Influence of particle shape and size on the dynamic soil properties

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ABSTRACT

Shear modulus and damping ratio which are important parameters in any dynamic analysis of soil are related to the frictional behavior at inter particle contacts and rearrangement of grains. The strength loss of granular soil when subjected to cyclic loading is affected by particle size, shape and its distribution. Soil structure also termed as fabric which encompasses grains/particle distribution, particle orientation and arrangement, the voids and the fluid present in between the voids, continually changes during cyclic loading. Such changes are imperative of mechanisms that result in liquefaction of saturated sands and compaction of dry sands. Unlike, experimental and analytical testing methods, numerical methods are equipped to provide insight into these underlying mechanisms. To understand the interactions between particles that influence the dynamic behavior of soil, modeling of cyclic triaxial test was conducted using 3D DEM for different particle shapes and sizes. Before the testing, validation of the generated soil sample is conducted by checking the pressure dependent behavior and dilative response of the sample. Contact normals and contact forces were also used to validate the sample at different stages. From the cyclic testing, normalized shear modulus curves, contact normal and force distributions were calculated. It was observed that the rounded particles had more shear stiffness compared to other shapes. This behavior was not evident when the shear modulus was normalized and the spherical particles had the highest modulus. The effect of particle shape was verified with the change in the coefficient of uniformity (Cu) with shear modulus and it is observed that shear modulus decrease with an increase of Cu. Micromechanical expression for the same was also been discussed.

Keywords: discrete element method, particle size, particle shape, shear modulus, cyclic triaxial test

1. INTRODUCTION

The overall behavior of the soil composite is significantly influenced by the material parameters such as stiffness of the particle, nature of fluid filling the voids, grain geometry etc. For example, in dry sand only repulsive forces exist between the particles whereas, in wet sand, both adhesive and cohesive forces act. Granular soils can either flow or maintain an inclined surface and form a pile in a specific state. When this state is disturbed by any external force, it may lead to disaster situations such as avalanches/landslides, cyclic mobility, flow liquefaction etc. Liquefaction damage during earthquakes can be prevented if the performance of liquefiable soils and its interaction with structure can be predicted beforehand. Even though there was significant development in the field of testing (Centrifuge and 1-g testing) phenomena such as liquefaction are dictated by micro-level mechanisms.

Analytical methods are useful in geomechanics to provide results with limited effort and identify the variables the affect the problem solution. Numerical simulations facilitate the study of phenomena which cannot be predicted by analytical methods, and inaccessible to experiments. To understand the micromechanical interactions that cause the failure in granular materials like soils, one needs to take the aid of numerical simulations.

Shape of a particle represents the external morphology of a particle. Particle shape is identified by sphericity/form (overall shape or uniformity), roundness (curvature of corners or edges of the particle) and surface texture/roughness (Barrett, 1980). Particle shape and size control the packing density, i.e, the void ratio range, particle roughness strongly influences the inter particle friction. The fabric which represents the particle orientations, contact patterns etc of soil (especially clays) is also controlled by the shape of the particles.

To understand the effect of particle shape and size on the dynamic behavior of granular soil, in specific sand, the discrete element modeling of cyclic triaxial test was considered. The numerical soil model was first validated to confirm the exact sand behavior and establishment of the required test conditions. The model is then tested to record the shear modulus for different shapes and sizes.
of particles.

2 SIGNIFICANCE OF PARTICLE SIZE AND SHAPE

It was observed by Lee (1992) that the tensile strength of two different types of limestones decreased with the increase in particle size which can be attributed to the increase in imperfections with sizes and resulting in internal crushing due to larger flaws. The inter particle coefficient of friction was observed to be increasing with the increase in the diameter of spheres (Skinner, 1969). The void ratio range of a two different sizes is significantly affected by the smaller particle fraction (Cubrinovski and Ishihara, 2002).

Several empirical formulae are suggested for obtaining the small strain shear modulus, Gmax which are mostly a function of void ratio (e) and effective mean pressure p. A function F(γ) describes the decrease of secant shear modulus, Gsec with the increase in shear strain. Gsec is equal to one for small strains (<0.0001%).

\[ G_{sec} = G_{max} (e,p) F(\gamma) \]  
\[ G_{max} = A \left( \frac{a-e}{1+e} \right)^2 p^n \]

Proposed by Hardin and Richart (1963), the equation was developed based on the tests in Ottawa sand and on crushed quartz sand. Constants in the equation are different for round aggregates and for angular grains. Iwasaki and Tatsuoka (1977) reported that at a constant void ratio Gmax is strongly affected by the grain size distribution. With an increase in Cu, there is a decrease in Gmax at a constant void ratio but could not observe a significant influence of the grain shape. Also, with an increase in void ratio, Gmax was observed to be decreasing. For a constant relative density, Menq and Stokoe (2003) reported that Gmax to be increasing with Cu for dense sands (with Cu ~ 10 than those with 1.2 ). They also found that the value of n in Eq.2 increases with Cu at constant Dr (relative density). Hardin and Kalinski (2005) from the resonant column tests identified that gravels had significantly larger shear moduli than that for uniform sand which indicates the increase in Gmax with increase in D50. Similar, to Menq and Stokoe (2003), Wichmann and Triantafyllidis (2009) from resonant column testing observed that while considering relative density there was rather small influence on Gmax and for constant void ratio, there is large influence of the Cu.

During cyclic loading, soil is strained to large amplitudes and the pore water pressure accumulation is influenced by the grain size distribution. Castro and Poulos (1977) observed a faster increase in the excess pore water pressure in materials with lower values of D10 (whose D50 is greater than 0.1mm). Vaid et al. (1990) and Kokusho et al. (2004) worked on the liquefaction resistance of sands with different Cu at same Dr and a relationship could not be established. Duku et al. (2008) also observed similar conclusion for drained testing.

A micromechanical explanation of the increasing coefficient of uniformity for a constant void ratio was given by Radjai and Wolff (1998) and Radjai et al. (1998). The authors compares the force transmission chains in monodisperse and polydisperse materials. In a monodisperse material, which represents a uniformly graded sand (for sands for Cu <4 represents uniformly graded and Cu >6 represents well graded distribution), the force chains are equally distributed whereas for poly disperse material that represents non-uniform distribution, strong and weak force chains are formed throughout the inter particle contacts.

The maximum shear modulus ratio of angular sand is always larger than that of sub rounded sand at the same void ratio, confining pressure, grain size, and content of the minus 400 sieve size fraction (Qian et al., 1993). Tong and Wang (2014) observed that the shear modulus increased with increase in the aspect ratio (i.e., by decreasing sphericity). This is attributed to the increase in co-ordination number. The influence of particle shape on dynamic soil properties is still debated. Some researchers believe that the influence is due to the change in void ratio and not only due to particle shape itself. DEM simulations could provide insight into this by considering particles of different aspect ratios.

3 DISCRETE ELEMENT METHOD

In Discrete Element Method the behavior of the system is simulated using particles which are rigid. It is an assemblage of particles that interact with each other at the specified contact point. The DEM methods used in geotechnics can be classified as smooth /hard contact approach. Smooth contact approach is suitable for the study and is selected for application. In the classical discrete element method, the computations are governed by two laws: Newton’s second law of motion and force displacement law. The contacts of each particle are detected and using the relative position of each particle, from the force displacement law, the contact forces acting on each particle are evaluated.

4 NUMERICAL TESTING

In this study, cyclic triaxial test is modeled in 3D using discrete element method in PFC3D. The study consists of four stages namely: i) Creation of the model ii) determination of relevant macro-properties after subjecting to simulated loading, iii) installation of required stress state and boundary conditions, iv)
visualization and management of required properties.

4.1 Sample generation and stress establishment
For the simplicity and lesser time duration, spherical particles and their clusters have been used in this investigation. Such modeling produces the resistance to sliding of the particles and is also computationally efficient. The particles are generated at a specific void ratio using radius explosion method. In this method, the particles are initially generated at a lesser radius than specified and are multiplied by a factor to establish the required size. (Fig. 1a). The walls are used to confine and load the sample are made longer than necessary to allow for large straining to occur during the test. The walls intersecting one another pose no difficulty as the walls interact only with balls. The sample height to width ratio is maintained at 2:1. After the particles are packed as required, the sample is consolidated by invoking servo control algorithm. To achieved and in the second stage triaxial test is simulated. Particle cloud is generated and then the triaxial conditions are imposed (Fig. 1b) using the walls. Later, isotropic compaction is performed until a uniform stress is reached in all directions.

![Fig. 1. Cylindrical specimen generated](image)

4.2 Validation
To verify the discrete soil model generated, an isotropic principal stress of 2MPa was applied along the assembly (Fig. 1) along the three principal stress directions using the servo controlled boundary stress mode. The relationship between the measured stress and the applied boundary stress is shown in Fig. 2. The required stress is applied in small time steps (cycles). The sample reaches the target stress in about 2000 cycles beyond which it remains constant.

![Fig. 2. Number of cycles to achieve the required boundary stress](image)

To verify the confining pressure dependent behavior of the soil sample, drained triaxial compression tests at confining pressures 2, 1 and 0.5 MPa were performed up to 0.3% strain. Fig. 3 shows the variation of deviatoric stress and volumetric compressibility versus axial strain. It is observed that there is an increase in the deviatoric stress and volumetric compressibility with increase in the confining pressure. The trend is similar to that observed experimentally. From Fig.3, the confining pressure dependent behavior and dilation response of the granular material at low confining pressure (0.5MPa) can be observed.

![Fig. 3 (a,b). Confining pressure dependent behavior](image)

In order to check the isotropic distribution of forces and contacts in the sample (strong chains indicate the biased distribution of force), contact normals and contact force after the generation of sample and after the consolidation are plotted (Figs.4-7). From Fig.4 & 5 it is evident that the sample had no significant force chains and also there is an isotropic distribution of the applied stress. As the shear force in the specimen is zero, there is no distribution of shear force shown and also the contact orientation (Fig.4b) infers the same. Similar observation can be made from the contact distribution and forces.
after the specimen is consolidated (Figs.6 &7).

4.3 Cyclic triaxial testing

From the validation of the generated soil sample, it was confirmed that further testing can be performed on the specimen to understand the macroscopic and microscopic dependence of the shear modulus on particle morphology.

To evaluate the particle shape effects, three samples with different particle shapes have been generated for the simulation. The first sample (Fig.8a) consists of spheres of diameter ranging from 0.075m - 0.1m. The second sample (Fig.8b) is generated by overlapping two spheres termed as a clump in PFC. Initially spheres are generated with the required void ratio and are then replaced by the clumps of required shape. Third sample (Fig.8c) which represent the shape of a peanut consists of three spheres that overlap at a specified distance, is also produced in the similar fashion. After generation of the balls/particles of the sample, they are later allowed to expand gradually with a multiplying factor of 1.6 and consolidated until equilibrium is achieved. Based on the angularity, roundness and texture of the particles, dyads are considered to represent rounded particles and dyads to represent angular shapes.

The walls used to confine and load the sample are made longer than necessary to allow for large straining to occur during the test. The walls intersecting one another pose no difficulty as the walls interact only with balls. The assembly contains 358 particles in case of dyad sample and 1074 particles in case of peanut sample. Dyad particles are formed by overlapping spheres of equal radius and clumps by overlapping a bigger spheres with two small spheres of radius about half of the bigger sphere. Porosity of 0.36 has been achieved by radius explosion algorithm.

To check the particle sizes effects, sample with different co-efficients of uniformity ($C_u$) are generated. $C_u$ is calculated by diving $D_{10}$ by $D_{60}$. $D_{10}$ is the percentage of particles that are smaller than 10mm and $D_{60}$ is the percentage of particles that are smaller than 60mm.

- $C_u=1.36, e=0.48$
- $C_u=1.18, e=0.69$

Shear modulus from cyclic triaxial testing, is obtained from elastic modulus ($E$). Elastic modulus is calculated from Eq.6 and shear strain and shear modulus are evaluated from the Young’s modulus, Poisson ratio ($\mu$),
deviatoric stress ($\sigma_d$) and deviatoric strain ($\varepsilon_{\text{max}}$) as shown in Eq.7.

\[
E = \frac{\sigma_{d,\text{(max)}}}{\varepsilon_{\text{max}}}
\]

\[
\gamma = (1 + \mu)\varepsilon_{\text{max}}, \quad G = \frac{E}{2(1+\mu)}
\]

The drained cyclic triaxial tests were carried out on different samples using servo-controlled boundary mode keeping $\sigma_{22}$ constant and applying a strain rate $\varepsilon_{11}$ in the z direction. The results of the simulation are obtained in the form of deviatoric stress, strain, volumetric strain, shear modulus and shear strain.

5 RESULTS AND CONCLUSIONS

5.1 Shape effects

Cyclic drained tests were conducted at strain amplitudes ranging from 0.1-0.0001. The results of the tests are presented in the following figures. Figure 4 shows the variation of deviatoric stress with deviatoric strain for different particle assemblies. This plot clearly indicates that the deviatoric stress reaches a maximum value in the case of dyad compared to that for peanut and sphere shapes. The values of peanut and sphere are comparatively higher for peanut and dyad. Also, it is observed that the magnitude of the stress is more on the compression side than the tension side.

The degradation of the modulus can be clearly seen in this plot (Fig.9). It is interesting to observe that the shape effects are prevalent. Shear modulus is observed to be high for dyad shaped particles that has less interlocking compared to peanut shaped particles. The effect is high at low strains < 0.01 and is less prevalent during high strains (>0.01). When the shear modulus is normalized using $G_{\text{max}}$ which is obtained at $10^{-4}$ strain and plotted against shear strain (Fig.10), it was observed that shape effect was less prevalent during high strains as observed earlier. Even though there it was observed that there is a variation, the difference of $G/G_{\text{max}}$ is more at strain 0.001 compared to other strains between the dyad/peanut and sphere sample.

5.2 Size effects

It is identified that shear modulus is not much affected by void ratio from these simulations. The maximum shear modulus has been observed to increase with decrease in 'e' (Fig.11). In this dynamic testing it has been identified that $G_{\text{max}}$ decreased with an increase in $C_u$ (uniformity coefficient, Fig.12). Shear strain has also been observed to be linearly dependent on $C_u$. Coordination number (Z) when compared for different specimens, it has been observed to decrease with an increase in 'e'.

Failure of a specimen is related to the failure of strong force chains. Experimental evidence suggests that the real sand specimens confirmed this hypothesis that material response is dominated by strong force chains. From observing the contact force distribution in the specimens (Fig.13), it was observed that due to weak forces in the specimen, the stiffness of the poly disperse(low $C_u$) sample is high compared to that of the mono disperse sample as observed by Radjai et al. (1998). The histograms of the normal and shear force; contact normals of the polydisperse sample is attached (Figs. 14 & 15).
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