Strain accumulation in soils due to repeated sinusoidal loading

Ashish Juneja i) and Mohammed Aslam A. K. ii)

i) Associate Professor, Department of Civil Engineering, Indian Institute of Technology Bombay, Mumbai- 400 076, India.
ii) Ph.D Student, Department of Civil Engineering, Indian Institute of Technology Bombay, Mumbai- 400 076, India.

ABSTRACT

Low amplitude repeated loading can cause soil particles to rearrange into a denser and more stable state. When the number of load cycles exceeds the threshold, it results in overstressed soils. This results in accumulation of plastic strains and reduction in shear strength. In some cases, failure planes begin to develop within the soils. Magnitude, duration and the type of loading, all affect strain accumulation and, this needs to be well understood. This paper investigates the effect of multiple loading cycles followed by drainage. This step was repeated a few times before testing the sample up to critical state. In this setup, 100 mm diameter and 200 mm long cylindrical soil samples were arranged on the cyclic triaxial frame. Thousands to a million of sinusoidal loading and unloading cycles were then imposed on the samples. In each case, liquefaction failure was prevented by maintaining low amplitude cyclic stress ratios. In some cases, the samples were allowed to drain in between the cycles e.g. to simulate rail and road embankments. This enabled dissipation of pore pressures from the sample to reach a new state before repeating the cyclic loading. In each case, the rate-of-accumulation of pore pressures and axial-strains reduced with the increase in the number of cycles. The samples were then subjected to undrained shearing at 100 kPa effective stress to investigate the change in strength with the number of load and drainage cycles. The results show that there was a remarkable increase in the stress-strain response of the samples when subjected to recompression. This change was dependent on the number of cycles and cyclic stress amplitude.

Keywords: cyclic triaxial test, axial strain, elastic modulus

1 INTRODUCTION

Pore pressures and axial strains build up when soils are subjected to cyclic loading. Dissipation of these pore pressures recompress the sample leading to strain accumulation. The change in volume is significant if the cycles are increased. These soils tend to reach a new stress state with a bigger hardening parameter. The change in shear strength of such recompressed samples can be significance in the long term stability. Several studies have been conducted on soils subjected to a few high intensity cycles. Unfortunately, these studies have focused on post-earthquake or post liquefaction soil behaviour. For example, Yasuhara (1994), Moses et al. (2003) and Jigheh and Soroush (2006) investigated the post cyclic shear behaviour by capturing the pore water pressures. Whilst Matsui et al. (1980), Yasuhara and Andersen (1991), Yildirim and Ersan (2007) and Wang et al. (2013) investigated the post cyclic behaviour of consolidated samples. There is dearth of literature on soil response to low intensity, large number of cyclic load (Wichtmann et al. 2005.). This study is important for soils below railway or roadway foundation, offshore and storage structure or machine foundation. In each of these cases, the loading is followed by a drainage period, which causes the soils to recompress. In most of the cases, the cyclic stress and, the number of cycles, are the two variables. A convenient way to account for all of these effects is to record the updated shear strength of the soils subjected to the above repeated loading. This paper focuses on the laboratory study of post-cyclic response of two different soils. The soils were subjected to a large number of cyclic loads. Volumetric compression and stress-strain curves of the soils were obtained after recompression and completion of the cyclic loading.

2 METHODOLOGY

Fine alluvial Gujarat sand (GS) and Delhi silt (DS) were used in this study. Fig. 1 shows the particle size distribution of the two soils. Specific gravity of both the soils was equal to 2.65. GS was subrounded to subangular and had minimum and maximum void ratios of 0.542 and 0.778, respectively. DS had LL equal to 22% and PL equal to 19%. Samples of 100 mm diameter and 200 mm height were prepared using oven dried soils. The samples were directly prepared onto the triaxial base. GS samples were prepared by air pluviation at low relative
densities. DS samples were prepared by tamping the dry soil in 10 layers using a 50 mm diameter tamper falling from a height of about 145 mm. Each layer was given 10 blows using the 175g tamper. In both the cases, the samples were prepared by stretching a rubber membrane over a split mould fixed to the triaxial base. The mass of soil trapped within the mould was carefully noted. A top cap was then placed over the sample before sealing it. At the same time, a suction of about 12 kPa was applied to hold the sample before removing the split mould. The height and diameter of the sample was then carefully measured.

After preparing the sample, the triaxial cell was filled with water and about 20 kPa cell pressure applied, before releasing the suction. It was then flushed with carbon dioxide before permitting the sample to saturate using distilled water and back pressure. After the saturation was complete, the sample was then consolidated to 100 kPa mean effective stress.

The cyclic triaxial tests were performed using a servo-pneumatic stress controlled triaxial frame set at 1 Hz frequency. Two way sinusoidal waves were used. The data was logged at 1ms for each cycle. Drainage was permitted once the cyclic loading was complete. The sample was then subjected to monotonic shear at a rate of about 0.25 m m per minute. In essence, the testing program consisted of the following steps:

(1) Conduct cyclic triaxial tests on consolidated samples. The tests were performed at different cyclic stress amplitude and cycles ranging from hundreds to a million cycles.

(2) Permit the excess pore pressures to dissipate.

(3) Repeat Steps 1 and 2 for multiple stages.

(4) Perform monotonic undrained test.

In each case, the stress amplitude was low to prevent pore pressure ratio, $r_u$ to exceed unity and thereby prevent liquefaction. Here $r_u$ is equal to the ratio of excess pore pressure to the initial effective consolidation pressure

$$r_u = \frac{\Delta u}{p_c}$$

Excess pore pressures were drained and the volumetric compression was noted after each test before shearing them to failure. The new strength of the soil was compared to the strength when the sample was not reconsolidated. Table 1 shows the details of the tests. As can be seen, 15 tests were performed on GS and 8 tests on DS.

Table 1. Test details.

| Test | Dry density after consolidation (Mg/m$^3$) | Cyclic stress amplitude (kPa) | Number of cycles in 1 stage | No of load-drainage stage | Total number of cycles |
|------|------------------------------------------|-------------------------------|----------------------------|--------------------------|------------------------|
| GS00 | 1.63                                     | -                             | -                          | -                        | -                      |
| GS01 | 1.54                                     | 5                             | 1000                       | 1                        | 1000                   |
| GS02 | 1.55                                     | 5                             | 1000                       | 2                        | 2000                   |
| GS03 | 1.53                                     | 5                             | 10000                      | 1                        | 10000                  |
| GS04 | 1.62                                     | 5                             | 50468                      | 1                        | 50468                  |
| GS05 | 1.55                                     | 10                            | 100                        | 10                       | 1000                   |
| GS06 | 1.51                                     | 10                            | 1000                       | 1                        | 1000                   |
| GS07 | 1.53                                     | 10                            | 1000                       | 2                        | 2000                   |
| GS08 | 1.57                                     | 10                            | 10000                      | 3                        | 3000                   |
| GS09 | 1.54                                     | 10                            | 1000                       | 5                        | 5000                   |
| GS10 | 1.54                                     | 10                            | 1000                       | 6                        | 6000                   |
| GS11 | 1.53                                     | 10                            | 1000                       | 10                       | 10000                  |
| GS12 | 1.54                                     | 10                            | 10000                      | 1                        | 10000                  |
| GS13 | 1.56                                     | 10                            | 10000                      | 2                        | 20000                  |
| GS14 | 1.56                                     | 10                            | 20000                      | 2                        | 40000                  |
| GS15 | 1.55                                     | 10                            | 50000                      | 1                        | 50000                  |
| DS00 | 1.59                                     | -                             | -                          | -                        | -                      |
| DS01 | 1.59                                     | 5                             | 10000                      | 1                        | 10000                  |
| DS02 | 1.58                                     | 5                             | 100000                     | 1                        | 100000                 |
| DS03 | 1.62                                     | 5                             | 1000000                    | 1                        | 1000000                |
| DS04 | 1.59                                     | 10                            | 50000                      | 1                        | 50000                  |
| DS05 | 1.60                                     | 10                            | 100000                     | 1                        | 100000                 |
| DS06 | 1.61                                     | 10                            | 100000                     | 1                        | 100000                 |
| DS07 | 1.58                                     | 15                            | 50000                      | 1                        | 50000                  |
| DS08 | 1.60                                     | 15                            | 100000                     | 1                        | 100000                 |

3 EXPERIMENTAL RESULTS

3.1 Cyclic shear response

Both elastic and plastic strains were developed during the cyclic loading. Fig. 2 shows the stress strain plot in test DS03. As can be seen, $r_u$ accumulated during the cyclic loading. This accumulation was noted to be proportional to the cyclic stress amplitude. Fig. 3 and 4 show change of $r_u$ with number of cycles. The figures show that when the cyclic stress amplitude was less than 5 kPa, the change in $r_u$ was insignificant in both the soils and rose marginally even when the cycles reached 50,000. The rate of increase in pore pressure dropped after peaking at about 5,000 and 10,000 cycles in GS and DS, respectively. In some tests, there was a
small reduction in the pore pressure when the cycles exceeded 10,000 although the cyclic stress amplitude was less than 15 kPa. It is likely that, these samples exhibited a tendency to dilate before reaching its ultimate state. These soils have a greater tendency to exhibit catastrophic failure when the cycles exceed a threshold (e.g. Take and Bolton 2004). Further investigation is needed to elude this phenomenon.

Figs. 6 and 7 also show that axial strain, \( \varepsilon \), accumulation of GS and DS were low when the cyclic stress amplitude was less than 15 kPa. It would not be incorrect to assume that most of these strains would be plastic because none of the soils had any clay. Furthermore, at cyclic stress amplitude of 15 kPa, DS exhibited significant accumulation of \( \varepsilon \). Fig. 8 shows that the elastic modulus, \( E \) degraded in both the soils. The figure shows degradation in some of the tests for the first 100,000 cycles. The degradation increased with the increase in cyclic stress amplitude and cycles. Degradation of \( E \) increases the \( \varepsilon \), as expected.

Fig. 9 shows that volumetric compression was in proportion to the dissipation of the excess pore pressure. However, the change in volume was controlled by the number of steps in samples with intermittent drainage.

Fig. 10 shows that the tests which had similar number of cycles but tested in different modes, that is, whether or not intermittent drainage was permitted, they all showed identical behaviour e.g. tests GS05 is identical to test GS06. Similarly, test GS11 is identical to test GS12, although 10 drainage periods were permitted in GS05 and GS10. Fig. 11 shows the peak shear strength with cycles of all the tests.

3.2 Post cyclic shear response

After the cyclic loading and drainage, all the samples showed an improvement in their strength in proportional to the cyclic stress amplitude and cycles. Fig. 10 shows that the tests which had similar number of cycles but tested in different modes, that is, whether or not intermittent drainage was permitted, they all showed identical behaviour e.g. tests GS05 is identical to test GS06. Similarly, test GS11 is identical to test GS12, although 10 drainage periods were permitted in GS05 and GS10. Fig. 11 shows the peak shear strength with cycles of all the tests.
4 SUMMARY

Post-cyclic response of two soils was examined using cyclic triaxial testing. Rate-of-accumulation of pore pressure and axial strain reduced with the increase in the number of cycles. Pore pressures were permitted to dissipate upon drainage to allow the samples to reach a new equilibrium and stiffer state. There was significant change in the stress-strain curve of the samples which were subjected to the recompression. This change was dependent on the number of cycles and cyclic stress amplitude.

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