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DIRECT SHEAR PROPERTIES OF LIGHTLY CEMENTED SOIL – POLYSTYRENE BACKFILL MATERIAL

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Abstract

Geocomposite materials consisting of lightly cemented mixtures of expanded polystyrene beads (EPS) and backfill soils are commonly used as fillers and for the construction of light weight embankments. The constant normal stress (CNS) and constant volume (CV) direct shear parameters as well as the relationship between gradation properties of four residual granular backfill soils and direct shear parameters of lightly cemented mixture of EPS and backfill soils was evaluated. The maximum density and optimum moisture content of the backfill materials ranged from 1850 kg/m^3 to 1730 kg/m^3 and 9% to 13%, respectively. The addition of 1.5% of EPS per mass of dry soil resulted in maximum density and optimum moisture content of the blended specimens of backfills soils and EPS that varied from 1303 kg/m^3 to 1368 kg/m^3 and 7% to 9%. For the range of normal stress used, the CNS friction angles of the four light weight composites were greater than the CV friction angles due to changes in initial normal stress and pore water pressure. Results from series of CNS tests show that both the vertical strain induced by consolidation normal stress and the shear induced vertical displacement for consolidation normal stress of 200 kPa and 400 kPa were significantly greater than the magnitudes induced by normal stress of 50 kPa and 100 kPa, due to reduction in bond strength during consolidation. The relationship between friction coefficients of the composites derived from CNS and CV test data and the uniformity coefficient of the backfill soils was non linear and dependent on the ratio of the sand to the gravel fractions. A practical implication of the two mode of tests is that for a composite material to mobilized the same magnitude of CNS friction angle under constant volume condition, a change in soil particle constitution is required. Both CNS and CV shear parameters should be considered in the design of retaining wall and embankments exposed to poor drainage conditions.

Key words: backfills, expanded polystyrene, friction coefficient, texture, shear strength

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1. Introduction

Infrastructures like bridge abutment, embankment, retaining walls and flexible pipelines founded on soft and sensitive profiles may be exposed to differential settlement, lateral displacement and connection failures when naturally occurring earth fill materials are used for their foundations backfills. The failure of such infrastructures can be prevented by the use of lightweight fill as an alternative geomaterial for the relief of the overburden pressure of the conventional earthfilling materials.

Polystyrene is a synthetic material produced from Naphtha, a by-product of petroleum. Heat induced expansion of pentene gas infused in polystyrene results in expanded polystyrene (EPS) beads and blocks. Polystyrene is a relatively cheap material, readily available, durable in harsh environments and often outlives the life expectancy of conventional construction materials. It is commonly used for pipe insulation, light weight fill material, sheet wall insulation, concrete moulds and backfill insulation (Esveld and Markine, 1993). Polythene bags in low economies with poor waste disposal programme often constitute an environmental nightmare; contribute to the reduction in landfill life and blockages of drainage infrastructures. Polystyrene fill was used for the extension of Highway

Bridge in Salt Lake City area that was underlain by alluvium deposit of soft clay and lacustrine silt. EPS block geofoam was applied to mitigate settlement of the bridge approach embankments that were constructed over compressible alluvial soils (Ossa and Romo, 2009). It has however been noted that because of their shape, EPS blocks were not suitable for constructions in confined spaces, inaccessible locations and areas with irregular shapes (Abdelrahman, 2010).

The use of lightweight fill materials for the construction of embankments on sandy soils, weak soils and dredged mud was investigated by a number soil engineering practitioners (Abdelrahman, 2010; Ghazavi, 2004; Liu et al., 2006; Tsuchida et al., 2001). The lightweight materials were produced by mixing EPS beads, polystyrene pre-puff (PSPP) beads, soils, water and cement. The lightweight materials were successfully used in the construction of road embankments but were found to be more expensive when cement stabilizer was used. Onishi et al. (2010) also investigated the strength and small strain modulus of cement stabilised sand mixed with expanded polystyrene beads and found that both properties can be improved or degraded depending on the amount of cement stabilizers. The shear behaviour of sand-EPS lightweight fills plays an important role in the stability and deformation of construction works, and thus deserves detailed investigation. The shear behaviour of the two-phase (sand-EPS) geomaterial composite is more complicated than that of naturally occurring geomaterials consisting of pure soils. The behaviour was found to be essentially associated with mixing ratios and mechanical interaction of sands and beads (Deng and Yang, 2010).

Ertugral and Trandafir (2011) investigated changes in lateral earth forces acting on non-yielding retaining wall by EPS inclusions and observed that the deformation of the EPS was concentrated in the bottom half of the retaining wall because of higher stresses in that zone. The inclusion of the polystyrene decreased the lateral pressure on the retaining wall to such an extent that it was possible to decrease the quantity of reinforcing required in the retaining wall and effectively save costs. The elastic and plastic deformation phases of EPS based backfill make the prediction of field settlement more complicated because the magnitude of elasto plastic deformation was found to be dependent on the magnitude of external load, loading rate and field drainage paths Xenaki and Athanasopoulos (2001).

The efficient disposal, recycling and reuse of polythene bags are common practice in the Provincial capitals and metropolitan areas of South Africa. They are however poorly disposed in the townships and rural areas and constitute environmental hazard and blockage of drainage systems. In order to reduce the reduce landfill loads, especially in Greater Johannesburg where the four major landfill sites are close to their service life, The use of marginal or composites materials for the construction of facilities like low cost road embankments that encourage the use of large volumes of recycled plastic materials are very important. The shear strength properties of compost materials are very important and useful in the design of road embankment. In this study, the shear strength properties of composite materials made from cement stabilized mixtures of EPS and Brixton red soil was investigated for embankment fill application in areas especially in the east rand that are underlain by heavily weathered Witwatersrand shale and carbonate soils and lightweight materials for the backfilling of retaining wall. Shear induce deformation parameters of lightly cemented soft soils containing EPS are determined in soil mechanics laboratory from the conventional constant normal stress (CNS) triaxial or direct shear tests. The conventional triaxial test mode assume that the external load on the sample remain constant up till failure. In reality for most in situ loading conditions, the principal stresses rotate during shear and are not constant, more so loading rate may be such that undrained condition exist in the material, and thus the material can deform in shear without volume change (Airey et al., 1988). Shear strength induced by constant volume deformation are well detailed in literature and are important design parameter and back analysis of rainfall induced failure of embankments, and stability of backfill supported structures at early stages of construction in poor drainage environment. Conventional direct shear tests at very slow strain rate that disallows pore pressure build up can be classified as drain tests and effective stress shear parameters can be obtained. However undrained shear strength parameters cannot be determined from conventional direct shear test data because of the drainage from the shear box, and thus constant volume tests provide an alternative method of evaluating the effect of restricted drainage on direct shear parameters.

Owing to the difficulty in the prevention of drainage in direct shear tests, Kjellman (1951) and Bjerrum and Landva (1966), determined undrained shear strength parameters of soft Drammen clayey soils from series of constant volume tests. They compared the constant volume tests data and undrained shear data in membrane walled shear apparatus and concluded that when pore pressure build up was prevented by the combination of strain rate and change in applied vertical stress, the effective stress in the sample was equal to the applied total stress. Escario and Saez (1986) determined the constant volume shear strength of compacted sands that exhibited strain softening behaviour after volume dilation and formulated useful relationships between rate of dilation and constant volume friction angle.

The aim of this research was to determine the shear strength parameters of mixtures of granular backfill soils and EPS beads (referred to as granular composites), and the shear strength parameters of lightly cemented mixture of EPS and granular backfill soils (referred to as cemented granular composites) under CNS and CV conditions. It was necessary for field applications to evaluate the effect of changes in gradation properties of the granular soil materials on the shear strength parameters of the lightly cemented granular composites as these relationships are important for the design of embankments constituting of cemented granular composites under different drainage conditions.

2. Material and methods

2.1. Material

The EPS beads used in this investigation and shown in Fig. 1, were collected from a textile factory in Johannesburg where they are further processed and used for the production of picture frames and furniture. They are spherical to near spherical superlight polymer materials which were made by pre-puffed polymer resins. The material was kept in a dry chamber before use.

The residual soil used is a common backfilling material derived from the weathering of quartzite. The reddish residual sandy soil of the Brixton formation was obtained from a road construction site in Auckland Park, Johannesburg. The Brixton Formation is part of the West Rand Group which forms part of the Witwatersrand Supergroup. The Witwatersrand Supergroup is made up of thick sequences of weathered shales, quartzites and conglomerates (Brink, 1984). X - Ray diffractographs of the mineral constitution, reveal the presence of quartz, hematite, muscovite, garronite and chloritoid. Quartz occurred at a relatively higher percentage in the soil followed by hematite, muscovite, garronite and chloritoid, respectively.

Bags of the residual quartzite was dried, mixed and sieves to remove soil fractions with diameter larger than 6.75mm and smaller than 0.075mm. Natural occurring red sandy soil of the Brixton formation underlying eastern Johannesburg and portions of eastern Gauteng Province does not contain particles larger than 6.75mm. The fines (< 0.075mm) vary from 0% to 12% and are either residually derived or are transported fines of the weathered Witwatersrand shales. The dry soil collected from a construction site contain 3% fines and was reconstituted to produce the following four major types of dry granular backfills; fines sands 0.075mm – 0.425mm, coarse sands 0.075mm – 4.75mm, and sandy gravel 0.075mm – 6.75 mm and gravelly sand 0.075 mm – 6.75 mm. Based on wet sieve analysis, the constitution were fines sands 0.075mm – 0.425mm (with 11% < 0.075 mm), coarse sands 0.075mm – 4.75mm (with 7% < 0.075 mm), sandy gravel 0.075 mm – 6.75mm (with 4% < 0.075 mm) and gravelly sand 0.075mm – 6.75mm (with 5% < 0.075mm). Most investigations on the behaviour of soil – EPS mixtures are focused on soft clays and uniform sands. The abundance of residual semi-arid sands in South Africa with different fine and coarse particle constitutions, most notably the Berea sand in Kwazulu Natal and the Kalahari sand of Northwest west province, provided the impetus for the investigation of the effect of gradation of granular backfill on the behaviour of Soil – EPS mixtures. The reconstitution also allow for the determination of shear strength properties in a conventional 100mm square shear box with minimal end effects naturally associated with larger diameter particles. Although the particle size diameter of the EPS in the factory was within the range of 2.35 mm and 6.75mm, only the major fractions with diameter range of 2.35mm to 4.75mm, i.e. uniformly graded EPS was investigated as variation in the particle size of EPS has been shown to affect the mobilized shear strength of EPS – Sand mixtures. The particle size distribution curves of the reconstituted dry soil and the EPS was presented in Fig. 2. The light weight specimens were produced by compacting and curing mixtures of the reconstituted soils with 1.5% EPS and 3% cement by mass of the backfill soil.

To determine the maximum dry density (MDD) and optimum moisture content (OMC), the cement stabilized specimens were compacted in a standard compaction mould by the application of 25 blows per layer to three layers of each material. The backfill soils and 3% cement by mass were mixed with different percentages of water. 1.5 % of EPS beads by mass of dry soil was added, mixed thoroughly to a homogenous state and transferred into the compaction mould in different portions and compacted. The series of compaction tests were conducted to determine the effect of EPS proportion on the optimum moisture content and dry density of cement stabilized backfill materials. Trial mixes with different percentages of EPS were also prepared. It was found that mixtures containing up to 2% EPS were rubbery, harsh, and difficult to mix homogenously and compact.

Ordinary Portland Cement (OPC) was used as a binder. The specific gravity of the cement was 3.60. The compressive strength of the mortar was 22 MPa after 3 days and 30 MPa after 7 days. 3% of the cement by mass of the backfill was used for the stabilization to provide adequate binding for the composite material. For the construction of embankments, and road subgrades higher percentage of cement was deemed to be too expensive. The optimum moisture content and dry densities of the cement stabilized mixture of EPS and backfill soils were used to prepare the specimens for shear strength test.

Series of preliminary tests were conducted to justify the selection of test conditions and initial states for the determination of shear parameters. The result of preliminary investigation on the effect of curing period on the UCS of 3% cement stabilized EPS – Fine Sand was presented in Table 1.

Table 1. The effect of curing period on the UCS of 3% cement stabilized EPS – Fine Sand

Number of Samples	*Curing Period (Days)*	*UCS (kPa)*
2	3	1200
3	7	1545
3	14	1567
2	28	1656

2.2. Direct shear test

For the direct shear tests, the freshly compacted materials were cut into 100mm square specimens with 40mm thickness. The direct shear specimens were kept soaked for 6 days in a curing room with constant humidity of 80% and temperature of 20˚C to ensure that sufficient water was available for cement hydrations. On the seventh day, the specimens were placed in a shear box, loaded to the desired normal pressure, saturated and then tested. Curing was effected by continuously dripping water to the specimens through wet cloths. After curing the specimens were transferred into the shear box, Water was again gradually poured into the shear box container and observed to rise to the surface of the sample to displace the air in the specimen voids, before the consolidation load was added. This is similar to the air displacement procedure before triaxial tests.

Water was poured into the shear box to ensure that the sample remain saturated during the shear phase of the test. The consolidation of the specimen induced by the application of normal tress was recorded and shearing was initiated after t_{90}. The conventional constant normal stress direct shear (CNS) tests and the constant volume direct shear (CV) tests were the two stress path tests conducted in a shear box apparatus and fabricated constant volume loading frame system shown in Fig. 3. For the constant volume tests, the bolts and the shaft were continuously adjusted to ensure that the vertical dial gauge remain in an initial position after consolidation thus ensuring zero shear induced volume change. A two way null load systems comparable to the function of the belloframs in triaxial tests were used. It was induced from a compressor to a counter frame to provide relief of the applied load. In addition, two layer bearing bolts were used to ensure ease of volume adjustment. The constant volume tests assembly consists of two threaded vertical stiff rods connected to the mainframe at the two sides of the simple shear device. A thick reaction frame, with holes drilled at the ends is slotted to the top of the vertical rods and held in position by means of nuts. Twine null pressure membranes were placed between the reaction frame and a pair of short pieces of rod below which an S - shape load cell is connected. The initial normal stress is applied by the two nuts on top of the stiff reaction bar and is transmitted from the load cell to the samples through a set of ball bearings resting on top of a slightly modified pressure pad.

The shear tests were conducted on compacted specimens of homogenously blended EPS beads and soil. The mixture was blended with water equal to the optimum moisture content of the soil and was compacted into the shear box mould at maximum dry densities and moisture content. The specimens were consolidated with applied normal stresses of 50kPa, 100kPa, 200kPa and 400kPa. The sample were then saturated and sheared at a displacement rate of 0.2mm per mins. The results of preliminary CNS tests on compacted specimens of fine sands showed that at 20% displacement, an average of 1mm dilatant vertical displacement was indicated after consolidation due to the applied normal stresses and the time for this shear displacement is 20mins. It was not possible to determine whether the selected displacement rate resulted in drained condition as the sample did not compress dilated and thus volume change could not related to the time for 90% of the consolidation of the specimens in the oedometer (t_{90}) and the coefficient of consolidation. However direct shear conditions in which the specimen dilatancy was suppressed are likely to result in undrained conditions as drainage is inhibited by the increase in initial normal stress. However for compacted specimens consisting of fine sand and EPS, the t_{90} decreased marginally with the consolidation stress; the average value was 14 mins. Thus the shear displacement rate of 0.2mm per mins, resulted in drained conditions in the specimen. Thus to ensure constant during shear, the initial normal stress was decreased. For a given value of normal stress, the shear tests were run two or three times and the averages of the shear stress results were used.

Thus the strength envelope can be represented that Mohr Coulomb failure criterion typically associated with triaxial test parameter (Coduto, 1999) given by Eq. (1).

$$\tau' = \sigma'_n \tan\varphi' + c'. \tag{1}$$

where: τ' = shear strength; σ_n'= effective normal stress; $tan\,\varphi'$ = coefficient of intergranular friction; c' = effective cohesion.

Effective stress parameters can be determined from undrained triaxial tests where pore pressures are monitored. In the conventional direct shear tests without membranes, pore pressures can be induced when the specimens are subject to fast displacement rate and volume change is prevented. The CV tests at the same displacement rate permit the study of mobilized shear stress at undrained conditions, and for this case the strength envelope is dependent on the normal stress at failure which may be higher or lower than the initial normal stress, depending on the volume change exhibited by the specimen.

For direct shear tests conditions where the strength envelopes are dependent on specimen volume, pore water pressure and changes in initial normal stress, the CNS direct shear strength envelope can be represented by Eq. (2):

$$\tau_d = \sigma_d \tan\varphi_d + c_d \tag{2}$$

For CV conditions (Eq. 3):

$$\sigma_{cv} = \sigma_d \pm \Delta\sigma_d \tag{3}$$

Noting that drained conditions in the shear box test is only prevalent in tests conducted at very slow strain rate, in this study, the effective stress sign was used to distinguish the CNS and CV envelopes.

Fig. 1. Expanded polystyrene beads (EPS)

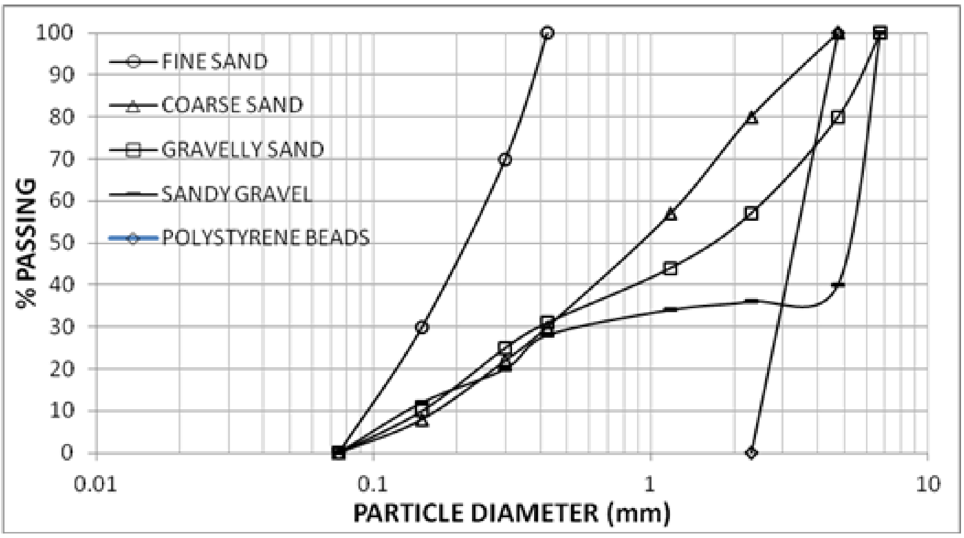


Fig. 2. Particle size distribution curves of granular backfill soils and EPS

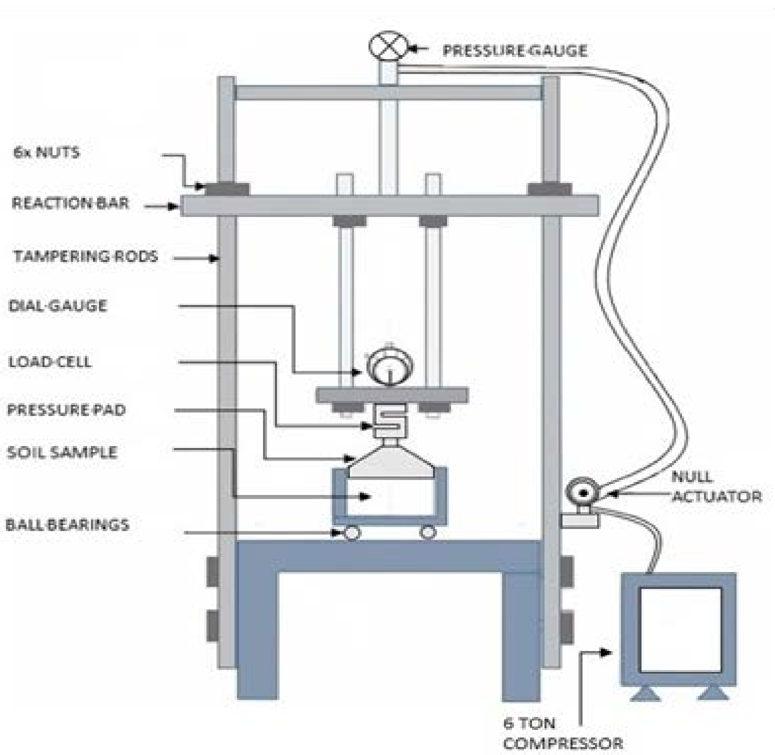


Fig. 3. Direct shear apparatus with constant volume frame

Table 2. The textural properties of the granular backfill materials

Particle size distribution	*FINE SAND*	*COARSE SAND*	*GRAVELLY SAND*	*SANDY GRAVEL*
D_{10}	0.095	0.17	0.16	0.14
D_{30}	0.16	0.42	0.43	0.55
D_{50}	0.22	0.9	1.7	5.2
D_{60}	0.26	1.3	2.6	5.6
Cu	2.736842	7.647059	16.25	40
Cc	1.036437	0.79819	0.444471	0.385842
Gs	2.68	2.67	2.67	2.66
USC	SP	SP	SP	GP

3. Results and discussion

3.1. Gradation properties

The textural properties of the backfill soils were determined from the particle size distribution curves shown in Fig. 2. The major textural properties of the granular backfill i.e. the particle size that will permit 10%, 30%, 50% and 60% of the granular backfill soils (D_{10}, D_{30}, D_{50}, D_{60}) materials as well as the coefficient of uniformity Cu and the coefficient of curvature Cc were presented in Table 1. Based on the unified system of soil classification, for a soil specimen with less than 12% of fine particles it is necessary to determine the Cu and Cc. It was noted that a well graded soil has a uniformity coefficient of greater than 4 for gravel and 6 for sands, and a coefficient of curvature or gradation between 1 and 3 (Coduto, 1999; Das and Sobhan, 2013). Values outside this range indicate a poorly graded soil. Poorly graded soils are gap graded or uniformly graded soils. Table 1 showed that soils are poorly graded sands ad gravel. Since the PI of all the soils is 6%, dual symbols were used for the soils classifications (Holts and Kovacs, 1981).

The particle size range of the EPS beads was 2.3 mm to 4.74 mm and thus the beads are uniformly graded. The specific gravity and dry density are 0.021 and 21.097 kg/m^3.

3.2. Compaction

The compaction curves of the backfill materials and the stabilized specimens of EPS and backfill materials are shown in Fig. 4 and Fig. 5, respectively. The maximum density and optimum moisture content of the backfill materials ranged from 1850 kg/m^3 to 1730 kg/m^3 and 9% to 13%, respectively. Thus, a trend of increasing dry density with increase in the textural properties was evident. 90% of these values are above the 1650 kg/m^3 benchmark of in situ residual soils from Southern Africa that are commonly associated with high likelihood of moisture induced collapse settlement (Brink et al., 1982; Knight, 1961).

The addition of 1.5% of EPS resulted in decrease in maximum dry density. The maximum density and optimum moisture content of the blended specimen of backfills soils and EPS varied from 1303 kg/m^3 to 1368kg/m^3 and 9% to 7% and unlike Fig. 4, the trend of increasing dry density with increase in textural properties was not evident in Fig. 5.

3.3. Direct shear stress and deformation of compacted fine sand, by CNS tests

The shear stress and deformation curves of fine sand were presented in Fig. 6. For the range of applied normal stress, the fine sand exhibited direct shear induced volume dilatancy, after consolidation due to applied normal stress.

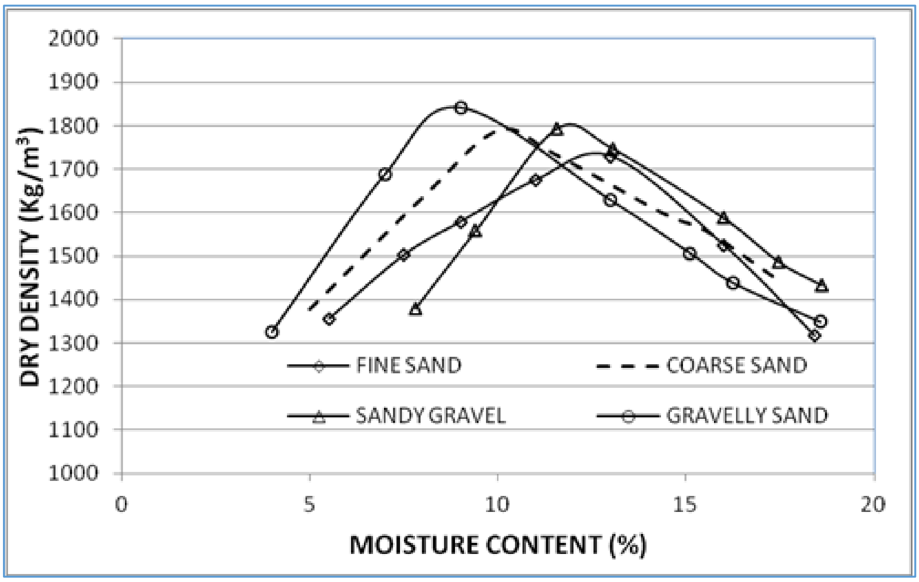


Fig. 4. Compaction curves for granular backfill materials

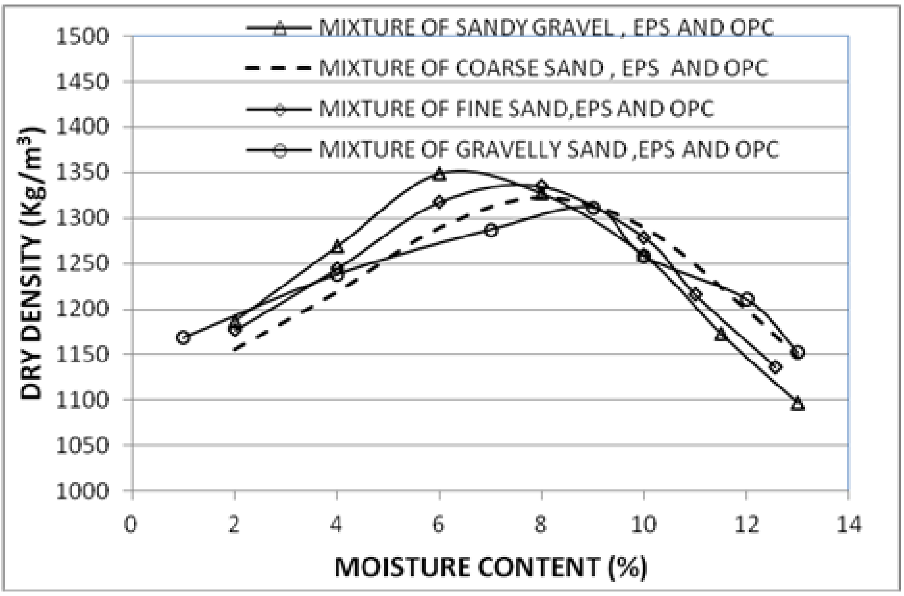
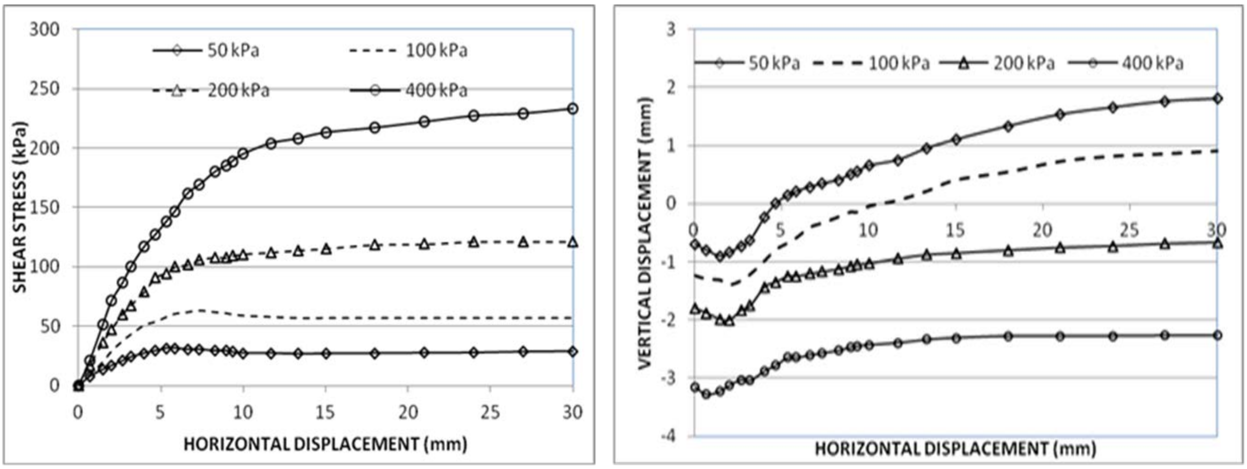


Fig. 5. Compaction curves of 3% cement stabilized backfill and EPS beads



(a) (b)

Fig. 6. CNS stress and deformation curves for fine sand

The degree of dilantancy i.e. the slope of the vertical displacement curves (dv/du) decreased with the magnitude of applied normal stress, while the specimen consolidation or vertical strain (dv/H) increased with the applied normal stress. Thus a general inverse relationship exists between magnitude of consolidation and degree of dilatancy. For applied normal stress of 50 kPa and 100 kPa, the peak shear stress was mobilized at horizontal displacement less than 10mm, while specimens subject to normal stress of 200 kPa and 400 kPa exhibited mild strain hardening with up to 80% of the shear stress mobilized at displacement less than 10 mm. The strength of specimens subject to normal stress of 200 kPa and 400 kPa was taken as the shear stress mobilized at 20% displacement or displacement of 20 mm.

The constant volume shear stress curves of compacted fine sand specimens subject to direct displacement were presented in Fig. 7. The constant volume stress displacement curves reveal mild strain hardening behaviour. The strain softening behaviour observed in Fig. 6 at initial normal stress of 50 kPa and 100 kPa were not evident in Fig. 7. The dilatancy observed in Fig. 6 was prevented by the increase in normal stress. Both the magnitude and the ratio of increase in normal stress to the initial normal stress increased with the initial applied normal stress.

The strength envelope for CNS condition was based on the peak stress of the strain hardening curves (50 kPa and 100 kPa) and the shear stress at 20 mm displacement for the strain hardening curves (200 kPa and 400 kPa) of Fig. 6. The strength envelope for CV condition was based on the shear stress mobilized at 20 mm displacement for the strain hardening curves of Fig. 7 and the corresponding normal stress. The CNS and CV strength envelopes were presented in Fig. 8. The constant volume friction angle φ'_{cv} (19°) was significantly smaller than the constant normal stress friction angle φ'_{CNS} (28°), because of the effect of pore water on the intergranular contact friction in constant volume test. The constant volume cohesion was greater than the constant normal stress cohesion again because it reflects to some degree, the consolidated undrained behaviour. The CNS and CV strength envelopes are given as Eqs. (4) and (5), respectively. The constant volume strength envelope was a linear approximation of a curved envelope. Curved envelopes are common in rocks and naturally cemented soils, and for these materials the stress ratio decreases significantly at high applied normal stress.

$$\tau = 0.533\sigma_n' + 8.5174 \quad R^2 = 0.9984 \tag{4}$$

$$\tau = 0.3471\sigma_n + 38.592 \quad R^2 = 0.9814 \tag{5}$$

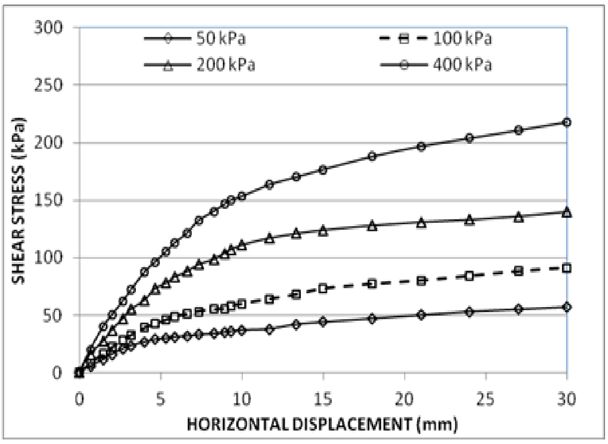
3.4. Direct shear stress and deformation of compacted mixtures of fine sand, EPS by CNS and CV tests

The results of CNS direct shear tests on the compacted mixture of fine sand and EPS are shown in Fig. 9. For the range of applied normal stress, the blended material exhibited mild strain hardening behaviour; however the maximum shear stress was evident at displacement greater than 20mm. The specimens consolidated due to applied normal stress and the magnitude of consolidation i.e. the initial vertical strain, increased with the normal stress.

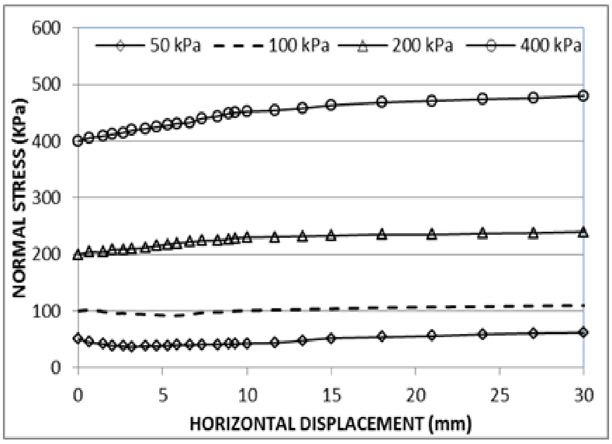
The shear induced volume compression increased with the applied normal stress. The consolidation induced vertical strain (-dv/H) and the magnitude of shear induced compression (-dv) also increased with applied normal stress. At large imposed shear displacement greater than 20 mm, residual conditions was not evident and there is coincidence of shear stress and shear induced volume compression as both parameters continue to increase. The shear stress at 20 mm displacement was thus used to determine the shear strength. The stress ratio also decreased with applied normal stress. The constant volume shear stress curves of compacted fine sand specimens subject to direct displacement were presented in Fig. 10.

Table 3. Shear stress parameters of Fine sand in CNS and CV test conditions

CNS (kPa)		*CV (kPa)*	
Normal Stress	***Shear Stress***	***Normal Stress***	***Shear Stress***
50	31.6	56	49
100	63.5	107	79
200	119	235	131
400	222	471	197



(a)



(b)

Fig. 7. Constant volume shear stress displacement curves for fine sand

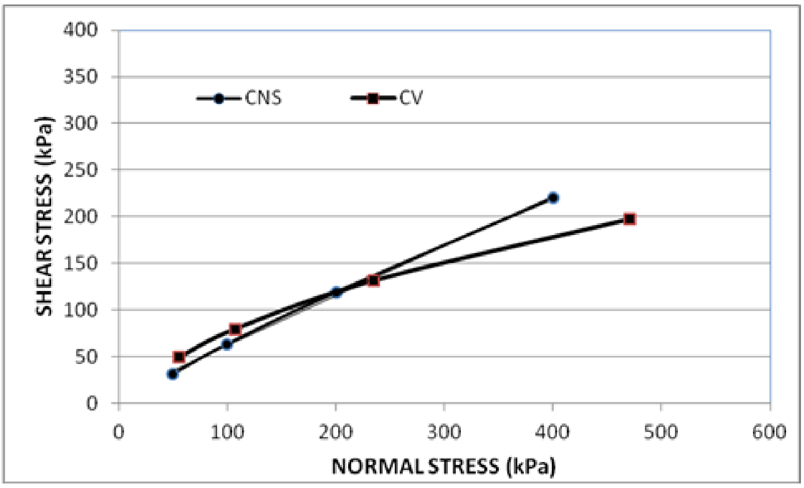


Fig. 8. CNS and CV strength envelope of fine sand

The constant volume shear stress curves reached maximum shear stress values at horizontal displacement of 10 mm and the stress ratio i.e. the ratio of the maximum shear stress to corresponding normal stress decreased with increase in initial normal stress. The shear induced compression observed in Fig. 9 was prevented by decrease in normal stress and both the magnitude and the ratio of increase in normal stress to the initial normal stress increased with the initial applied normal stress. At horizontal displacement greater than 10mm, the mobilized shear stress and vertical displacement tend to constant values.

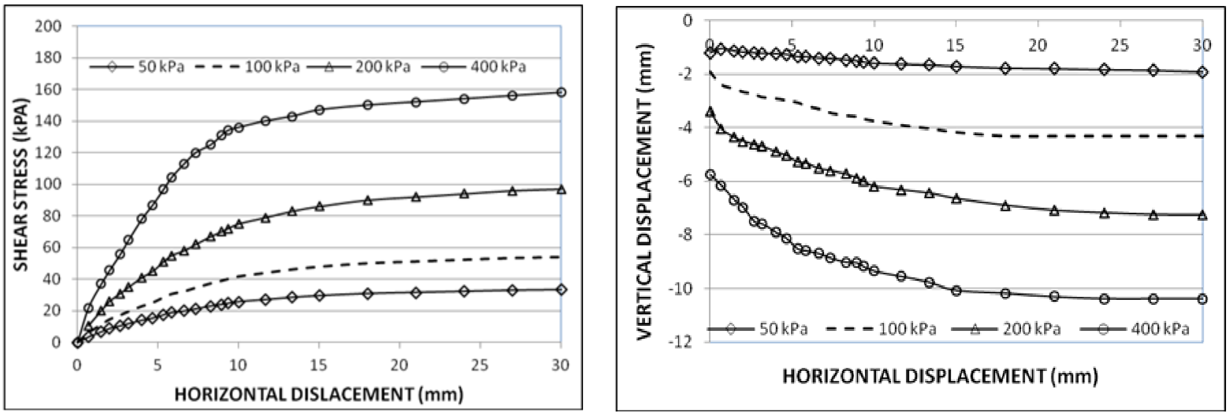
The strength envelope for CNS test condition was based on the shear stress mobilized at 20 mm displacement for the strain hardening curves of Figs. 9 and 10. The strength envelopes shown in Fig. 11 revealed that the effect of cementation was evident in the lower range of applied normal stress. The constant volume friction angle ϕ_{cv} (10°) was less than the constant normal stress friction angle ϕ'_{CNS} (17.7°), while the constant normal stress cohesion was insignificant due to the low plasticity of the composite. The CNS and CV strength envelopes are given as Eqs. (6) and (7), respectively.

$$\tau = 0.321\sigma'_n + 4.304 \qquad R^2 = 0.9916 \tag{6}$$

$$\tau = 0.1818\sigma_n + 10.733 \quad R^2 = 0.9966 \tag{7}$$

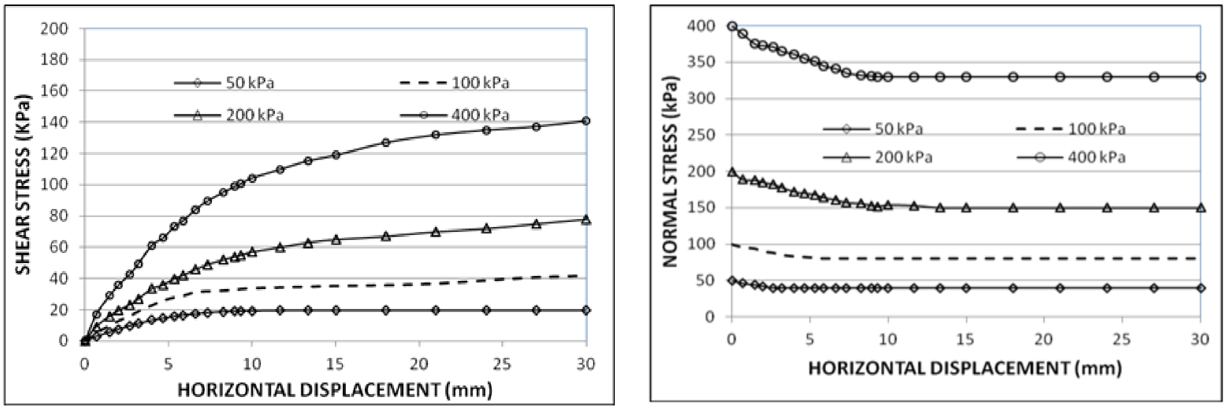
3.5. Direct shear stress and deformation of compacted mixtures of cement stabilized fine sand, EPS and 3% cement by CV tests

The result of direct shear tests on cement stabilized specimens of fine sand and EPS was presented in Fig. 12. In comparison with Fig. 9, the effect of 3% cement stabilization was a reduction in magnitude of normal stress induced vertical compression and increase in mobilized shear stress. For applied normal stress of 50 kPa and 100 kPa, the specimens exhibited shear induced dilatancy at displacement less than 10 mm and shear induced compression at larger displacement. For normal stress of 200 kPa and 400 kPa the specimen exhibited shear induced compression. The difference between shear induced vertical compression of the uncemented specimens and uncemented specimens decreased with increase in normal stress and was zero for applied normal stress of 400 kPa.



(a) (b)

Fig. 9. CNS stress and deformation curves for fine sand composite



(a) (b)

Fig. 10. CV stress and deformation curves for fine sand composite

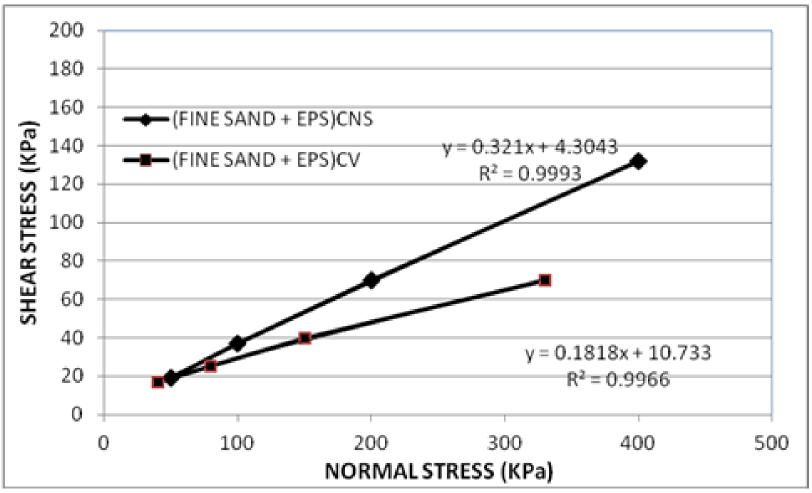


Fig. 11. CNS strength envelope for fine sand composite

However the vertical displacement or the consolidation settlement of the uncemented composite was greater than the settlement of the cemented specimen for the entire range of applied normal stress (50 kPa – 400 kPa). Thus, the applications of normal stress of 400 kPa did not result in complete destructuring of the cemented specimen. The effect of cement bond was however most evident in specimens that were consolidated with normal stresses of 50 kPa and 100 kPa. The mild dilatancy exhibited by the specimens was evident of the presence of cement bond (Hamidi et al., 2009).

The effect of cement stabilization on the constant volume direct shear stress and displacements of compacted fine sand specimens were presented in Fig. 13. For applied normal stress of 50 kPa, 100 kPa and 200 kPa the constant volume shear stress curves reached maximum shear stress values at horizontal displacement of 10 mm, while applied normal stress of 400 kPa resulted in strain hardening behaviour. The difference in shear stress mobilized by the uncemented and cemented specimens increased with increase in initial normal stress. The shear induced compression or vertical displacement observed in Fig. 10 for applied normal stresses of 50 kPa and 100 kPa exhibited mild dilatants behaviour at shear displacement up to 9 mm, followed by volume compression at greater horizontal displacement. This behaviour was reflected in the change in normal stress for initial normal stress of 50 kPa and 100 kPa in Fig. 13. However for initial normal stress of 200 kPa and 400 kPa, decrease in applied normal stress was observed for the range of imposed horizontal displacement.

The strength envelope for CV condition was based on the shear stress mobilized at 20mm displacement for the strain hardening curves of Fig. 12 and Fig. 13. The strength envelopes for the uncemented and cemented specimens were presented in Fig. 14. The constant volume friction angle ϕ'_{cv} (19.2) was less than the constant normal stress friction angle ϕ'_{CNS} (12.8), while the constant volume cohesion the constant normal stress cohesion were equal. The CNS and CV strength envelopes are given as Eqs. (8) and (9), respectively.

The effect of initial normal stress on stress ratio of fine sand and cemented fine sand composite was presented in Fig. 15. For the CNS and CV test condition, as the normal stress increased the stress ratio of the cemented speciemen tend to the stress ratio of the uncemented speciemen. Fig. 15 also indicate that the rate of reduction of stress ratio was maximum for increase in normal stress from 50 kPa to 100 kPa. This behaviour was also reflected in Fig. 16 which illustrates the effect of normal stress on consolidation strain of cemented and uncemented fine sand composite for CNS tests. Here both curves indicate the presence of structure in the speciemens for normal stress range of 50 kPa to 100 kPa. For norla stress greater than 100 kPa, the speciemens behaved like a normally consolidated material.

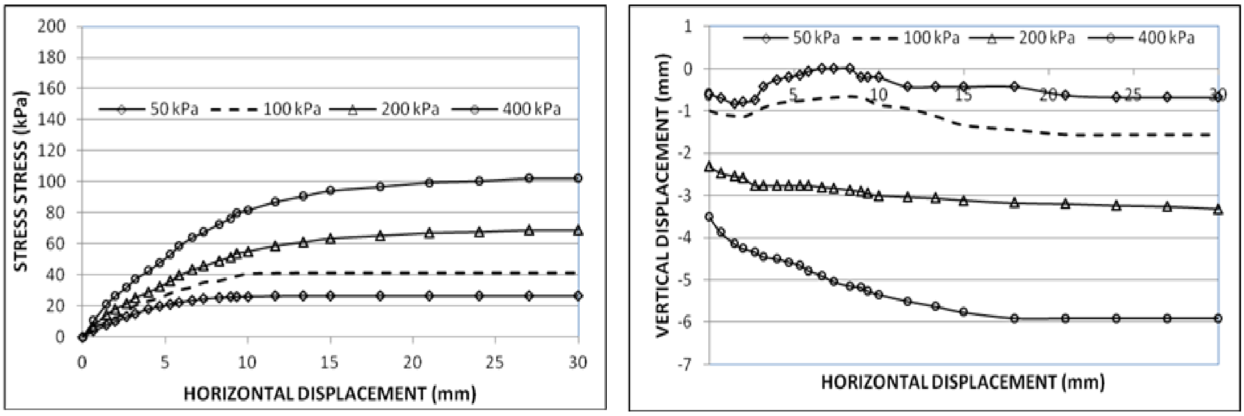
Thus for speciemens tested at high normal stress, destructuration was not due to shear indced compression. The substantial shear indcued compression indicated by speciemnes subject to normal stress of 200 kPa and 400 kPa in Fig. 9 was due to consolidation induced remoulding of the speciemens. Thus, 3% cement stabilized lightweight embankment subject to live loads greater than 100 kPa have a high likelihood to exhibit significant consolidation settlemet and slope instability.

$$\tau = 0.3449\sigma'_n + 18.08 \qquad R^2 = 0.996 \qquad (8)$$

$$\tau = 0.2295\sigma_n + 19.12 \qquad R^2 = 0.9826 \qquad (9)$$

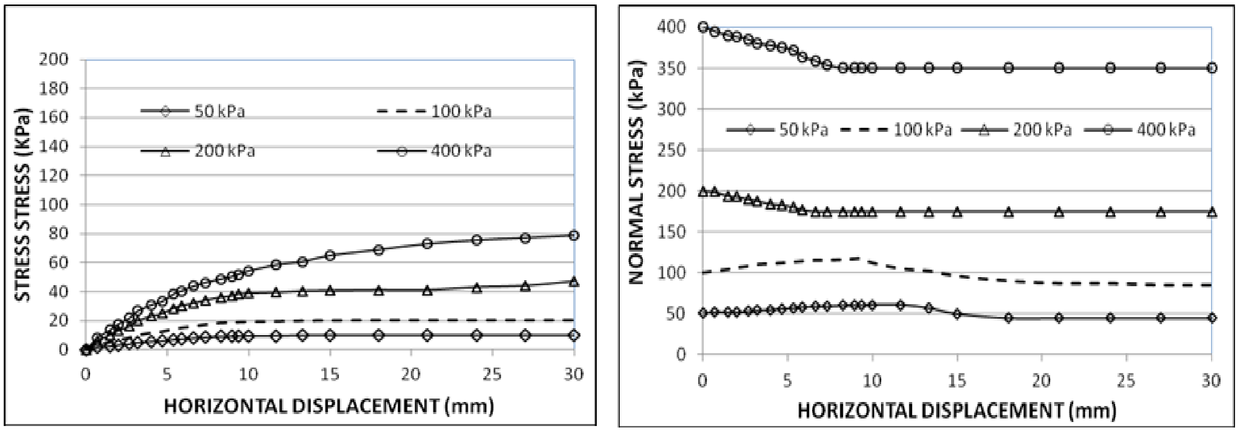
3.6. Effect of Gradation on strength of cement stabilized granular lightweight backfills

The direct shear stress and deformation behaviour of specimens prepared from cement stabilized mixtures of coarse sand, gravel sand and sandy gravel respectively with EPS followed similar trend, however the magnitude of mobilized maximum shear stress and shear induced vertical deformation varied.



(a) (b)

Fig. 12. CNS stress and deformation curves for cemented fine sand composite



(a) (b)

Fig. 13. CV stress and deformation curves for cemented fine sand composite

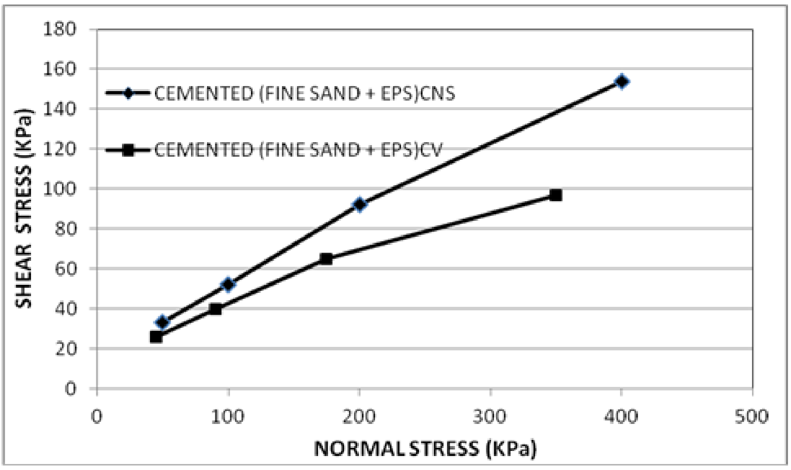


Fig. 14. CV strength envelopes for fine sand and cemented fine sand composite

The Mohr Coulomb strength envelopes of the lightweight materials were presented in Table 4. For the more granular materials, i.e. with the exception of fine sand, cementation resulted in significant increase in friction coefficient. This trend is true for both the CNS and CV strength envelopes.

However the frictional coefficient of the CNS envelopes were respectively greater than the CV values. The magnitude of cohesion did not follow any particular trend both in relation to the fines content and the mode of test. The relationship between friction coefficients of the cemented composites and the coefficient of the granular backfill soils was presented in Fig. 17. The relationship was non linear, the friction coefficients mobilized in the CNS and CV tests increased sharply for Cu between 2.73 and 16.25 and gradually tends to a constant value at Cu value greater than 40.

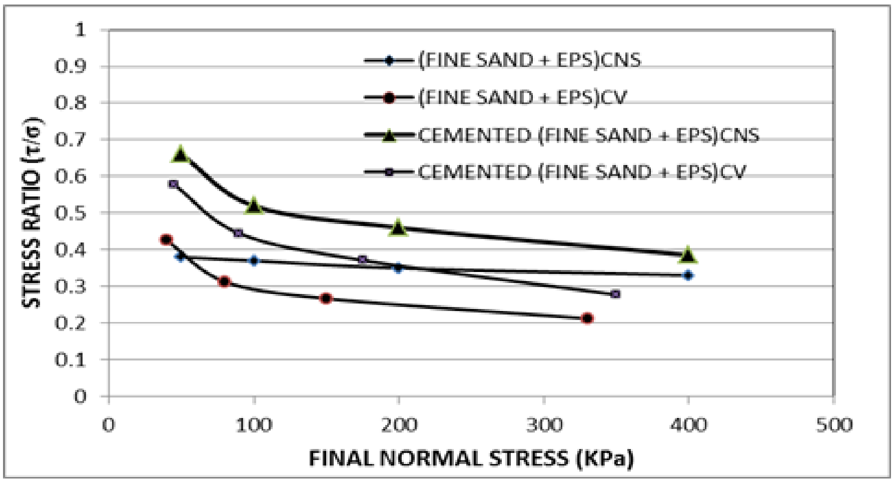


Fig. 15. The effect of initial normal stress on stress ratio of fine sand and cemented fine sand composite

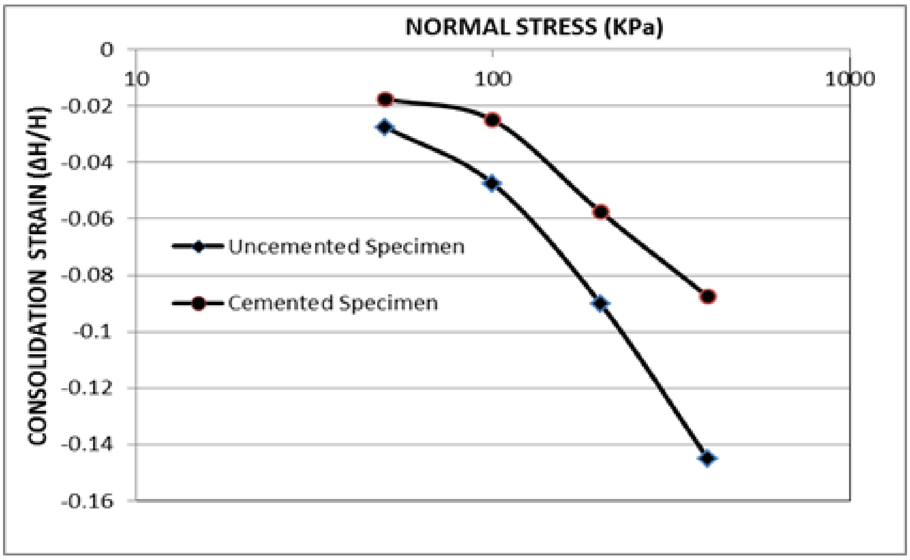


Fig. 16. The effect of normal stress on consolidation strain of cemented and uncemented fine sand composite for CNS tests

The Cu of the granular soils investigated and shown in Fig. 2, have a very narrow margin of d_{10} and very wide margin of d_{60}. The fines content are almost equal, thus the change in frictional coefficient was due to the content of coarse materials.

For coarse fractions or gravel content (< 4.75mm) greater than 20%, the increase in mobilized strength decreased and gradually tend to a constant value. Thus for both the CNS and CV tests, the shear strength of the light weight granular backfill was dependent on the ratio of the sand to the gravel fractions. For a speciemen to mobilize a friction angle in constant volume equal to the CNS friction angle, a more granular soil with a greater value of Cu is required.

3.7. Slope stability analysis and embankment Walls

Charts derived from the friction circle method has been used for the stability analysis of clay pit slopes, projected open pit slopes for crushed waste rock mass, mining and construction waste dumps and highly altered and weathered rocks based on limit equilibrium conditions (Hoek and Bray, 1999). For such materials, failure may occur along a surface which approaches a circular shape. The general structure of highly cemented granular materials and EPS beads falls under this category.

The factor of safety against failure (F) of a drained slope embankment with slope angle ψ and slope height H is expressed by Eq. (10).

$$F = \frac{\tan\varphi'}{\tan\psi} + \frac{c'}{\gamma H \cos^2\psi \tan\psi} \tag{10}$$

F is generally defines as the ratio of the shear strength for sliding resistance to shear strength mobilized along the failure surface. Slope stability analysis based on direct shear parameters derived from CV and CNS test conditions will yield different factor of safety.

Table 4. Direct shear strength envelopes for cemented and uncemented mixture of granular backfills and EPS beads

	CNS	***CV***
Fine Sand	$\tau' = 0.5334\sigma_n' + 8.517$; $R^2 = 0.9984$	$\tau = 0.3471\sigma_n + 38.592$; $R^2 = 0.9814$
Fine sand+ EPS	$\tau' = 0.321\sigma_n' + 4.304$; $R^2 = 0.9916$	$\tau = 0.1818\sigma_n + 10.433$; $R^2 = 0.9966$
Cemented fine sand + EPS	$\tau' = 0.34\sigma_n' + 18.08$; $R^2 = 0.9977$	$\tau = 0.22\sigma_n + 18.12$; $R^2 = 0.961$
Coarse sand + EPS	$\tau = 0.33\sigma_n' + 6.7$; $R^2 = 0.981$	$\tau = 0.25\sigma_n + 3.3$; $R^2 = 0.961$
Cemented coarse sand+EPS	$\tau = 0.41\sigma_n' + 29.34$; $R^2 = 0.985$	$\tau = 0.29\sigma_n + 12$; $R^2 = 0.955$
Gravelly sand+EPS	$\tau = 0.40\sigma_n' + 13$; $R^2 = 0.985$	$\tau = 0.34\sigma_n + 2.7$; $R^2 = 0.966$
Cemented gravelly sand+ EPS	$\tau = 0.50\sigma_n' + 18$; $R^2 = 0.966$	$\tau = 0.40\sigma_n + 13.8$; $R^2 = 0.958$
Sandy gravel+EPS	$\tau = 0.43\sigma_n' + 8.9$; $R^2 = 0.966$	$\tau = 0.33\sigma_n + 0.89$; $R^2 = 0.975$
Cemented sandy gravel+EPS	$\tau = 0.52\sigma_n' + 14$; $R^2 = 0.985$	$\tau = 0.46\sigma_n + 9.9$; $R^2 = 0.989$

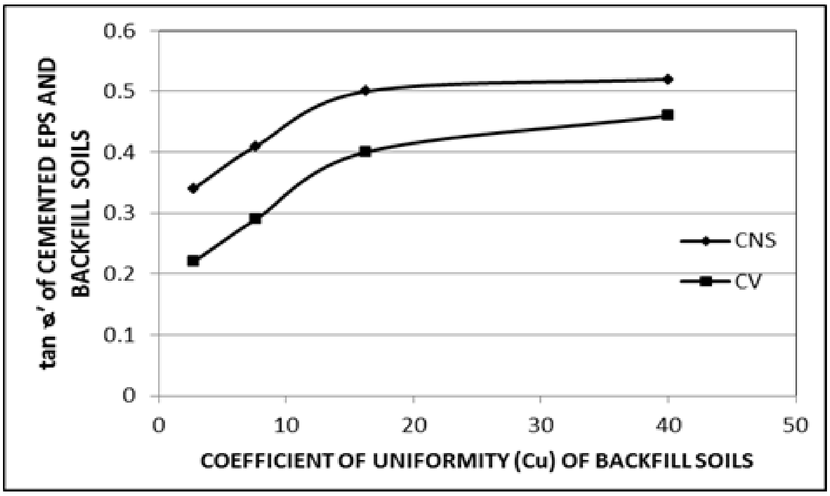


Fig. 17. Friction coefficient of light weight composites versus coefficient of curvature of the granular backfill

The depth of embedment, wall thickness, volume of steel reinforcement and spacing of counterforts are related to the horizontal stresses imposed by the backfill. The density of the cemented light weight backfill yield low horizontal stresses and reduced cost of retaining wall.

Mejeoumov et al. (2010) developed a chart relating different classes of road embankment materials to mobilized friction angle. For a broad application of the chart, direct shear parameters derived from CV and CNS test conditions can be used to accommodate the effect of drainage conditions on classification of road embankment materials.

4. Conclusions

The direct shear parameters of cemented mixture of granular backfills and EPS as well as the effect of the gradation properties of four backfills on the direct shear parameters was investigated under constant normal stress and constant volume conditions.

The maximum density and optimum moisture content of the backfill materials ranged from 1850 kg/m^3 to 1730 kg/m^3 and 9% to 13% respectively. The addition of 1.5% of EPS resulted in maximum density and optimum moisture content of the blended specimen of backfills soils and EPS that varied from 1303 kg/m^3 to 1368kg/m^3 and 7% to 9%.

For the range of normal stress used, the results reveal that the CNS friction angles of the four light weights composite were greater the CV friction angles. The CV friction angles reflect the undrained condition to some degree with reduced interparticle contacts.

For the CNS test conditions, both the consolidation strain and the shear induced vertical displacement increased with applied normal stress. The large values of shear induced vertical displacement indicated by speciemens subject to normal stress of 200 kPa and 400 kPa was due to consolidation induced remoulding of the speciemens. Thus 3% cement stabilized lightweight embankment subject to live loads greater than 100 kPa have a high likelihood to exhibit significant consolidation settlement and slope instability.

The relationship between friction coefficients of the cemented light weight composites derived from CNS and CV test data and the uniformity coefficient of the granular backfill was non linear, the rate of increase in friction coefficients was maximum for Cu between 2.73 and 16.25. For coarse fractions or gravel content (< 4.75 mm) greater than 20 %, the increase in mobilized strength decreases and gradually tend to a constant value. Thus for both the CNS and CV tests, the shear strength of the light weight granular backfill was dependent on the ratio of the sand to the gravel fractions.

The direct shear parameters of the lightweight materials can be used for the design of embankment with low bearing pressure and thus reduced settlement as well as low horizontal stress on retaining walls.

References

Abdelrahman G.E., (2010), Lightweight fill using sand, polystyrene beads and cement, *Ground Improvement*, **12**, 95-100.

Airey D.W., Budhu M., Wood D.M., (1988), *Some Aspect of the Behaviour of Soils in Simple Shear*, Proceedings of the 5th Australian and New Zealand Geomechanics Conference, Sydney, 18-39.

Bjerrum L., Landva A., (1966), Direct simple shear tests on a Norwegian quick clay, *Geotechnique*, **16**, 1-20.

Brink A.B.A, Partridge T.C., Williams A.A.B., (1982), *Soil Survey for Engineering*, Clarendon Press, Oxford, UK.

Brink A.B.A., (1984), *Engineering Geology of Southern Africa*, vol. 1, Building Publications, Pretoria.

Coduto D.P., (1999), *Geotechnical Engineering: Principles and Practices*, 1st Edition, Prentice Hall, Upper Saddle River, New Jersey.

Das B.M., Sobhan K., (2013), *Principles of Geotechnical Engineering* , Cengage Learning, 8th Edition, Australia.

Deng A., Yang X., (2010), Measuring and modeling proportion-dependent stress-strain, *International Journal of Geomechanics*, **10**, 214-222.

Ertugral O., Trandafir A., (2011), Reduction of lateral earth forces acting on rigid non-yielding retaining wall by EPS inclusion, *Journal of Materials in Civil Engineering*, **23**, 1711-1718.

Escario V., Saez J., (1986), The shear strength of partly saturated soils, *Geotechnique*, **36**, 453-456.

Esveld C., Markine V., (1993), Use of expanded polystyrene (EPS) sub-base in railway track design, On line at: http://www.esveld.com/download/tud/ant186_markine_esveld.pdf.

Geofoam Research Centre, (2000), Use of geofoam for reconstruction of I-15 in Salt Lake City, UT, Geofoam Research Centre, On line at: http://www.geofoam.syr.edu/GRC_15.asp.

Ghazavi M., (2004), Shear strength characteristics of sand-mixed with granular rubber, *Geotechnical and Geological Engineering*, **22**, 401-416.

Hamidi A., Alizadeh M., Soleimani S.M., (2009), Effect of particle crushing on shear strength and dilation, *International Journal of Civil Engineering*. **7**, 24-32.

Holts R.D., Kovacs W.D., (1981), *Introduction to Geotechnical Engineering*, New Jersey, Prentice Hall Inc. 234.

Hoek E., Bray J.W., (1999), *Rock Slope Engineering*, The Institution of Mining and Metallurgy, 7th Edition, London.

Kjellman W., (1951), Testing the shear strength of clays in Sweden, *Geotechnique*, **2**, 225-232.

Knight K., (1961), *The collapse structure of sandy sub-soils on wetting*, PhD thesis, University of Witwatersrand, South Africa.

Liu H.-l., Deng A., Chu J., (2006), Effect of different mixing ratios of polystyrene pre-puff beads and cement on the mechanical behaviour of lightweight fill, *Geotextiles and Geomembranes*, **24**, 331-338.

Mejeoumov G.G., Shon C.-S., Saylak D., Estakhri C., (2010), Beneficiation of stockpiled fluidized bed coal ash in road base course construction, *Construction and Building Materials*, **24**, 2072-2078.

Onishi K., Tsukamoto Y., Saito R., Chiyoda T., (2010), Strength and small-strain modulus of lightweight materials, *Geosynthetics International*, **6**, 380-388.

Ossa A., Romo M.P., (2009), Micro- and macro-mechanical study of compressive behavior of expanded polystyrene geofoam, *Geosynthetics International*, **5**, 327-338.

Xenaki V.E., Athanasopoulos G.A., (2001), The experimental investigation of interaction mechanism at the EPS Geofoam-Sand interface by direct shear testing, *Geosynthetics International*, **8**, 471-499.

Tsuchida T., Poorbaha A., Yamane N., (2001), Development of a geomaterial from dredged bay mud, *Journal of Materials in Civil Engineering*, **13**, 152-160.