International Research Journal of Engineering and Technology (IRJET) e-ISSN: 2395 -0056
Volume: 04 Issue: 04 | Apr -2017 www.irjet.net p-ISSN: 2395-0072

EFFECT OF PARTICLE SHAPE ON SAND-GEOSYNTHETIC INTERFACES

Krupa Sara Thomas[1], Vishnu M Prakash[2]

[1]*P.G. Student, Department of Civil Engineering, Saintgits College of Engineering, Kerala, India*
E-mail: krupasarathomas93@gmail.com
[2]*Assistant Professor, Department of Civil Engineering, Saintgits College of Engineering, Kerala, India*
E-mail: vishnu.mp@saintgits.org

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Abstract - *Soil reinforcement (geosynthetics) techniques are adopted to improve the performance of earth structures like reinforced walls, soft ground improvement, roads and railways embankments, slope stabilization and foundations. Therefore, the soil-geosynthetic interface behaviour plays an important role and it should be accurately estimated. The mechanical behaviour of soil- geosynthetic interfaces depends on the physical soil properties and the geosynthetic characteristics. This study concentrate on the influence of particle shape on the mechanical behaviour of sand-geosynthetic interfaces for different initial void ratios and normal stresses. Experiments were conducted using a conventional-sized direct shear box. Angular sand and well-rounded glass beads with identical particle size distributions are selected as granular material. Geonet and non-woven geotextile are used as structural materials. Experiments revealed that all geosynthetic interfaces as the contact surface exhibit an abrupt loss of shear strength in the post-peak regime of behaviour. Also the results showed that particle shape significantly influences the interface behaviour.*

***Key Words*: Dilation, Direct shear, Geosynthetics, Particle shape, Sand-geosynthetic interfaces, Shear stress.**

1. INTRODUCTION

The use of geosynthetics to reinforce soil masses has been used for the past three decades, and they are now a well – accepted as construction material. Geosynthetics are used on many soil structures, such as, reinforced earth walls, reinforced slopes, embankments on soft soils, vertical landfills, and foundation soils. The use of reinforcements increase the resisting forces in the soil mass through the tensile force provided by the reinforcement, and thus reduces the horizontal deformations and increases the overall stability of the soil structure. In many cases, geosynthetics are not only an engineering solution but also an economic and environmental one. Geosynthetic-soil interface properties also play an important role in the safe and economical design of geosynthetic soil structures [2]. Therefore, accurate evaluation of the soil-geosynthetic interface behaviour by means of laboratory and numerical methods is an important factor.

The mechanical behaviour of soil-geosynthetic interfaces depends on the physical properties of soil (e.g., particle shape and size distribution, particle mean size, density, and degree of saturation), as well as the geosynthetics characteristics (such as structure and texture).

Among various experimental techniques developed for the investigation of soil-geosynthetic interaction, the most common ones are pull-out and direct shear tests. However, it is suggested that soil reinforcement interaction can be better characterized by direct shear test when sliding at the soil-geosynthetic interface is likely to occur. Such a condition may happen at the toe of reinforced soil slopes [5]. Large size direct shear boxes can be to study the mechanical behaviour of soil-geosynthetic interfaces. However, Anubhav and Basudhar (2013) reported that the maximum and residual shear strengths obtained from large size and traditional direct shear devices are relatively close. It should be noted that the peak shear stress is usually achieved in lower horizontal displacements in large scale shear tests [2].

This paper reports result of a study on the effects of particle shape on the mechanical behaviour of soil-geosynthetic interfaces. For this, the mobilization of shear strength and volume change response of interfaces between geosynthetics (geonet and non-woven geotextile) and fine angular sand and glass-beads were studied. The testing program covers a rather wide range of normal stress and initial void ratio values.

2. INTERFACE MATERIALS

2.1 Granular materials

A graded sand and glass beads were selected as angular and well-rounded granular materials for this study. The physical properties of angular sand and glass beads are given in Table 1. Scanning Electron Microscope (SEM) images for sand particles and glass beads are shown in Fig. 1. Both sand and glass beads specimens have nearly identical particle size distributions which was plotted in Chart. 1. So in this study, particle size is not considered as a variable. In accordance with the IS Classification System, both are categorized as poorly graded sand (SP).

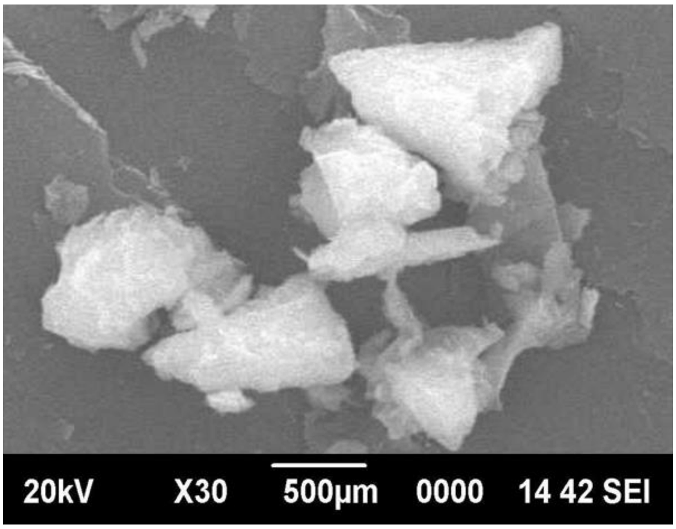
2.2 Structural materials

A non-woven geotextile and a geonet are used as structural materials. Table 2 presents general properties of the non-woven geotextile and geonet used is this study.

Table -1: Physical properties of granular materials used

Granular material	G	d_{50} (mm)	C_U	C_C	e_{max}	e_{min}
Angular sand	2.85	0.82	1.375	0.92	1.07	0.63
Glassbeads	2.5	0.82	1.375	0.92	0.7	0.48

(a)



(b)

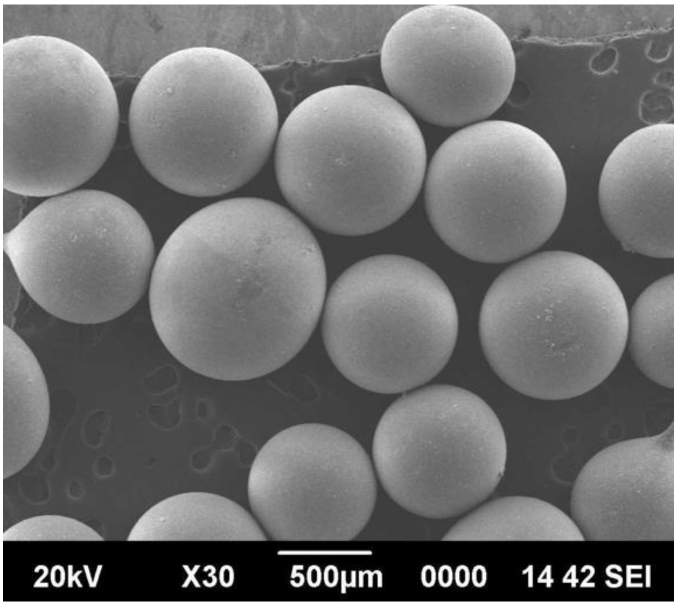


Fig -1: Scanning Electron Microscope (SEM) images for: (a) angular sand; (b) glassbeads.

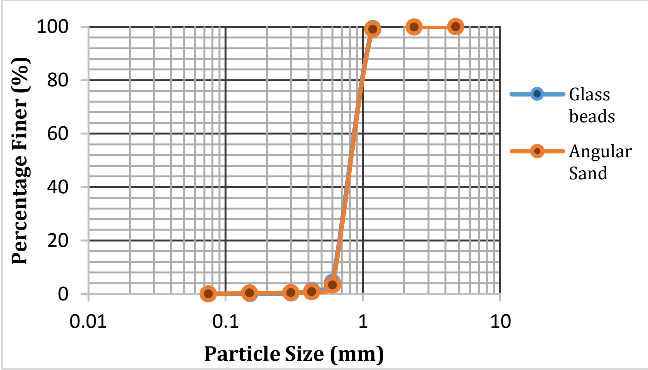


Chart - 1: Particle size distribution of fine angular sand and glass beads.

Table -2: Geosynthetic properties

Properties	Geosynthetic	
	Non-woven geotextile	Geonet
Material	Polypropylene (70%) with Polyethylene (30%)	Polyester
Thickness[mm]	0.75	2.75
Mass per unit area [g/m^2]	120	250
Mean peak strength [kN/m]	8.0	50
Tensile strength @ 5% elongation [kN/m]	3.4	15

3. TESTING PROGRAM

A series of soil-soil direct shear tests under normal stresses 49, 74, and 98 kPa was conducted to study the peak and residual shear strengths as well as the volume change response (dilation angle) of the angular sand and glass beads specimens. In all tests, dry specimens were poured into the shear box in three consecutive layers. For preparing medium and dense specimens, each layer was subjected to rodding. But layers were not compacted in preparation of loose specimens. Tests were conducted by a conventional direct shear apparatus. The shear box provides an inner dimension of 60 mm [length] x 60 mm [width] x 25 mm [height] for soil specimen. Before shearing, the initial void ratio of each specimen was calculated by direct measurement of the soil mass and the height of the specimen reduced as a result of normal stress:

$$e = G\rho_w\left(\frac{Ah}{m_s}\right) - 1 \quad (1)$$

where h, A (=36 cm^2) and m_s are, respectively, height, area, and mass of the soil specimen within the shear box. ρ_w = 1000 kg/m^3 is the density of water, and Gs = ρ_s/ρ_w in which ρ_s is the density of soil. The lower half of the shear box was kept fixed during experiments and the upper half was subjected to horizontal displacement.

For interface tests with geosynthetics, they were glued to a dummy steel block and was placed in the lower half of the shear box. Consequently, the upper half of the box was filled with granular material (sand or glass beads), and then compacted (if required) to the target density. The remaining stages of the experiment are identical to conventional soil-soil direct shear tests.

4. TEST RESULTS

4.1 Results of soil-soil direct shear tests

The direct shear test results of angular sand is shown in chart 2 and 3. The tests cover void ratios of e0 = 0.68, 0.87, and 1.03 as well as normal stress values 49, 74, and 98 kPa. Dense (e0 = 0.68) specimens have a peak shear strength and the volume change response is dilative. But, peak shear strength fades away gradually with increase in e0. Also, it is observed that the tendency towards dilation mitigates with the increase in void ratio (e0) and normal stress.

Granular soils with highly angular particles can be prepared in a wide domain of initial void ratios. On the contrary, in well-rounded granular soils, particles slip easily on each other and therefore, the difference between e_{max} and e_{min} is usually less than that for soils with angular particles. So, the mechanical behaviour of glass beads with e0 = 0.68 under normal stresses 49, 74 and 98 kPa is only studied here (Chart 4 & 5).

Comparing the results obtained for glass beads with angular sand, it is seen that both peak and residual shear strengths, as well as dilation of glass beads specimens are significantly less than the corresponding values for angular sand specimens. Also, the difference between peak and residual shear strengths in glass beads specimens is small.

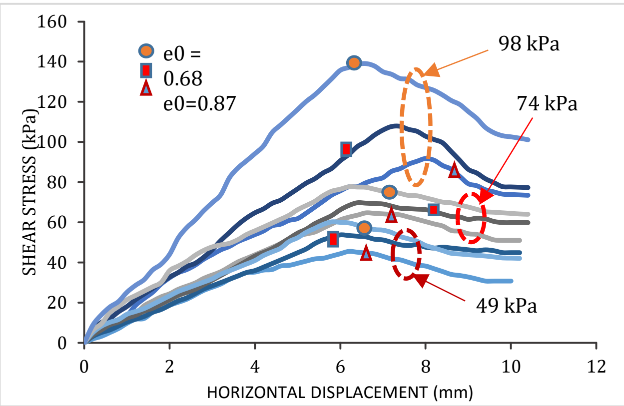


Chart -2: Shear stress vs. horizontal displacement of angular sand

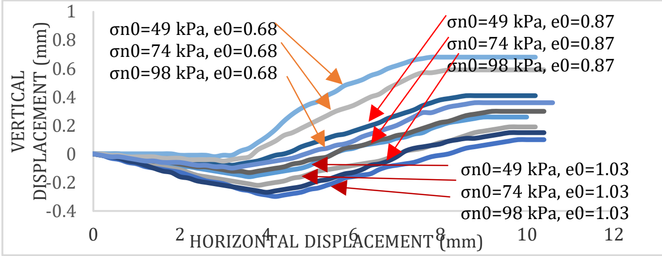


Chart -3: Vertical displacement vs. horizontal displacement of angular sand

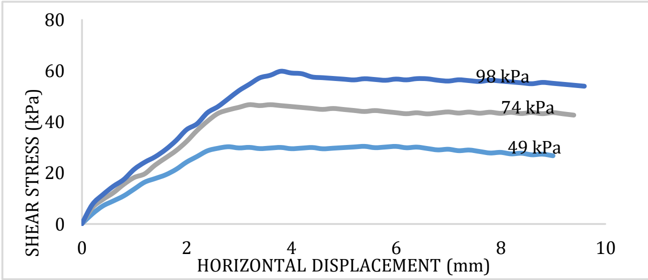


Chart -4: Shear stress vs. horizontal displacement of glassbeads

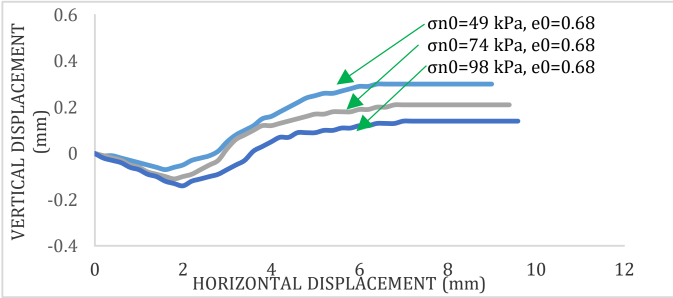


Chart -5: Vertical displacement vs. horizontal displacement of glassbeads

4.2 Results of interface tests

The behaviours observed in direct shear tests on interfaces between angular sand and non-woven geotextile are shown in chart 6 and 7. Also results for interfaces between angular sand and geonet are shown in chart 8 and 9. Sand specimens were initially prepared at e0 = 0.68, 0.87, and 0.81 under normal stresses 49, 74 and 98 kPa. Comparison of these results with the results for soil-soil direct shear tests indicates that shear strength mobilization in angular sand-geosynthetic interfaces are slightly faster. Among the geosynthetics used, shear strength mobilization of geonet was faster than non-woven geotextile. Dense interfaces are more prone to strain localization. The volume change response of dense angular sand-geosynthetic confirms this idea, in view of the fact that dense angular sand- geosynthetic interfaces exhibit a tendency towards dilation depending on the normal stress.

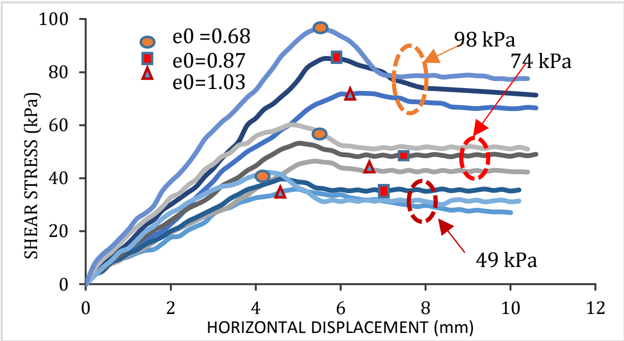


Chart -6: Shear stress vs. horizontal displacement for angular sand-non-woven geotextile interfaces

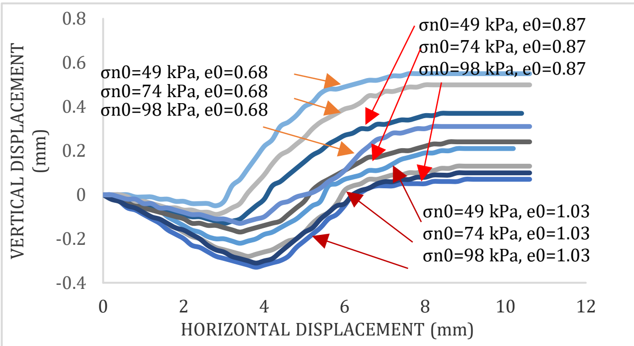


Chart -7: Vertical displacement vs. horizontal displacement for angular sand-non-woven geotextile interfaces

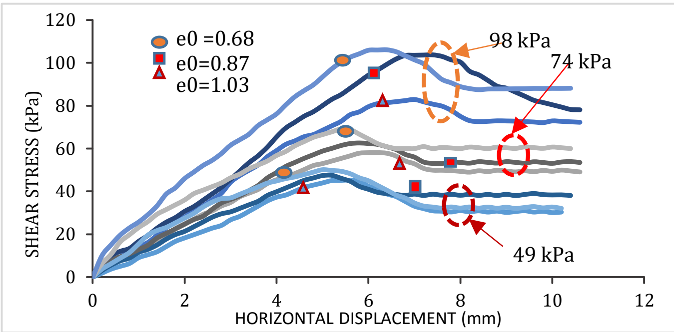


Chart -8: Shear stress vs. horizontal displacement for angular sand-geonet interfaces

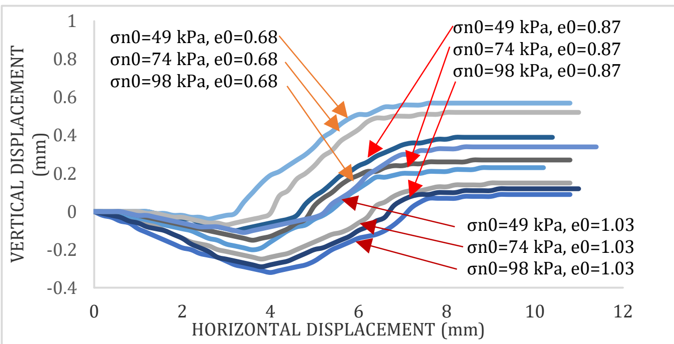


Chart -9: Vertical displacement vs. horizontal displacement for angular sand- geonet interfaces

For three tests on interfaces between glass beads (e0 =0.68) and the non-woven geotextile under normal stresses 49, 74 and 98 kPa, shear strength mobilization and volume change response are studied in chart 10 and 11. The results for glassbeads-geonet interface are given in chart 12 and 13. Comparing the results the mechanical behaviour of glass beads-geosynthetic interfaces is considered brittle. For both the interface tests, peak shear strength is mobilized and then it is followed by abrupt loss of shear strength, and dilation termination in the post-peak behaviour, resulting from strain localization within the interface zone.

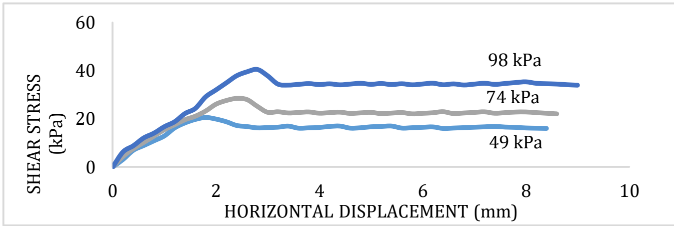


Chart-10: Shear stress vs. horizontal displacement for glassbeads-non-woven geotextile interfaces

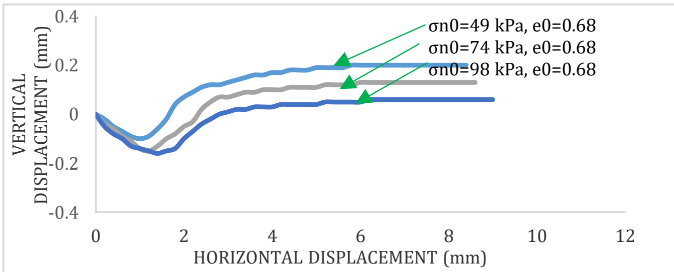


Chart -11: Vertical displacement vs. horizontal displacement for glassbeads -non-woven geotextile interfaces

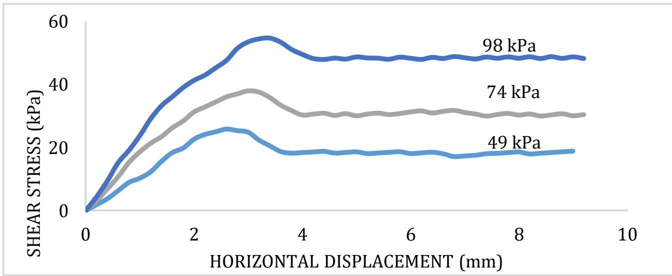


Chart-12: Shear stress vs. horizontal displacement for glassbeads-geonet interfaces

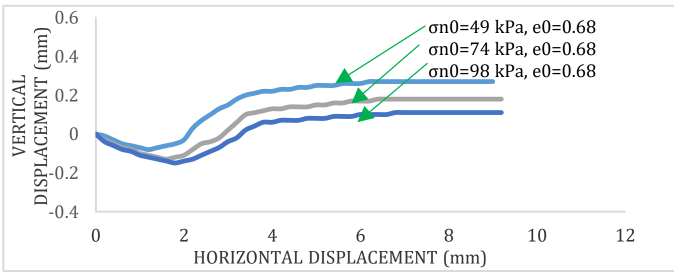


Chart -13: Vertical displacement vs. horizontal displacement for glassbeads-geonet interfaces

5. PEAK FRICTION AND MAXIMUM DILATION ANGLES

For soil-soil direct shear tests on angular sand and glassbeads, variation of peak friction angle (Φ_p) and maximum dilation angle (ψ_{max}), with respect to normal stress, are shown in (a) and (d) of chart 14 & 15. Peak friction angle is calculated by,

$$\phi_p = \tan^{-1}\left(\frac{\tau_p}{\sigma_n}\right) \quad (2)$$

where τ_p is peak shear strength. The maximum dilation angle is calculated by,

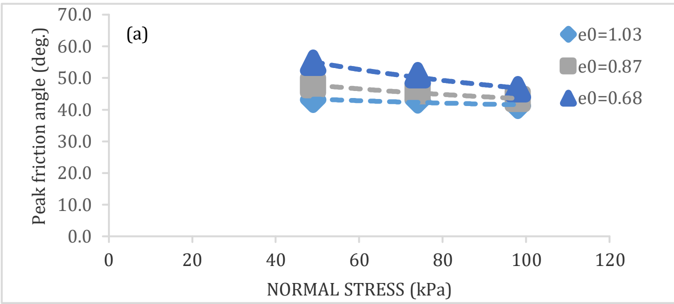
$$\psi_{max} = \tan^{-1}\left(\frac{\delta v}{\delta u}\right)_{max} \quad (3)$$

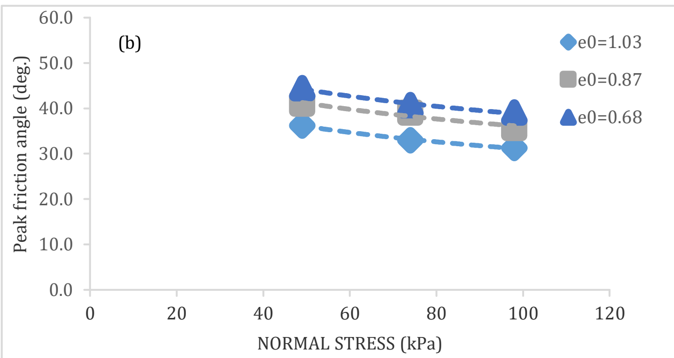
where $(\delta v/\delta u)_{max}$ is the maximum value of $\delta v/\delta u$ that is regularly attained around the state of peak shear strength.

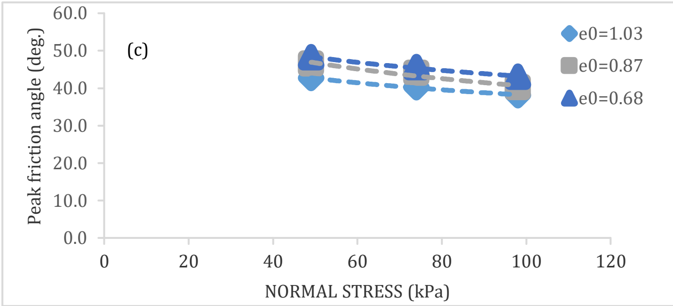
It is observed that Φ_p and ψ_{max} increase with the decrease in initial void ratio at identical normal stress. For the same value of initial void ratio, both Φ_p and ψ_{max} decrease gradually with the increase in normal stress.

The angular sand-non-woven geotextile interfaces and angular sand- geonet interfaces follows a pattern similar to those for angular sand which are plotted in (b) and (c) of chart 14 and 15.

The variation of Φ_p and ψ_{max} for glass beads-non-woven geotextile and glass beads-geonet interfaces with the change in normal stresses is studied in (d) of chart 14 and 15. For completeness, data obtained from direct shear tests on glass beads are also included in (d) of chart 14 and 15.







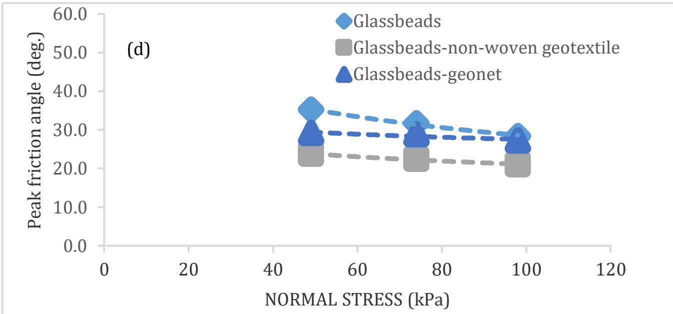


Chart -14: Variation of the peak friction with initial void ratio and normal stress for (a) angular sand, (b) angular sand - non-woven geotextile interfaces, (c) angular sand-geonet interfaces, (d) glassbeads, glassbeads-non-woven geotextile interfaces, and glassbeads-geonet interfaces

6. RESIDUAL FRICTION ANGLES

Large amplitude shearing in direct shear test brings the state of specimen to such an eventual steady state at which granular media/ interfaces shear continuously without any further change in stress state and void ratio. Most of the tests shown reached the asymptotic state of stress described above for which dilation angle becomes zero [1]. The residual angle is calculated by,

$$\phi_{res} = \tan^{-1}\left(\frac{\tau_{res}}{\sigma_n}\right) \quad (4)$$

where τ_{res} is the residual shear strength. For the granular materials and interfaces, values of residual friction angle , Φ_{res} = 38.66° and 28.81° are obtained, respectively, for angular sand and glass beads in soil-soil direct shear tests. The residual friction angles for angular sand-non-woven geotextile and angular sand-geonet interfaces are also presented in Table 3. It is observed that Φ_{res} of angular sand-non-woven geotextile is slightly less than that of soil-soil tests on angular sand. Similar behaviour was observed for angular sand-geonet interface. It is seen that Φ_{res} of angular sand-geonet is greater than angular sand-non-woven geotextile.

A similar comparison for the glass beads-non-woven geotextile interfaces and glass beads-geonet is given in Table 3. It can be observed that Φ_{res} of glass beads-non-woven geotextile and glass beads- geonet interfaces are less than that of glass beads obtained from direct shear tests.

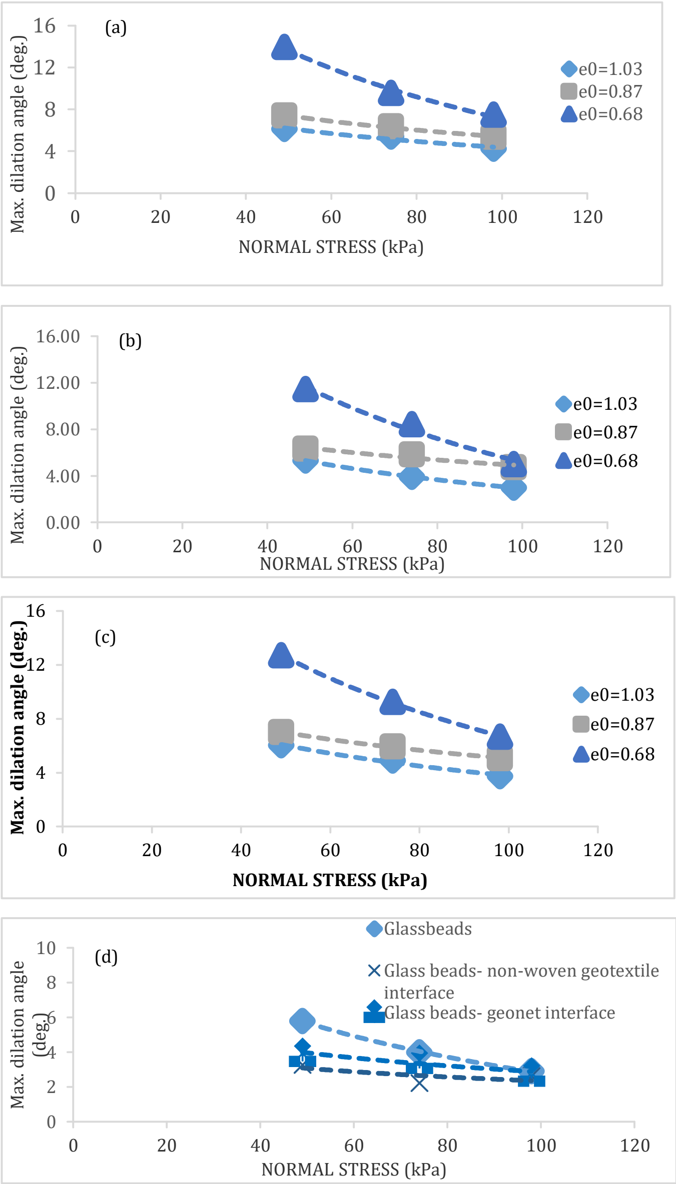


Chart -15: Variation of the maximum dilation angles with initial void ratio and normal stress for (a) angular sand, (b) angular sand -non-woven geotextile interfaces, (c) angular sand-geonet interfaces, (d) glassbeads, glassbeads-non-woven geotextile interfaces, and glassbeads-geonet interfaces

Table -3: Residual friction angles

Tests	Φ_{res}
Angular sand-non-woven geotextile interface	33.02°
Angular sand-geonet interfaces	36.87°
Glassbeads-non-woven geotextile interfaces	19.29°
Glassbeads-geonet interfaces	26.57°

7. CONCLUSIONS

The influence of particle shape on the behavior of sand-geosynthetic interfaces over a wide range of void ratios and normal stress was investigated. A uniformly graded angular sand, and well-rounded glass beads were used to simulate sands with angular and well-rounded particles. Based on the observations of this study, the following conclusions can be drawn:

- Particle shape have a great influence on peak and residual friction angles, and maximum dilation angle in sand-geosynthetic interfaces
- The peak friction angle and the maximum dilation angle of angular sand-geosynthetic interfaces were slightly lower than the corresponding values for angular sand in soil-soil direct shear test.
- The peak friction angle and the maximum dilation angle of well-rounded glass beads-geosynthetic interfaces were observed to be insensitive to the normal stress in the range of values studied here.
- All dense interfaces with geosynthetics as contact material exhibit a sudden loss of shear strength in the post-peak regime of the behavior.
- The peak friction angle of angular sand-geonet interfaces was slightly greater angular sand-non-woven geotextile.

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