variation of shear modulus with shear strain for a confining pressure of 100 kPa – dry sands

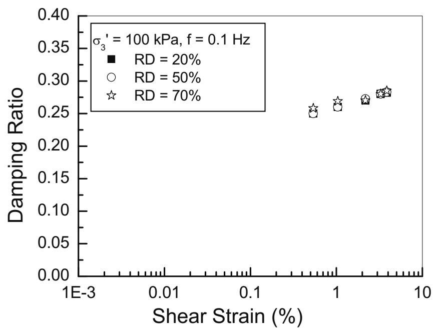

Fig. 6 Variation of damping ratio with shear strain for a confining pressure of 100 kPa – dry sands

Figure 7 and 8 shows the effect of confining pressure on the dynamic properties of dry sand prepared at a relative density of 20% and subjected to 0.1 Hz frequency. The effect of confining pressure on the stress strain relation is evident from Fig. 7. It is clear that the shear modulus increases substantially with increase in the confining pressure in the range of shear strain tested. However, even though the increase in the damping ratios is observed with increase in confining pressure, the increase is not significant (Fig. 8).

On partly saturated samples, the strain controlled triaxial cyclic tests were carried out similar to dry sands as explained earlier. Figures 9 and 10 depict the effect of degree of saturation on the dynamic properties of sands at a relative density of 30%, frequency of loading 1 Hz and confining pressure of 100 kPa.

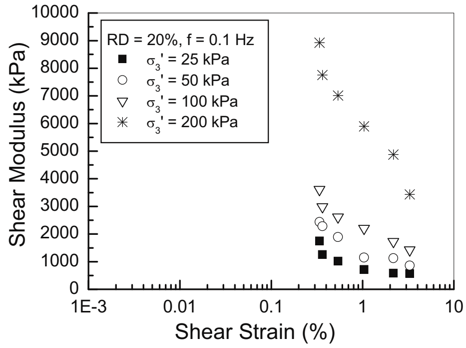

Fig. 7 Variation of shear modulus with shear strain for relative density 20% -dry sands

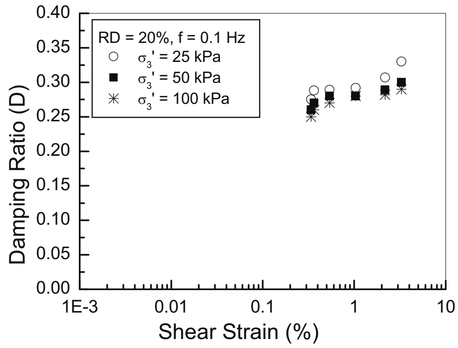

Fig. 8 Variation of damping ratio with shear strain for relative density 20% - dry sands

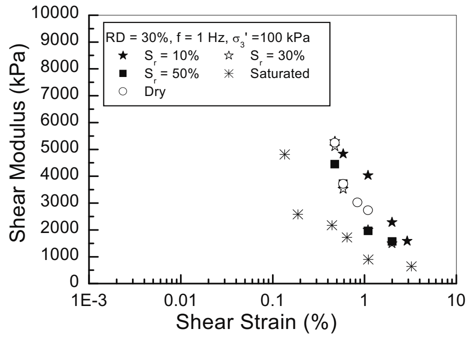

Fig. 9 variation of shear modulus for dry, partly saturated and saturated sands

As seen from the figure 9, sands exhibit higher modulus values at lower saturation levels (at about 10%) compared to dry and completely saturated sands. Beyond 10% saturation level, no significant increase in the shear modulus is observed. As observed from the figure 10, it is evident that dry sands exhibit slightly higher damping ratios in comparison with partly and fully saturated sands. However, the damping values fall in a narrow band in the range of shear strain tested for dry, partly and completely saturated soils.

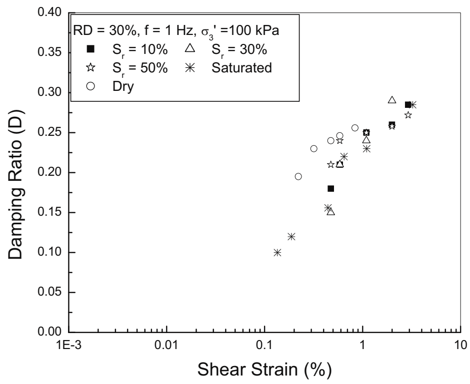

Fig. 10 Variation of damping ratio with shear strain for dry, partly saturated and saturated sands

As observed from the figure11, it is clear that the magnitude of post liquefaction volumetric strain depends on the amplitude of the shear strain. As the shear strain increases, the post liquefaction volumetric strains increase significantly at lower strain amplitudes (up to 1.5 %) and thereafter not much variation is observed. It is also evident from the results that volumetric strains are a function of relative density of sand. As the relative density of the sample increases, the volumetric strains decrease at a given shear strain amplitude.

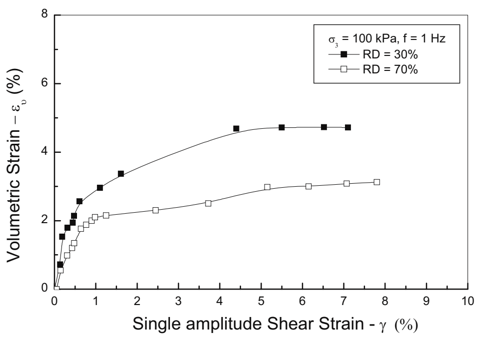

Fig. 11 variation of volumetric strain with single amplitude shear strain

Similar results have been reported by many researchers (Tokimatsu and Seed, 1987 and Ishihara, 1993). 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 saturated sands.

4 CONCLUSIONS

From the test results on Ahmedabad sands using strain controlled cyclic triaxial tests, the following conclusions are drawn:

i. Strain controlled cyclic triaxial tests on dry sands reveal that shear modulus increases with increase in relative density at lower strain levels and practically remains constant at large strain values irrespective of relative density. However, the variation in the damping ratios with increase in relative density is not significant in the range of shear strain tested. Shear modulus increases substantially with increase in the confining pressure in the range of shear strain tested. However, there is no significant increase in the damping ratio with increase in confining pressure.

ii. Ahmedabad sands exhibit higher modulus values at lower saturation levels (about 10%) compared to dry and completely saturated sands. Beyond 10% saturation level, no significant increase in the shear modulus is observed. Dry sands exhibit slightly higher damping ratios in comparison with partly and fully saturated sands. However, the damping values fall in a narrow band in the range of shear strain tested for dry, partly and completely saturated soils.

iii. Post liquefaction volumetric strain in saturated sands is a function of shear strain and relative density of soil. As the shear strain increases, the post liquefaction volumetric strains increase significantly at lower strain amplitudes (up to 1.5 %) and thereafter not much variation is observed. Further, the volumetric strains decrease as the relative density of soil increases for a given shear strain level.

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