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CYCIC BEHAVIOUR OF SAND: SEQUENTIAL INCREASING PEAK STRESS RATIO AND σ_z BEHAVIOUR

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ABSTRACT

The strength and deformation behaviour of dense and loose samples of Leighton Buzzard sand were examined by means of cyclic biaxial loading tests in which the stress orthogonal to the plane of strain was initially applied at different magnitudes of σ_z=20, 50 and 80 kPa. The experimental results showed that the general trend of volume change during cyclic stressing is to cause densification and build of structure (fabric force). Also it was revealed that after many cycles the values of σ_z are independent of their initial values and settle very close to the mean principal biaxial stress.

INTRODUCTION

The tendency of granular material to densify under cyclic loading with 90° changes in the principal stress direction, has been recognized as the primary reason of the build-up in pore water pressure which leads to liquefaction. Therefore a better understanding of soil response subjected to cyclic loading is essential. However, undrained cyclic tests, to assess pore pressure and resistance of soils to liquefaction do not reveal the stress-strain behaviour of sands in drained cyclic conditions [1]. In addition, there are many situations where granular soil is subjected to slow cyclic stress changes under drained conditions. Two examples, the filter bed problem and the abutment wall, have been investigated [2]. This paper describes the investigation into the strength and deformation behavior of dense and loose sands when subjected to cyclic stressing in which the peak stress ratio was sequentially increased when a repeating state was observed over many cycles.

MATERIAL

Standard Leighton Buzzard sand (BS sieve sizes 18-25; 850-600 μm) with a specific gravity of 2.66 was used for the present investigation. Two different methods of preparing sand samples, slow raining through air and releasing a large quantity of sand from a height of 750 mm above the ground level, resulted in dense and loose sand samples with initial voids ratio of e_0= 0.52±0.01 and e_0=0.72±0.01 respectively. The sand samples were formed in a cubic rubber membrane, supported by a perspex box. All samples were deposited along a direction, namely Z, that would subsequently be orthogonal to the plane of strain. Thus, samples were always isotropic in the plane of strain. The samples were 100 mm cubes confined in thin rubber membrane which were then sealed and the air in the voids evacuated to give the sample strength and enable the removal the sample from the preparation box and the positioning of it in the apparatus. This was achieved by applying a negative pressure to the sample, an isotropic stress of around 50 kPa. Sand samples to be considered were dry and subsequently all stresses were effective stress. For convenience, therefore, the small dashed notation for effective stress has been omitted.

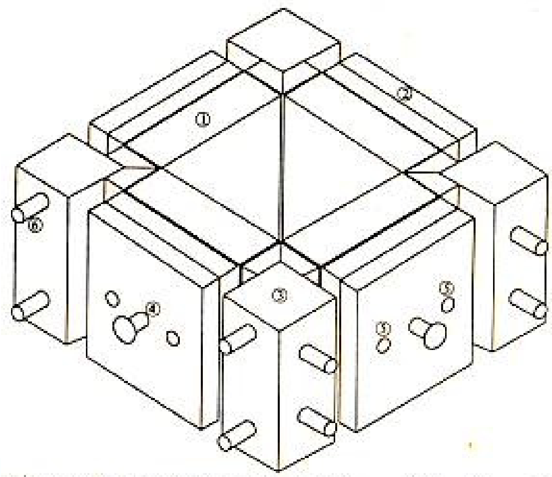

Figure-1: Isometric representation of the Biaxial Tester.

1) Latex rubber pressure bag, the front of which forms the flexible loading boundary.
2) Pressure bag packing plate.
3) Corner retaining guide block.
4) Rigid adjustable screw of make and break contact.
5) Holes for supply and measurement of pressure to the loading face.
6) Steel bars used to couple movement of backing plates with adjacent corner retaining guide blocks.

APPARATUS

The Biaxial Tester, Figure 1, which has been fully described elsewhere [3] and [4] shears soil under plane strain conditions. The principal stresses in the plane of strain are applied to the cubic soil samples through flexible boundaries with no trace of shear stress. The Biaxial Tester was developed to accommodate large sample deformations whilst maintaining uniform boundaries stresses. It employs a concept in which any incremental sample deformation resulting from small changes in applied stresses, is matched with a corresponding apparatus boundary adjustment perpendicular to the applied load direction. This provides for uniform stress and strain throughout the sample. In addition to 90° jump rotations of the principal stress direction, the Biaxial Tester offers the ability to apply/record σ_z.

STRESS PATHS

The stresses applied to the sample through pressure rubber bags in the plane of strain are called σ_x and σ_y. The stress orthogonal to the plane of strain is named σ_z. The corresponding strains are denoted by ε_x, ε_y and ε_z. Cyclic stress-controlled tests were performed under plane strain condition (i.e. ε_z=0). After positioning in the Biaxial Tester, the specimen was biaxially loaded with σ_x=σ_y=50 kPa. The stress orthogonal to the plane of strain, σ_z was initially applied at 20, 50 or 80 kPa via the water filled plane strain boundary. At this stage the tap of the top plane strain boundary was closed in order to record the variation of σ_z during the test. The negative pressure on the sample was then released and the test commenced by increasing σ_x and decreasing σ_y whilst the mean biaxial stress, $(\sigma_x+\sigma_y)/2$, was held constant at 50 kPa through the test.

Fluctuating stresses, varying between principal stress ratios $R_{1max}=\sigma_x/\sigma_y$ and $R_{2max}=\sigma_y/\sigma_x$ (where $R_{max}=R_{1max}=R_{2max}$) were imposed until a repeating state of strain was achieved. At this time the peak stress ratio was increased and the cyclic stressing repeated. After imposing many cycles of stressing and when a state of no further significant change of σ_z was observed the samples were monotonically sheared to failure. Typical stress path for this kind of test are shown in Figure 2.

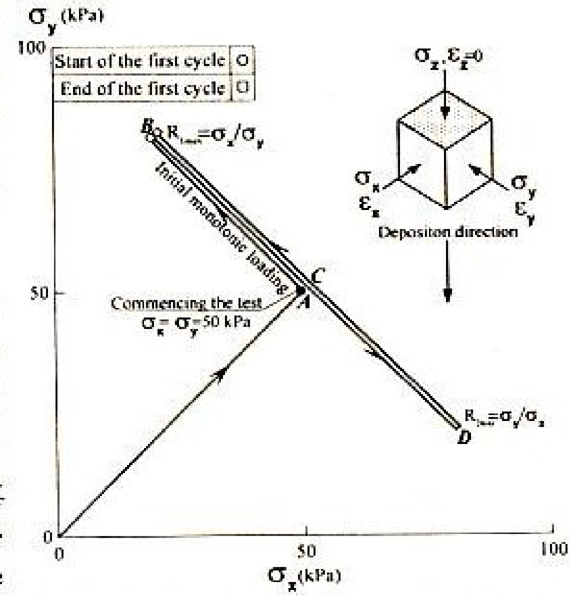

Figure-2: Stress path for cyclic stress-controlled tests.

It should be mentioned that the entire loading and unloading loop considered here, starts from *B* and includes the stress paths *BC*, *CD*, *DC* and *CB*. The stress path *AB* is the initial monotonic loading. The general form of these stress controlled tests in which cycles of fluctuating stress amplitudes were imposed at various deviatoric stresses is illustrated in Figure 3. At *D* the principal stress ratio, $R_{1max}=\sigma_x/\sigma_y$, was the same as that at *B*, $R_{2max}=\sigma_y/\sigma_x$, where $R_{1max}=R_{2max}$.

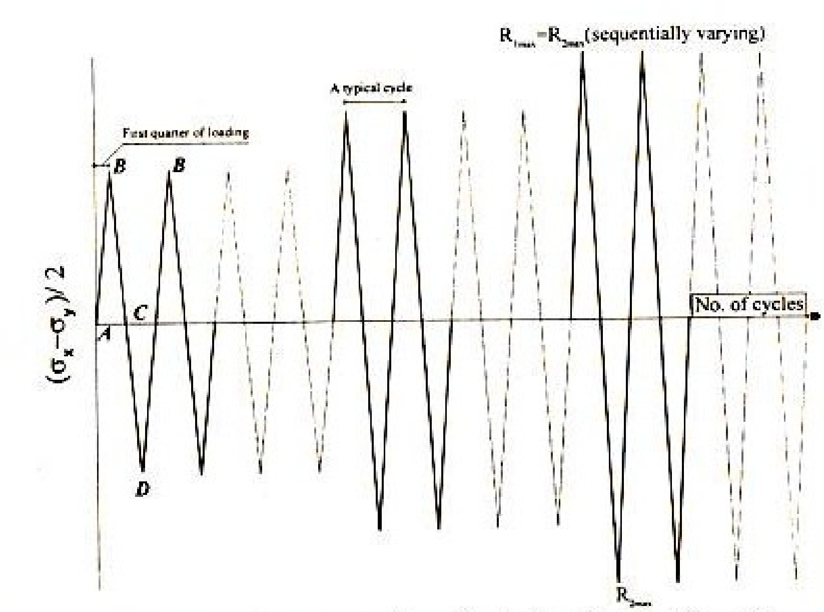

Figure-3: Schematic representation of tests described by Figure 2

STRESS-STRAIN RESPONSE

In the course of cyclic stress loading during the first few cycles sand exhibits significant plastic deformation accompanied by open hysteresis loops. Subsequent cycles give less plastic strain and the loops appear nearly closed. Eventually after many cycles the sand attains a form of equilibrium so that the induced shear strain during each cycle remains approximately constant. When the sample is subsequently subjected to a higher peak stress ratio, plastic strain again develops and larger hysteresis loops are generated which again reduce in amplitude with number of cycles.

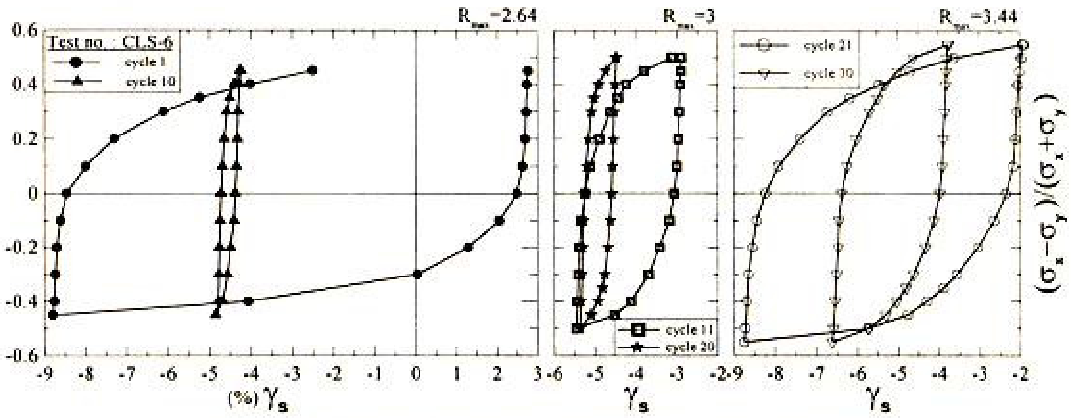

Figure-4: Hysteresis loops for different cycles for loose sand.

The variation of shear strain, γ_s, with stress ratio, $t/s=(\sigma_x-\sigma_y)/(\sigma_x-\sigma_y)$, for test CLS-6 (loose sand) is illustrated in Figure 4. During the first cycle of unloading and reloading significant plastic strain develops. The stress-strain loop is at its largest in the first cycle and then becomes much smaller after a few cycles, i.e. the shear strain amplitude decreases gradually as the number of cycles proceeds, indicating that stiffening due to the creating structure was taking place. The Figure clearly shows that at any stage of increasing peak stress ratio plastic strain develops and again reduces with number of cycles. It is thought that after a large number of stressing cycles the sand behaves elastically.

VOLUMETRIC STRAIN RESPONSE

Figure 5 illustrates the volumetric strain, γ_s, against volumetric strain, ε_v, during different cycles whose stress-strain response is shown in Figure 4. Forms of non-closed butterfly loops developed from the first or second cycle of the dense sand whilst no butterfly shape was observed for loose sand samples in the first few cycles. This is due to the extreme tendency of loose specimens to densify, which is apparent in the Figure. The volume change in loose media during the first few cycles is irrecoverable densification. The results show in subsequent cycles there is no significant overall volume change over a cycle but significant rate of volume change within a cycle i.e. the densification and dilation within a cycle balance each other. The butterfly shape of volume change, Figure 5, within a typical cycle has also been illustrated by Wood and Budhu [5].

The overall magnitude of volume change during a cycle decreases as the number of cycles is increased, nevertheless the butterfly shape of volume change remains. It appears that a steady state of no significant further volume change occurs after many cycles for both dense and loose sands even though the strain amplitude decreases.

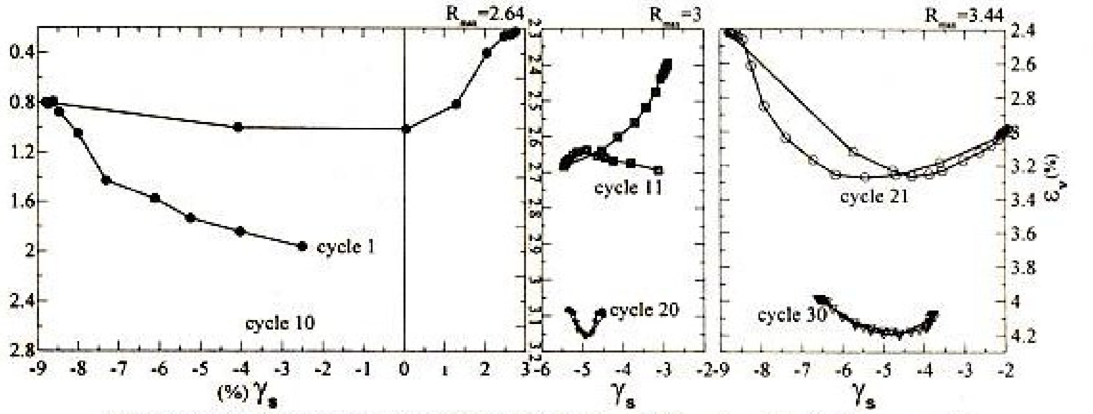

Figure-5: Variation of volumetric strain during different cycles for loose sand.

EFFECT OF INCREASING PEAK STRESS RATO ON VOLUME CHANGE RESPONSE

When conducting cyclic stressing tests with 90° changes in the principal stress direction and with a maximum principal stress ratio constant, a hardening phenomena develops and causes the magnitude of the resultant shear strain amplitude to decrease gradually as the number of cycles proceeds. Eventually after many cycles the sand attains a form of equilibrium so that the induced shear strain during each cycle remains approximately constant. When the sample is subsequently subjected to a higher peak stress ratio, plastic strain again develops and larger hysteresis loops are generated which again reduces in amplitude with number of cycles.

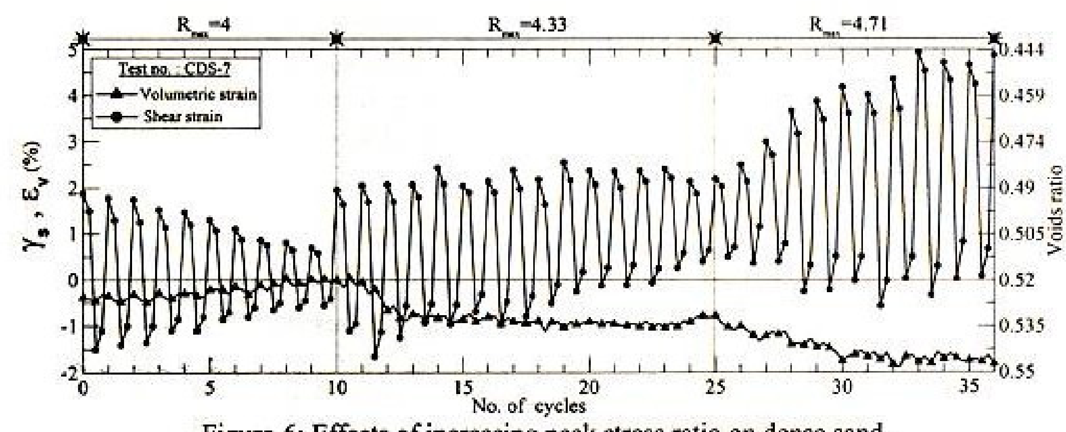

Figure-6: Effects of increasing peak stress ratio on dense sand.

Figure 6 illustrates the results of a test performed on a dense sand which included increasing the peak stress ratio at different stages. The dense sample, prepared by pluviation method, was thought to be close to the densest state for Leighton Buzzard sand and it was not expected to densify much during the test. A slight expansion in volume was observed when the peak stress ratio was increased and followed by no significant overall volume change, accompanying the hardening behaviour.

In a loose media after an increase of stress ratio, Figure 7, considerable shear strain resulted, indicating the previously induced structure was broken down. A new structure soon developed to resist the new stress level. The voids ratio reduced with number of cycles and the material eventually reached a voids ratio of 0.63. The number of cycles required for the sample to attain a form of equilibrium was dependent on the magnitude of the new applied stress ratio. Figure 7 shows that when a higher stress ratio was imposed on the sample, a greater number of cycles was required to reach a repeating shear strain state.

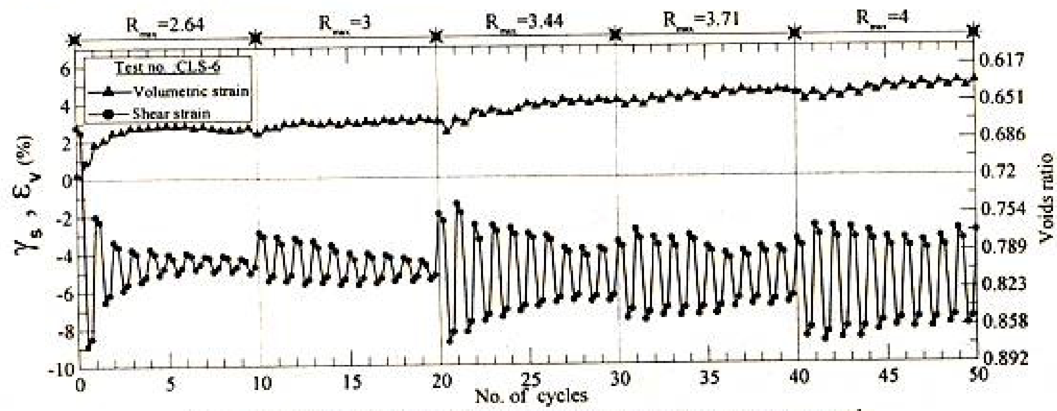

Figure-7: Effects of increasing peak stress ratio on loose sand.

σ_z BEHAVIOUR IN CYCLIC TESTS

The variation of the principal stress orthogonal to the plane of strain, σ_z, was recorded during the tests performed on both dense and loose sand samples. The measurement of σ_z was made for all cycles at the isotropic stress state and at the maximum principal stress ratio. This helped to highlight the general trend of σ_z variation during the test. More detailed records were taken for selected cycles.

Figure 8 illustrates the variation of σ_z from the start of shearing the sample for Tests CDS-9 and CDS-1, performed under different initial magnitudes of σ_z=80 and 20 kPa respectively. The value of σ_z either drops sharply from σ_z=80 kPa, to about 55 kPa or rises from the initial value, σ_z=20 kPa, to about 40 kPa after the initial monotonic shear loading before commencing the first cycle. After many stress cycles σ_z eventually settles at 49 and 44 kPa in Tests CDS-9 and CDS-1 respectively (Figure 8).

In order to study the variation of σ_z with number of cycles and its final value after a number of stressing cycles, cyclic tests were performed on dense and loose sands in which σ_z was initially applied at σ_z=20, 50 & 80 kPa [4]. The results are given in Figure 9 for dense and loose sands.

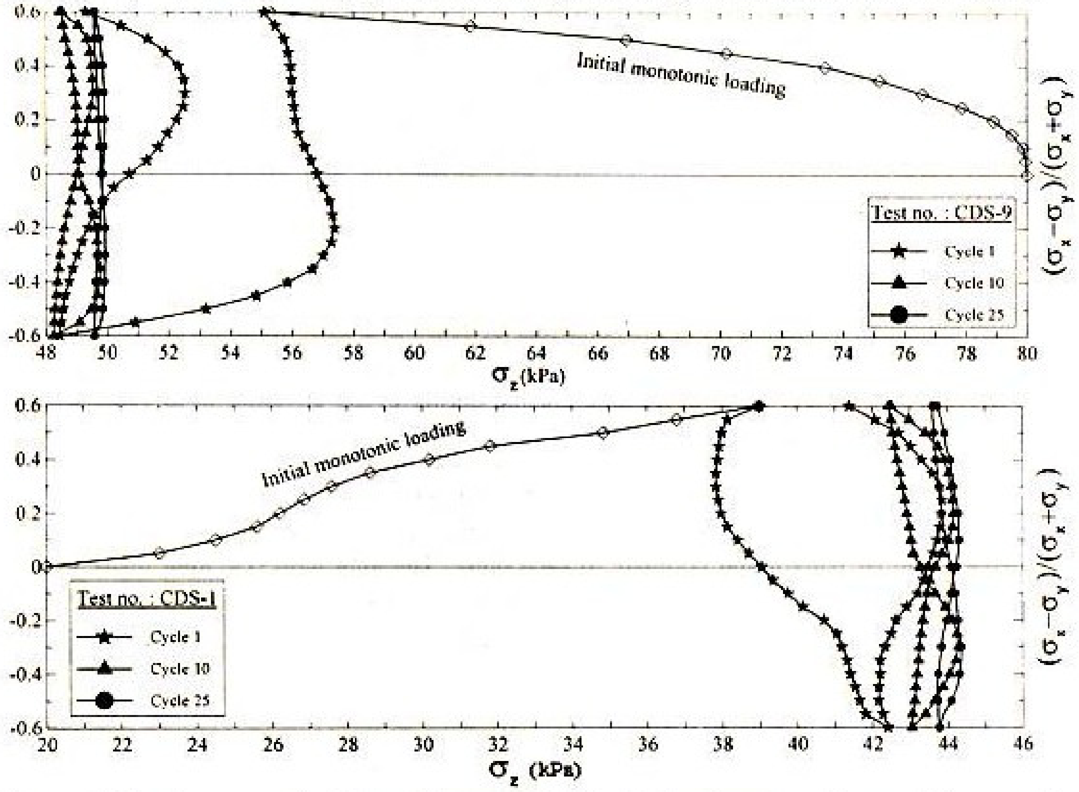

Figure-8: Development of σ_z during the test and its final value for test on dense and loose sands.

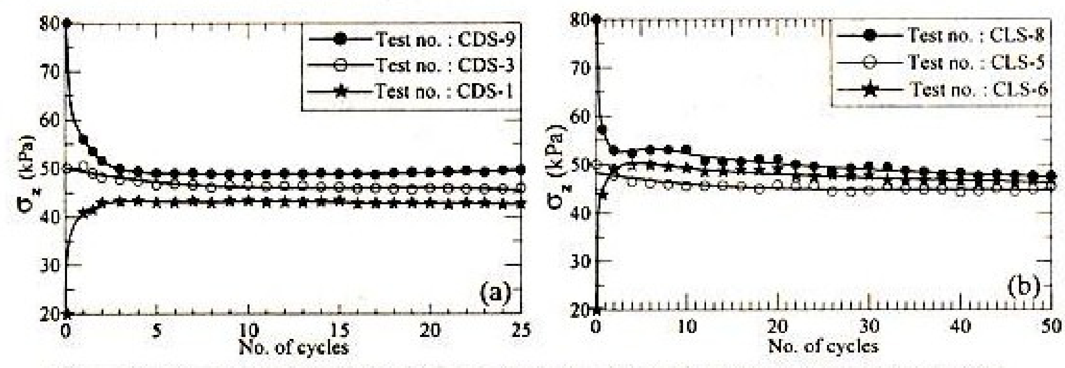

Figure-9: Development of σ_z during cyclic tests with different initial magnitudes of σ_z.

The results on dense sand in which R_{max}=4 was held constant, Figure 9(a), show that after a few cycles, say 5, the values of σ_z span between 40 and 50 kPa. After 5 cycles, however, a very slow trend of building up of σ_z was observed. An average of σ_z=46.1 kPa was achieved when the tests terminated.

Figure 9(b) summarizes the results of tests performed on loose sand in which R_{max} was changed after a number of cycles. The Figure shows a similar trend to that of dense sand. On termination of tests an average of σ_z=46.7 kPa was achieved. It seems that after a large number of cycles the variation of σ_z settles very close to the mean principal biaxial stress $\sigma=\frac{1}{2}(\sigma_x+\sigma_y)=50$ kPa, irrespective of its initial value and initial voids ratio.

CONCLUSIONS

This paper presented the results of cyclic behaviour of dense and loose sands when subjected to a limited number of stress cycles under different initial magnitudes of σ_z. In summary it can be concluded that the general trend of volume change during cyclic stressing is to cause overall densification and a reduction in the associated cyclic shear strain. However, a change of peak stress ratio to a higher value results in a softening behaviour and slight expansion in volume. After many cycles the sand particles re-arrange themselves to sustain the new stress state and a new anisotropic structure develops. The variations of σ_z and rate of volume change during a single cycle have a similar pattern and correlate to each other. The long term (i.e. after many cycles) values of σ_z (which was initially applied at 20, 50 and 80 kPa) are independent of their initial values and settle very close to the mean principal biaxial stress, i.e. $\sigma_z=(\sigma_x+\sigma_y)/2=50$ kPa.

REFRENCE

[1] Wood D. M., (1982) "Laboratory investigations of the behaviour of soil under cyclic loading : A review", *Soil Mechanics-Transient and Cyclic Loads*, Pande G. N. & Zienkiewics O. C. (Eds.), pp. 513-582.

[2] England G. L, Dunstan T., Wan R. and Tsang C. M., (1997) "Drained granular material under cyclic loading with temperature-induced soil/structure interaction", *Applied Mechanics Rev.*, ASME, Vol. 50, No. 10, pp. 553-579.

[3] Ogunbekun O. O. O., (1988) "The influence of boundary constraints on the shearing characteristics of particulate solids at extremely low stress level", Ph.D. thesis, University of London.

[4] Bolouri-Bazaz J., (1999) "Sand subject to biaxial cyclic loading", Ph.D. thesis, University of London.

[5] Wood D. M. and Budhu M., (1980) "The behaviour of Leighton Buzzard sand in cyclic simple shear tests", *International Symposium on Soils under Cyclic and Transient loading*, Swansea.