# DYNAMIC STRENGTH OF GRAVELY SAND WITH EMPHASIS ON THE EFFECT OF MEMBRANE COMPLIANCE

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**ABSTRACT:**

Dynamic behavior of coarse granular soils is becoming increasingly important in design and analysis of geotechnical engineering problems. Naturally coarse-grained soils are widely present in many parts of the world such as Tehran, the capital city of Iran which is located in a seismic area. In order to understand the dynamic behavior of this soil, a series of undrained cyclic triaxial tests were performed on reconstituted samples. In this paper, cyclic strength of Tehran soil is evaluated considering the effect of membrane compliance on excess pore water pressure. The test results indicate that in general, by increase in cyclic deviatoric stress amplitude or cyclic stress ratio, the required number of cycles to failure decreases. Also, the maximum excess pore water pressure ratio ($r_u$) approaches to one at failure which can be considered as the initial liquefaction in Tehran alluvium. The medium to high relative density of Tehran soil results in cyclic mobility and limited shear strains as the number of stress cycles goes beyond initial liquefaction no flow liquefaction is observed during the conducted tests.

**KEYWORDS:** Dynamic Strength, Gravely Sand, Cyclic Triaxial Test, Membrane Compliance, Cyclic Mobility

## 1. INTRODUCTION

Most of the alluvial deposit of Tehran, the capital city of Iran, especially in northern and central parts has a soil with continuous grading nature scattered as sandy gravel to gravely sand. Evidences of partially cementation are observed in most of the deposit mainly in the north section of the city (Haeri et al., 2002). On the other hand, Tehran is situated in earthquake prone area and ground water table is high in central and southern parts of the city. Thus, it is important to characterize dynamic strength and liquefaction susceptibility of the deposit during ground shakings.

Based on previous studies performed by the first author and his co-workers, a base gradation has been introduced as the representative gradation of Tehran alluvial deposit for preparation of samples in laboratory as given in Table 2.1. Previous studies have mainly been concentrated on the static behaviour of cemented and uncemented Tehran gravely sand to evaluate the different aspects of cementation effects (Haeri et al., 2002; Haeri et al., 2005; Hosseini et al., 2005; Haeri et al., 2006). Based on the studies of Haeri et al. (2005) and Hosseini et al. (2005), fabric and structure variations of prepared samples strongly affect the behavior of gravely sand whereas the overall distribution of particle sizes and density were strictly controlled. Nevertheless the normalized data showed a unique boundary surface in normalized spaces. Dynamic behavior including shear modulus and damping ratio of cemented and uncemented gravely sand samples examined by Haeri et al. (2007 and 2008) show that shear modulus increases with increase in cement content. However, damping ratio does not show any clear trend with change in cement content. The studies also show that shear modulus increases with increase in confining pressure (in the range of 100 to 500 kPa). On the other hand, performed tests indicate a downward trend for damping ratio versus confining pressures. Degradation index has also shown a descending relationship with the number of load cycles.

During drained and undrained triaxial tests on saturated granular soils, penetration of the flexible rubber membrane into peripheral voids changes and influences on the volume change and pore pressure measurements. This effect is called membrane compliance and depends on various factors such as effective confining pressure, mean grain size and relative density of the soil, and membrane thickness. The membrane compliance may have strong effects on the volume change and pore pressure measurements at consolidation and shear stages of static and dynamic triaxial tests and may have drastic influences on the static and dynamic strength of granular soils especially medium to coarse sands and gravels. During last two decades, various researchers such as Evans and Seed (1987), Tokimatsu (1990) and Nicholson et al. (1993) have examined membrane compliance and tried to preclude, limit or quantify this effect on triaxial test results. Newland and Allely (1957) assumed that the skeletal volume change of a cylindrical sample subjected to isotropic confining pressure is equal to three times of the axial strain. The volume change due to membrane penetration can be calculated by Eqn. 1.1.

$$\Delta V_m = \Delta V_T - \Delta V_s = \Delta B_m \cdot A_m \tag{1.1}$$

where $\Delta V_T$ and $\Delta V_s$ are total and skeletal volume changes, respectively, and $\Delta B_m$ and $A_m$ are unit membrane compliance and membrane surface area, respectively. Nicholson et al. (1993) showed that $\Delta B_m$ correlates with $\log \sigma'_3$ ($\sigma'_3$ is effective confining pressure) by a linear relation and called this ratio normalized compliance (S). In undrained loadings, tendency of membrane penetration, changes the pore pressure and results in an unconservative strength (Tokimatsu and Nakamura, 1986). In this condition summation of skeletal volume change and membrane penetration is equal to pore water volume change (Baldi and Nova, 1984). Ansal and Erken (1996) proposed a correction to calculate non-compliant pore pressure as:

$$\Delta u_{non\text{-}compliant} = (1 + C_{RM}) \cdot \Delta u_{compliant} \tag{1.2}$$

where $C_{RM}$ is membrane compliance ratio and $\Delta u_{compliant}$ .is the measured excess pore pressure during cyclic test.

When an isotropic consolidated sample is subjected to symmetrical cyclic shearing, the stress reversal will occur. In this case the direction of the shear stress changes so that each cycle includes both compressional and extensional loading. Experimental evidences (e.g., Dobry et al., 1982; Mohamad and Dobry, 1986) have shown that the rate of pore pressure generation increases with increase in degree of stress reversal. Hence the effective stress path moves relatively quickly to the left (because excess pore pressure builds up quickly) and eventually oscillates along the compression and extension portions of the drained failure envelope. Each time the effective stress path passes through the origin (twice during each cycle), the specimen is in an instantaneous state of zero effective stress ($r_u$ = 100%). Although this state of zero effective stress is referred to as initial liquefaction (Seed and Lee, 1966), it should not be taken to imply that the soil has no shear strength. Significant permanent strains may accumulate during cyclic loading, but flow failure can not occur.

The liquefaction resistance of an element of soil depends on how close the initial state of the soil is to the state corresponding to "failure" and on the nature of the loading required to move it from the initial state to the failure state. The failure state is different for flow liquefaction and cyclic mobility. The failure state for flow liquefaction is easily defined using the Flow Liquefaction Surface (FLS), and its initiation is easily recognized in the field. The definition of failure for cyclic mobility is imprecise-a certain level of deformation caused by cyclic mobility may be excessive at some sites and acceptable at others. In contrast to flow liquefaction, there is no distinct point at which the failure associated with cyclic mobility can be defined. Cyclic mobility failure is generally considered to occur when pore pressures become large enough to produce ground oscillation, lateral spreading, or other evidence of damage at the ground surface. This definition of failure is imprecise; in practice the presence of sand boils is frequently taken as evidence of cyclic mobility. The development of sand boils, however, depends not only on the characteristics of the liquefiable sand but also on the characteristics (e.g., thickness, permeability, and intactness) of any overlying soils.

In this paper, the results of a series undrained cyclic triaxial tests performed on a gravely sand are presented to examine the effect of membrane compliance, confining pressure and cyclic stress ratio on shear strength and pore pressure build-up. The abovementioned procedure based on the measured data has been used to modify membrane compliance.

## 2. TEST METHOD

Since undisturbed cylindrical sampling from a gravely soil is extremely difficult and especially for this deposit that is heterogeneous, the coarse-grained samples, representative of Tehran alluvium are reconstituted in laboratory considering the sample and grain size limitations. In order to investigate the dynamic behavior of this gravely sand, 34 cyclic triaxial tests are carried out on samples with a relative density of 65%. The samples are isotropically consolidated and sheared in undrained conditions. Tests were carried out under three different confining pressures ($\sigma_3$), between 100 to 500 kPa and various double amplitude cyclic deviatoric stresses ($\sigma_{d,cyc}$).

### *2.1. Tested Material*

Tehran alluvium alters highly in gradation and cementation. The range of cementation is from completely uncemented to strongly cemented and the range of gradation is from gravel and boulder in the north to clay and silt in the south. This research is carried out on representative soil of course grain part of Tehran deposit. The average grain size distribution of Tehran alluvium, which is called the base soil and classified as SW-SM has been suggested and used by (Haeri et al., 2002, and 2005). The index characteristics of this soil is given in Table 2.1 and is considered for the reconstituted samples in this research. The maximum grain size is limited to 12.5 mm to comply with 100 mm diameter and 200 mm height of the sample.

Table 2.1. Index Properties for Tested Material.

| Fine Content $F_C$ | Sand Content $S_C$ | Gravel Content $G_C$ | Effective Grain Size $D_{10}$ | Medium Grain Size $D_{50}$ | Uniformity Coefficient $C_U$ | Curvature Coefficient $C_C$ | $G_s$ | $\gamma_{d,min}$ (kN/m$^3$) | $\gamma_{d,max}$ (kN/m$^3$) | $\gamma_d$ (kN/m$^3$) |
|---|---|---|---|---|---|---|---|---|---|---|
| 6 % | 49 % | 45 % | 0.2 mm | 4.0 mm | 28 | 1.8 | 2.57 | 16.14 | 18.78 | 17.75 |

### *2.2. Testing Procedure*

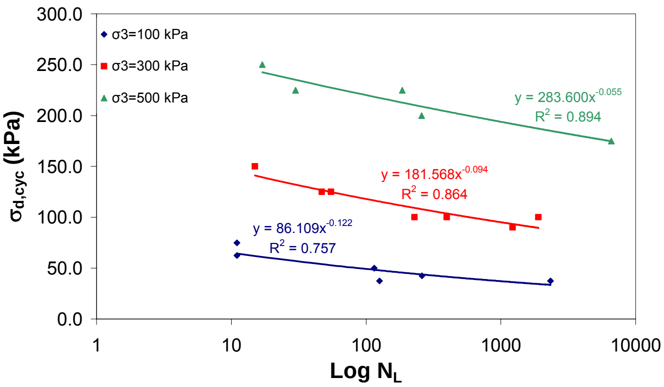
Consolidated undrained cyclic triaxial tests on reconstituted samples of the base soil were carried out. Each sample was compacted in four layers with determined gradation and density using wet tamping method with a water content of 8.5%. Under-compaction of lower layers was also considered during preparation (Ladd, 1978). Based on the preliminary cyclic test results, under-compaction percent ($U_n$) varied linearly between 3.0% for the first layer and 0.0% for the forth layer.

The samples were directly prepared on the pedestal by a two-split mould. When the preparation was completed, the samples were de-aired by flushing $CO_2$ and then saturated by gravitational flushing de-aired water. During this process a maximum cell pressure of 30 kPa was maintained on the sample. Saturation stage is then completed to reach a B-value of greater than 0.95 by applying appropriate back pressure. During this process, an effective stress of 10 kPa was maintained on the sample. After saturation, the samples were isotropically consolidated and then cyclically sheared with a frequency of 1 Hz in stress control and undrained condition. Principal stress reversal is also considered during cyclic loading for all tests. Some tests were re-examined to evaluate the repeatability of the tests.

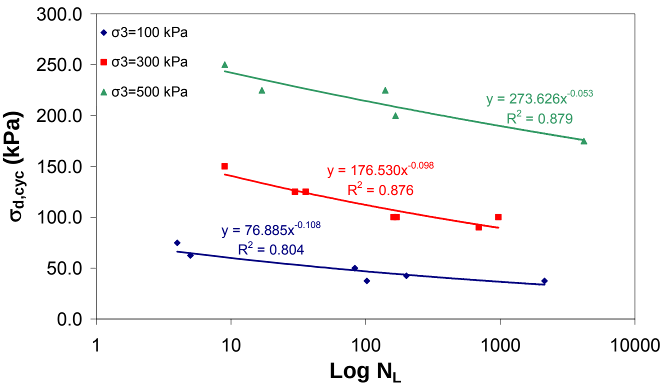
## 3. TEST RESULTS AND DISCUSSION

Considering the granular nature of the gravely sand specimens, the membrane compliance should have been evaluated. The consolidation stage of each test was used to obtain normalized compliance (S) and membrane compliance ratio ($C_{RM}$) by the procedure briefly presented in Section 1. Based on the performed tests, S and $C_{RM}$ are about 0.05115 $cm^3/cm^2$ and 1.16, respectively for the evaluated specimens.

Figure 3.1 (a) shows the cyclic strength of gravely sand specimens versus $N_L$ (the number of cycles required to produce initial liquefaction with $r_u$ = 1.00 or the number of failure cycles) in semi-logarithmic scale. The cyclic strength is the single amplitude of the cyclic deviatoric stress at failure and the failure cycle is the cycle that the excess pore water pressure ratio ($r_u = u_{excess}/\sigma_3$) equals to 1.0 which can be considered as the initial liquefaction. Figure 3.1 (a) is based on the raw data with no membrane correction effects. If the pore water pressure generation is corrected by Eqn. 1.2, the results illustrated in Figure 3.1 (b) can be obtained. The comparison of these two figures indicates that the non-compliant samples are liquefied more rapidly than compliant samples. The results also show that the more effective confining pressure, the more cyclic strength will be obtained at the same number of cycles. The shear strengths for $\sigma_3 = 500\,\text{kPa}$ and $\sigma_3 = 300\,\text{kPa}$ are more than 3.8 and 2.2 times greater than that for $\sigma_3 = 100\,\text{kPa}$.




(a)




(b)

Figure 3.1. Single Amplitude Cyclic Deviatoric Stress versus Number of Failure Cycles at Initial Liquefaction (a) for a Membrane-induced Compliant System and (b) for a Non-compliant System.

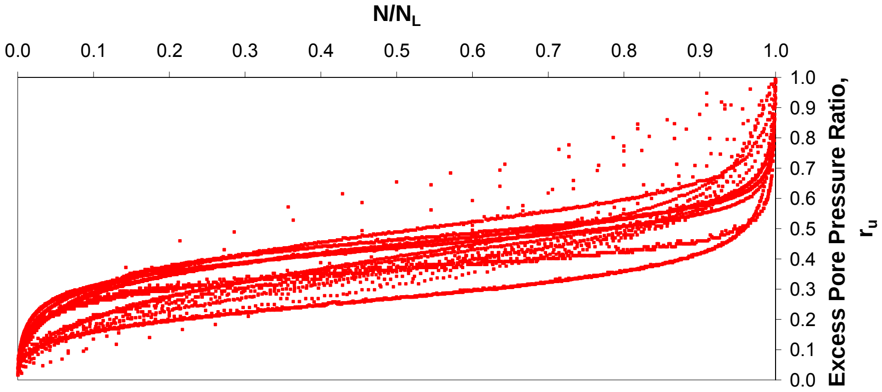
The variations of excess pore water pressure ratio ($r_u$) against normalized number of cycles during cyclic loading stage ($\frac{N}{N_L}$) are depicted in Figure 3.2. The normalized number of cycles is calculated by dividing the current number of load cycle to the number of load cycle at initial liquefaction. As seen both ratios simultaneously vary between 0 and 1 in the same direction. The effect of membrane compliance can be observed comparing Figures 3.2 (a) and (b). The form of pore water pressure generation is clearly different in two figures, especially in the last stages of the loading.

For stress-controlled cyclic tests with uniform loading, Lee and Albaisa (1974) and DeAlba et al. (1975) found that the pore pressure ratio, $r_u$, is related to the number of loading cycles by Eqn. 3.1:

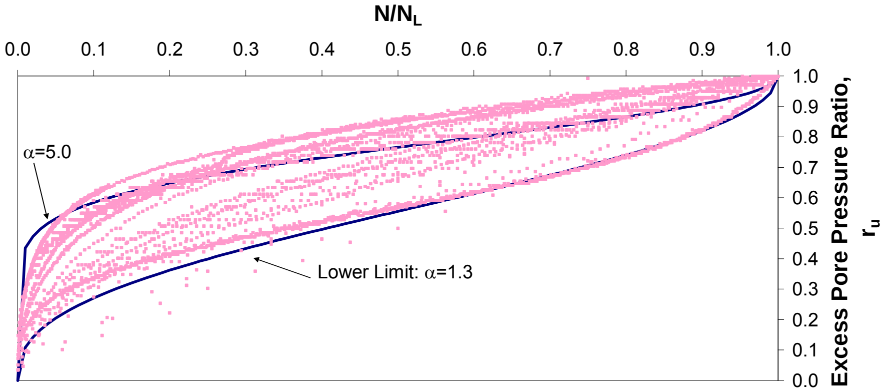
$$r_u = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_L}\right)^{1/\alpha} - 1\right] \tag{3.1}$$

where α is a function of the soil properties and test conditions. Eqn. 3.1 Can be used to estimate the excess pore pressure generated when initial liquefaction does not occur (i.e., when $N_{eq} < N_L$).

For the tested material in this research, α can be obtained by a trial procedure to adequately fit to the results. As seen in Figure 3.2 (b), α = 1.3 is a suitable lower limit and conforms well to the variations of $r_u$. But it sounds that the above relation can not fit appropriately to the upper portion of the results and other relations should be considered.




(a)



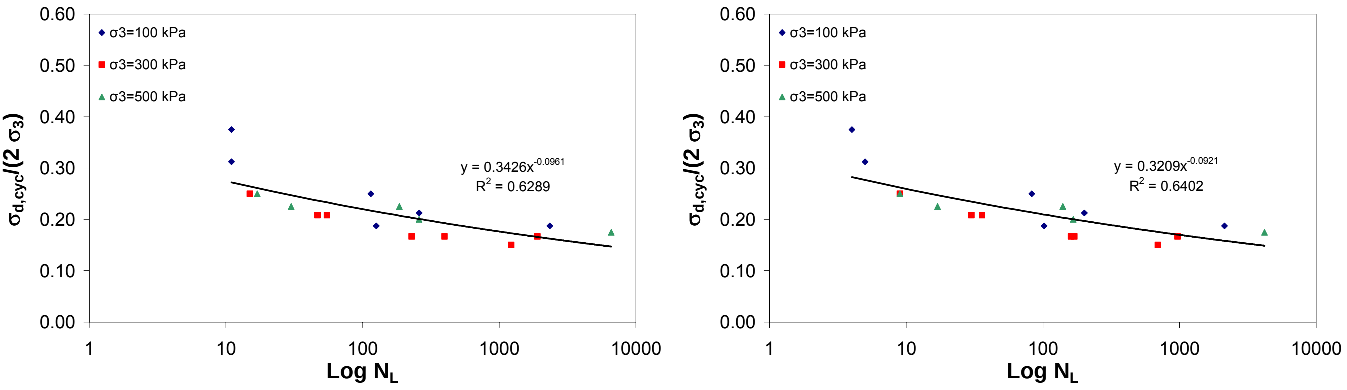

(b)

Figure 3.2. Variation of Excess Pore Pressure Ratio versus Normalized Number of Cycles (a) for a Membrane-induced Compliant System and (b) for a Non-compliant System.

Generally laboratory tests show that the number of loading cycles required to produce liquefaction failure, $N_L$, decreases with increasing shear stress amplitude and with decreasing density. While liquefaction failure can occur in only a few cycles in a loose specimen subjected to large cyclic shear stresses, thousands of cycles of low-amplitude shear stresses may be required to cause liquefaction failure of a dense specimen. The relationship between density, cyclic stress amplitude, and the number of cycles to initial liquefaction can be expressed graphically by laboratory cyclic strength curves. Cyclic strength curves are frequently normalized by the initial effective overburden pressure to produce a cyclic stress ratio (CSR). The CSR should be defined differently for different types of the tests. For the cyclic triaxial test, it is taken as the ratio of the single amplitude cyclic shear stress to the initial effective confining pressure as:

$$CSR=\frac{\sigma_{d,cyc}}{2\sigma_3} \tag{3.2}$$

The cyclic strength curve for this research is presented in Figure 3.3 both without and with considering the membrane compliance effect. The cyclic strength is normalized with respect to the effective confining pressure. Comparison of Figures 3.3 (a) and (b) shows that the cyclic stress ratio of a non-complaint system is about 95% of a compliant system. Therefore the normalized cyclic strength is not highly influenced by membrane compliance.




(a) (b)

Figure 3.3. Cyclic Stress Ratio versus Number of Failure Cycles due to Initial Liquefaction (a) for a Membrane-induced Compliant System and (b) for a Non-compliant System.

## 4. CONCLUSION

Considering the results obtained from the tests, the following items can be concluded:

- The membrane compliance strongly influences the generation of pore water pressure during cyclic tests on gravely sands.
- Increase in effective confining pressure results in increase in number of cycles at failure.
- The correlation proposed by Lee and Albaisa (1974) and DeAlba et al. (1975) can be used for estimation of lower limit of $r_u$ at each load cycle considering $\alpha = 1.3$. However, this correlation is not appropriate for upper limit of $r_u$.
- The cyclic stress ratio defined in Eqn. 3.2 can appropriately be used to normalize the cyclic strength of gravely sand samples.
- The membrane compliance reduces the cyclic stress ratio of gravely sands by about 5%.

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