Clay Reinforcement Using Geogrid Embedded In Thin Layers of Sand

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Abstract: *Large size direct shear tests (i.e.300 x 300mm) were conducted to investigate the interaction between clay reinforced with geogrids embedded in thin layers of sand. Test results for the clay, sand, clay-sand, clay-geogrid, sand-geogrid and clay-sand-geogrid are discussed. Thin layers of sand including 4, 6, 8, 10, 12 and 14mm were used to increase the interaction between the clay and the geogrids. Effects of sand layer thickness, normal pressure and transverse geogrid members were studied. All tests were conducted on saturated clay under unconsolidated-undrained (UU) conditions. Test results indicate that provision of thin layers of high strength sand on both sides of the geogrid is very effective in improving the strength and deformation behaviour of reinforced clay under UU loading conditions. Using geogrids embedded in thin layers of sand not only can improve performance of clay backfills but also it can provide drainage paths preventing pore water pressure generations. For the soil, geogrid and the normal pressures used, an optimum sand layer thickness of 10mm was determined which proved to be independent of the magnitude of the normal pressure used. Effect of sand layers combined with the geogrid reinforcement increased with increase in normal pressures. The improvement was more pronounced at higher normal pressures. Total shear resistance provided by the geogrids with transverse members removed was approximately 10% lower than shear resistance of geogrids with transverse members.*

Keywords: *geogrid, reinforcement, clay, sand, interaction, direct shear test*

1. Introduction

Soil reinforcement is a highly attractive alternative for embankment and retaining wall projects because of the economic benefits it offers in relation to conventional retaining structures. The rapid acceptance of soil reinforcement can be attributed to a number of factors, including low cost, aesthetics, reliability, simple construction techniques, and the ability to adapt to different site conditions. However, these economic benefits have often been limited by the availability of good-quality granular material. These materials have been the preferred backfill material due to their high strength and ability to prevent development of pore water pressure [1].

Build up of pore water pressure, lower frictional strength and compactibility as well as higher post-construction creep potential are the main concerns expressed about the use of cohesive soils in soil reinforcement [2]. These concerns may represent unrealistic restrictions in actual practice, where many highway embankments are constructed of compacted clays. One potential solution for reinforcing marginal soils is the use of permeable geosynthetics that function not only as reinforcement but also as lateral drains (Zornberg and Mitchell [3]). This would eliminate the need for expensive backfill and reduce transportation and structural costs as well as improving performance of compacted clay. Undoubtedly, substantial cost savings and new soil reinforcement applications would result if cohesive soil as well as industrial and mine wastes that would otherwise require disposal could be used in reinforced soil construction.

Interestingly, the first geotextile-reinforced wall constructed in 1971 by the French Highway Administration in Rouen, used poorly draining cohesive soil as backfill material. The purpose of this structure was to test its stability and to verify the magnitude of deformations caused by soil-geotextile interaction (Puig et al. [4]). Although

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marginal soils have been successfully reinforced, failures have also occurred mainly because pore water pressure generations were not correctly addressed during design.

2. Literature review

Reinforced soil derives its superior behaviour due to the stress transfer from the soil to the reinforcement at the interface. Adequate soil-reinforcement interaction has to be ensured to enable such a mechanism to take place. In the case of clay soil, the interfacial strength is low resulting in an early failure of the interface before the full strength of reinforcement can be mobilised. Thus the strength of reinforcement may be largely underutilized due to failure of the interface. A number of experimental studies using triaxial tests have been conducted to develop an understanding of the interaction between cohesive soil and different reinforcement systems. Results of drained and undrained compression tests on normally consolidated clay samples reinforced with several disks cut out of aluminum foil or porous plastics were presented by Ingold and Miller [5] and Ingold [6]. Results showed reductions in undrained axisymetric compressive strength of more than 50% relative to unreinforced samples. The premature failure of the specimen was attributed to pore-water pressures induced in the reinforced specimen which greatly exceeded those measured in a similar unreinforced specimen. Decreasing the spacing between the horizontal layers of reinforcement resulted in an increase in both the drained shear strength and the secant modulus of the reinforced sample. Based on the radiographic investigation, the strength enhancement was attributed, as in the case of sand reinforcement, to radial strain control arising from shear stress mobilized on the soil-reinforcement interface.

Fabian and Fourie [7] presented the results of undrained triaxial tests performed on silty clay samples reinforced with various geosynthetics having different in-plane transmissivities, including woven geotextiles, nonwovens, and geogrids. Their results showed that reinforcements with high transmissivity can increase the undrained strength of the clay by up to 40%, while reinforcements with low transmissivity can decrease the undrained strength by a similar magnitude. The use of non-woven geotextiles for reinforcing a near-saturated silty clay was also evaluated by Ling and Tatsuoka [8] using a plane strain device. The reinforcement effect, in terms of strength and stiffness, was reported to be greater in drained compared to undrained tests. At small strain levels, excess pore water pressures adversely affected the stress-strain response of the reinforced soil samples tested under undrained conditions. In the drained tests, tensile stresses were mobilized in the geotextile ensuring a positive reinforcement effect.

Shear failure at the interface may happen due to the high shear stresses near the reinforcement as seen in experimental observations by Jewell and Wroth [9], Milligan et al. [10] and Sridharan et al. [11. They have found that the shear stresses are highest around the reinforcement and decrease rapidly away from the reinforcement. Hence, when poor quality backfill is used for construction, it is advantageous to place thin layers of high-strength granular soil around the reinforcement to resist these high shear stresses near the interface. This method of construction called "sandwich technique" will improve the stress transfer mechanism because of the interface properties. Sridharan et al. [11] also reported significant improvement in the pullout capacity of geogrids embedded in weak soils because of sandwich layers. Based on laboratory tests on model retaining walls employing sandwich layers, Sreekantiah and Unnikrishnan [12] also reported improvement in the response of retaining walls. Unnikrishnan et al. [13] by conducting UU triaxial compression tests, reported improvement in strength and deformation behaviour of reinforced clay soils under static and cyclic loading.

A promising approach for design of reinforced marginal soils is to promote lateral drainage in combination with soil reinforcement. This maybe achieved by using geocomposites with in-plane drainage capabilities or thin layers of granular soil in combination with the geo-synthetic reinforcements. This design approach may even lead to the elimination of external drainage requirements. The potential use of permeable inclusions to reinforce poorly draining soils is well documented by Zornberg and Mitchell [3] and Tatsuoka and Yamauchi [14]. Although there is already strong experimental evidence that permeable inclusions can effectively reinforce poorly draining backfills, but there is no general design methodology for reinforced soil structures built with cohesive soils.

3. Scope of Research

This paper investigates the effects of embedding reinforcements in thin layers of granular material within a clay soil (i.e. sandwich technique) using large direct shear tests. A large number of tests were done by varying thickness of granular layer, magnitude of normal pressures and transverse members of the geogrids removed.

4. Testing program

Laboratory tests were conducted using 300 x 300 mm direct shear tests on samples of clay, sand, clay-sand and clay-sand-geogrids. A single horizontal layer of geogrid was used as reinforcement with variable sand layer thickness. Sand layer thicknesses of 2, 3, 4, 5, 6 and 7 mm on either side of the reinforcement were used to investigate its effects on the peak shear stress. Normal stress combinations of 25, 50 and 75 kN/m^2 were also used to investigate the effect of confining pressure for reinforced and unreinforced samples for a given sand layer thickness. Samples were tested under unconsolidated-undrained (UU) condition to simulate the behaviour of clays subjected to quick loading immediately after construction. Clay soils will have the least shear strength under this type of loading and hence it has been used to examine the influence of sand and sand-geogrid layers on its strength and performance. A constant horizontal displacement rate of 1mm/min was used throughout the tests as recommended by ASTM D:5321 and in order to be consistent with previous investigations. The geogrid specimens were large enough to completely cover the apparatus plan area. Complementary direct shear tests were also conducted with the transverse members of the geogrids removed in order to evaluate their contribution to the overall shear resistance.

5. Materials Used

5.1 Soils

Kaolinite was used as the clay soil and Firozkoh sand used for casting was selected as the granular material. The index properties of the clay and sand were determined according to the appropriate ASTM standards and are summarized in Table1. According to Unified Soil Classification System (USCS) the clay was classified as CL (inorganic clay of low plasticity) and the sand as SW (well

Table 1. Clay and sand characteristics

Soil	Description / Property	ASTM Standard	Value
Clay	Liquid limit	ASTM D4318	53%
	Plastic limit	ASTM D4318	33%
	Plasticity index	ASTM D4318	20%
	Optimum moisture content (Proctor compaction)	ASTM D698	23%
	Maximum dry density (Proctor compaction)	ASTM D698	1550(kg/m^3)
	Cohesion (at optimum moisture content)	ASTM D3080	23.2(kN/m^2)
	Angle of friction (at optimum moisture content)	ASTM D3080	10°
Sand	D_{10}	ASTM D2826	0.4
	D_{30}	"	1.3
	D_{60}	"	2.5
	Uniformity coefficient (c_u)	"	6.25
	Coefficient of curvature (c_c)	"	1.69
	Optimum moisture content (Proctor compaction)	ASTM D698	4%
	Maximum dry density (Proctor compaction)	ASTM D698	1600(kg/m^3)
	Angle of friction (at optimum moisture content)	ASTM D3080	33.7°

Table 2. Interfacial properties of the geogrid and its characteristics

Geogrid	Interfacial ϕ with sand (Deg.)	Interfacial ϕ with clay (Deg.)	Interfacial C with clay (kN/m^2)
Miragrid 50/25-30	36.1	7.9	28.8

Description	Symbol/Value
Raw material	PET
Coating	PVC
Ultimate longitudinal tensile strength (T_{ult})	50 (kN/m)
Ultimate lateral tensile strength (T_{ult})	25 (kN/m)
Longitudinal strain at T_{ult}	11%
Lateral strain at T_{ult}	13%
Ratio of reinforcement shear relative to total shear area (α_{ds})	10%

graded sand).

5.2 Geogrid

Miragrid 50/25-30 geogrid, a uniaxial polymer normally used for reinforced soil walls and steep slopes, was employed as reinforcement. Geogrid specimens 300x300mm covering the whole plan area of the shear box were used. The interface shear strength properties obtained by conducting modified direct shear tests together with the geogrid characteristics provided by the producer are listed in Table 2.

6. Sample preparation

Samples were prepared by static compaction of soil to a predetermined dry density and moisture content. Accurately measured quantities of dry powdered clay and water corresponding to maximum dry density (MDD) and optimum moisture content (OMC) were thoroughly mixed and kept in plastic containers for 24 hours for uniform moisture distribution. Initially the lower half of the shear box was filled with three equal layers of clay and lightly tamped with the specially adopted tamping device. Subsequently geogrid specimen covering the whole surface of the sample was horizontally laid and clamped to the inner face of the shear box. Then the upper half of the shear box was placed, secured and filled with moist soil in the same manner. After imposing the desired normal pressure and setting up gauges for measuring vertical and horizontal displacements as well as the shear force, testing commenced.

In the case of geogrid embedded in sand layers, after placing and compacting clay in three equal layers, predetermined amounts of moist sand were weighted, poured, spread and compacted to fill the lower half of the shear box. The geogrid was then laid and the top half of the shear box positioned and tightly secured. Subsequently the same amount of moist sand was poured, spread and compacted and the remaining volume of the top shear box filled with three equal layers of moist clay. Test set up including the position of the sand layer and the geogrid is schematically shown in Figure 1.

After completion of each test, samples were taken for density and moisture content determination. A maximum variation of 3% was observed which was considered acceptable. The procedures for specimen preparation and testing were standardized to achieve repeatability in the

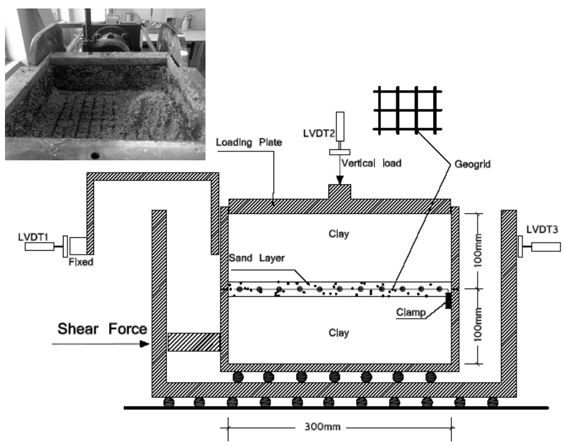

Fig.1. Cross – section of the shear box showing the position of sand layer and the reinforcement

test results. All the initial tests were repeated until consistent results were obtained.

7. Results and discussions

7.1. Reinforced and unreinforced clay

Results of direct shear tests conducted on samples of reinforced and unreinforced clay are shown in Figure 2. Variations of shear stress versus shear displacement for both the reinforced and unreinforced clay show an increasing trend by increase in normal pressures. Slope of the curves is significant at the early ages of shearing and it reduces by further shear displacement. Although reinforced clay samples consistently showed slightly higher shear stresses compared to unreinforced samples subjected to the same normal pressure, but the increase was not significant. This meant that failure occurred in the clay by way of full mobilization of cohesive strength. This behaviour is an indication that the clay-geogrid interface resistance is low which results in premature failure of the interface before the full strength of the reinforcement can be mobilised.

Thus, the strength of reinforcement may be largely underutilized due to the failure of the interface. Another possible reason for such behaviour can be the mesh size of the geogrid in comparison with the clay particle size. The geogrid can restrain particle movement and therefore increase the mobilised frictional resistance at particle contact points. Bergado et al. [15] investigating the interaction between cohesive-frictional soil and various grid reinforcements concluded that owing to the influence of the apertures on the grid reinforcements, the shear resistance between the grids and the soil in a direct shear test can be equal to or larger than the shear resistance between soil and soil. Touahamia et al. [16] investigating the shear strength of reinforced-recycled material also reported that overall restraint provided by the geogrid is determined by the particle size and particle grading.

The shear failure at the interface may happen due to high shear stresses developed near the reinforcement as observed experimentally by Jewell and Wroth [9], Milligan et al. [10] and Sridharan et al. [11].They have found that the shear stresses are the highest around the reinforcement and decrease rapidly away from the reinforcement. Hence, when poor quality backfill is used for construction, it is advantageous to place thin layers of high strength granular soil around the reinforcement to resist these high shear stresses near the interface. This will probably improve the stress transfer mechanism because of the better interface properties. Alfaro et al. [17] also showed the mobilization of direct shear resistance to be away from the interface into the soil and that mobilized shear strain in the direct shear test to be very uniform along the soil-geogrid interface.

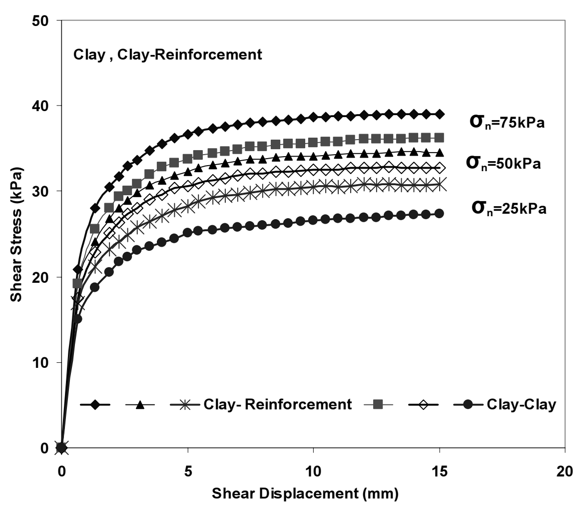

Fig. 2. Shear stress versus shear displacement curves for reinforced and unreinforced clay

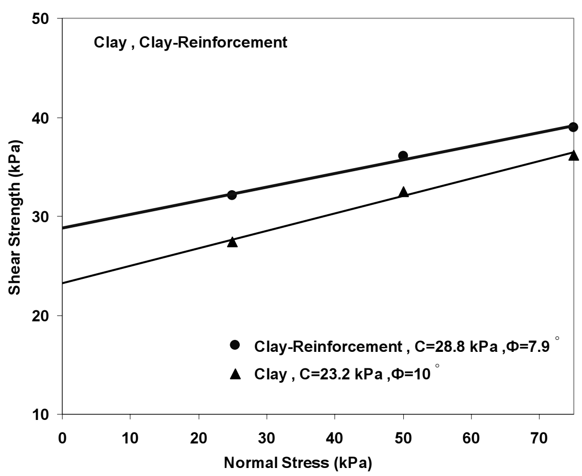

Fig. 3. Failure envelopes for clay and clay – reinforcement samples

Figure 3 shows the failure envelopes for the reinforced and the unreinforced clay samples. It can be observed that geogrid resulted in slightly reducing the angle of friction of the clay and increasing its cohesion. The overall effect of geogrid inclusion has been to slightly increase the shear strength of the clay. The linear envelopes are an indication of the absence of particle interlocking and subsequently no dilation was observed. As the shear resistance from the clay-geogrid test is close to that of the clay test, the results seem to agree with the suggestion of Jewell et al. [18]. They suggested that for a sandy gravel type of backfill and grid reinforcement, the direct sliding mechanism will be such that the rupture zone is forced away from the interface into the soil. In this case, the direct shear resistance of the soil-reinforcement interface would be equal to the full shear resistance of the soil.

7.2. Reinforced and unreinforced sand

Figure 4 shows the results of direct shear tests conducted on reinforced and unreinforced sand. It can clearly be seen that the curves do not display an obvious peak and by increasing the normal pressures the shear strength of both the reinforced and unreinforced samples increase. The shear strength of all samples initially increased significantly with only a limited shear displacement. After reaching their maximum shear strength samples exhibit post-peak displacement-softening behaviour and finally reach a steady state. The display of hardening and then softening behaviour is attributed to the amount of particle rearrangements that can occur.

At low normal pressure (i.e. 25kN/m^2), the reinforced and the unreinforced samples show approximately the same maximum shear and ultimate strength. By increasing normal pressure to 75kN/m^2, the reinforced samples show higher maximum shear and ultimate strengths. These changes are because the particles on the interface surface are less likely to be rearranged during shearing if the shear stress is not large enough to overcome the internal friction. The stress-hardening behaviour, especially at high normal stress, may result from the plowing of angular particles into geogrid material surface also reported by Han [19]. No dilatancy as such was observed, which confirms that the stress induced during shearing at the interface is not large enough to disturb the whole specimen. Under these conditions, sliding dominates the shear resistance as shown by Dove [20]. Results also show that horizontal displacement at failure is enhanced with an increase in normal pressure (i.e. confining pressure) in a way that it is 1-2 mm for 25kN/m^2 and 4-5 mm at 75kN/m^2. These changes have also been reported by Haeri et al. [21] from their investigation into the effect of geotextile reinforcement on the mechanical behaviour of sand.

The peak shear stress – normal stress

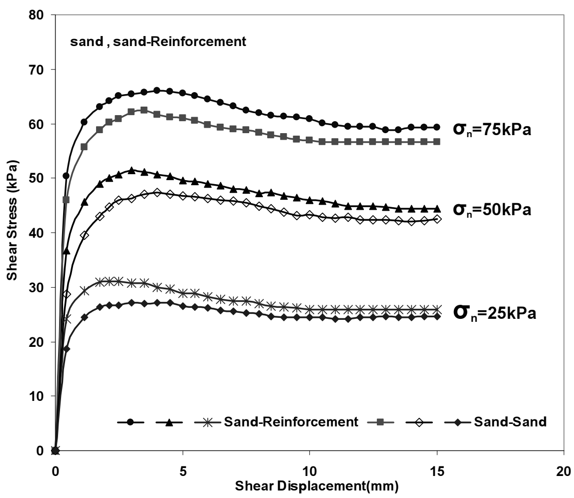

Fig. 4. Shear stress versus shear displacement curves for reinforced and unreinforced sand

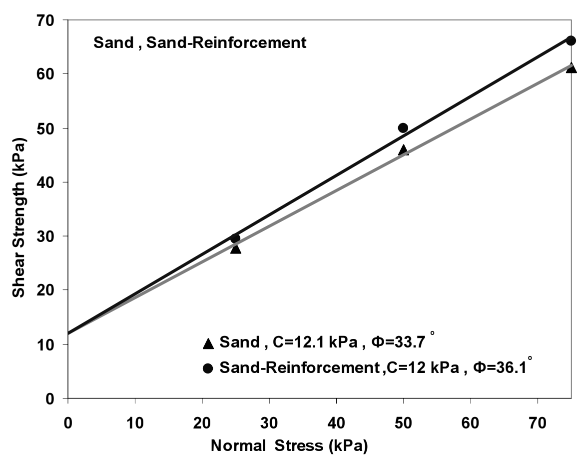

Fig. 5. Failure envelopes for sand and sand – reinforcement samples

envelopes for the reinforced as well as the unreinforced sand are shown in Figure 5. Failure envelopes for the peak shear stresses are linear which indicates the absence or very little dilatancy. The small adhesion intercepts obtained are attributed to the open apertures in the geogrids which permitted soil-to-soil contact and adhesion to exist because of suction in the soil. This cohesion is reflected as adhesion in the geogrid-soil interface, unlike other geosynthetics. Goodhue et al. [22] attributed the small adhesion observed to be caused by matric suction at soil-to-soil contact and machine friction. Suction causes an increase in adhesion at the interface between two porous materials but not at the interface between soil and geosynthetic. Athanasopoluos et al. [23] also reported the development of adhesion which they considered negligible for practical applications.

7.3. Clay reinforced with thin layers of sand

Changes in shear stress versus shear displacement for clay samples reinforced with thin sand layers of varying thickness are shown in Figure 6. It can be observed that the inclusion of sand layers significantly improves the shear strength of the clay soil. The improvement increases with increase in sand layer thickness. The shear stresses increase substantially during the early parts of the tests (i.e. 1-2mm shear displacement) with samples displaying a hardening behaviour. The increase in shear stress then continues gradually until it reaches a maximum at a shear displacement varying between 2 to 5mm depending on the sand layer thickness. It can be seen that only the clay sample reinforced with sand layer thickness of 14mm displays a distinctive maximum shear stress which reduces with further shear displacement. Other samples do not display a distinctive peak and their maximum shear stresses coincide with their ultimate shear stresses (i.e. steady state condition) displaying a plastic behaviour.

The points of maximum shear stress displayed by the samples seem to shift to the left by increasing the sand layer thickness and are reached at smaller shear displacements. For example, for the clay soil reinforced with 4mm and 14mm sand layers, shear displacements corresponding to the maximum shear stresses are 5mm and 3mm respectively. Examination of specimens with sand layers after the tests revealed that sand had penetrated a little into the clay and had established a good bond at the interface. As mentioned earlier, the shear stress in the soil reduces as the distance from reinforcement increases. This reduced shear stress at some distance from the reinforcement can be resisted easily by the sand-clay interface. The results are a clear indication of the effectiveness of including thin sand layers for improving clay soil performance. Inclusion of sand layers apart from improving the

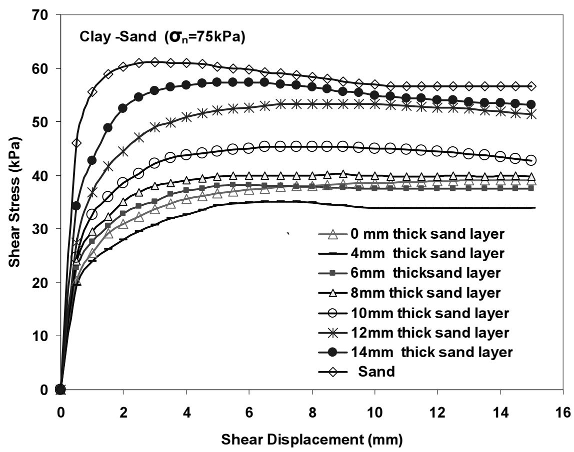

Fig. 6. Shear stress versus shear displacement curves for clay reinforced with thin layers of sand

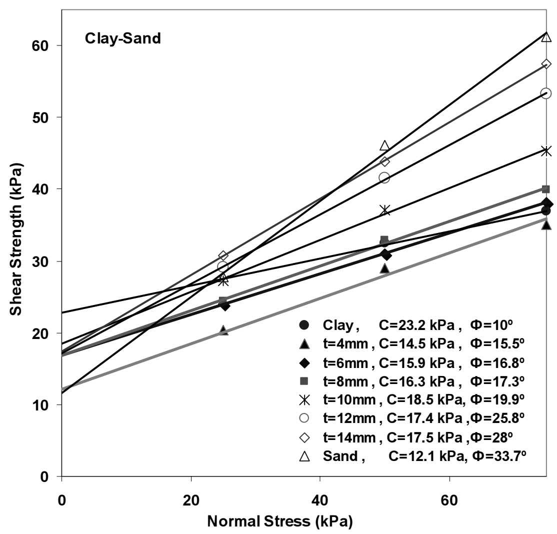

Fig. 7. Failure envelopes for clay, sand and clay – sand samples

performance of clay backfills, perhaps more importantly, can help drain and prevent pore water pressure build up.

Failure envelopes for the clay samples reinforced with different sand layer thickness are shown in Figure 7. Results clearly show the significant improvement in clay shear strength parameters by the inclusion of thin layers of sand. Taking clay shear strength parameters as the base, it can be observed the cohesion of the composites decrease and their angles of friction increase. For example, for the clay reinforced with 14mm sand layer, cohesion changes from 23.2 to 17.5 kPa, a reduction of 24.6%, and the angle of friction increases from 10 to 28 degree, showing an increase of 180%.

7.4. Clay–thin layers of sand-geogrid

The results of direct shear tests on samples of clay-sand-geogrid with different sand layer thicknesses using the whole plan area of (300x 300mm) the apparatus are shown on Figure 8. Results show that provision of sand layers around the geogrid reinforcement significantly improves the strength of the clay soil. The shear stresses initially increase sharply and become more gradual with further shear displacement. Clay samples reinforced with 4, 6 and 8mm sand layers around the geogrids do not display peak shear stresses and did not reach a steady state condition by the end of the tests. This behaviour indicated a progressive type of failure which can be useful in understanding the progressive global mobilization of direct-shear interaction resistance that will likely occur in actual cases.

By increasing sand layer thickness around the geogrid to 10, 12 and 14mm, gradually distinctive maximum and ultimate shear stresses can be observed. Unlike the clay samples which showed increase in shear stress with sand layer thickness (i.e. Figure 6), inclusion of geogrid increases the shear stress only up to 10mm sand layer thickness. Further increasing the sand layer thickness to 12 and 14mm resulted in lowering the maximum shear stress. The results clearly show that the full soil-reinforcement interface capacity has been mobilised even with thin layers of sand and further increase of sand layer thickness does not lead to improved performance of the composite. This means that there is an optimum sand layer thickness for achieving the maximum shear stress. After the tests all the geogrids remained intact which is mainly attributed to the fact that geogrid's modulus and strength is much higher than the surrounding soil. This is in accordance with observations of Unnikrishnan et al. [13] and Gray and Al-Refeai [24]. Unnikrishnan et al. [13] have also stated, the fact that none of the geogrids tested ruptured during the tests indicates that failure occurred mainly by pullout.

Shear stress versus normal stress envelopes for the clay-sand-geogrid composites are shown in Figure 9. It can be observed that embedding

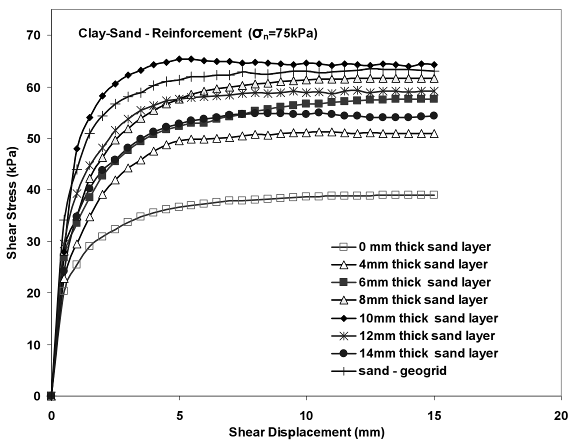

Fig. 8. Shear stress versus shear displacement curves for clay reinforced with thin layers of sand and reinforcement

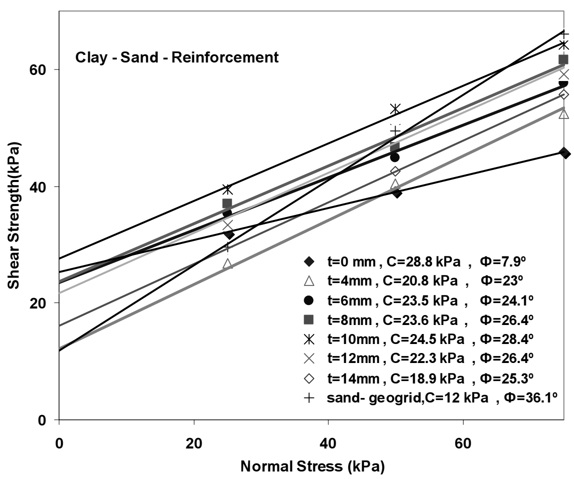

Fig. 9. Failure envelopes for clay – reinforcement, sand-reinforcement and Clay – sand - reinforcement samples

geogrid in thin layers of sand increases the shear resistance of the composite. The combined use of sand and geogrid as compared to using only sand for reinforcing clay resulted in mobilizing the maximum shear resistance at a smaller sand layer thickness (i.e. 10mm instead of 14mm). Compared to reinforced clay (i.e. C=28.8kPa, $\phi = 7.9^{\circ}$), the shear strength parameters of the clay-10mm sand-geogrid system changes to C=24.5kPa $\phi = 28.4^{\circ}$ and which are very close to the parameters produced by the clay-14mm sand system (i.e. C=17.5kPa, and $\phi = 28^{\circ}$). The combined effects of sand-geogrid in reinforcing clay has resulted in reducing cohesion by 15% and increasing the angle of friction by 259% which is a substantial improvement. This is a clear indication that for a particular soil, reinforcement, loading condition and normal pressure, an optimum sand layer thickness exists which mobilizes the maximum shear strength. The provision of thicker sand layers will not lead to further improvement in the performance of the system also reported by Unnikrishnan et al. [13] investigating the behaviour of reinforced clay under monotonic and cyclic loading.

7.5. Effect of normal pressure

To study the effects of normal pressures, results are presented as maximum shear stress versus thickness of sand layer in Figure 10. Results show that the combined effect of sand layers and geogrid reinforcement increases with increase in normal pressures. The change is more pronounced at higher normal pressures increasing at a faster rate. This behaviour is attributed to the greater confining effects provided by the geogrid. Results clearly show that increasing sand layer thickness up to 10mm increases maximum shear stress and further increase in sand layer thickness causes a reduction in maximum shear stress. The optimum sand layer thickness (i.e. 10mm) seems to be independent of the normal pressure used. These results are in contrast to the observations reported by Unnikrishnan et al. [13]. They reported that the relative increase in the additional confining stress induced by the woven geotextile reinforcement is higher at lower confining pressures. At higher normal pressures (i.e. 158kPa), beyond a sand layer thickness of 8mm, the maximum shear stress did not increase appreciably whereas at lower normal pressures the increase continued up to 15mm. They also concluded that the optimum thickness of sand layer depends on the operative range of stresses in the soil.

8. Bond strength

The bond coefficient between the soil and reinforcement is defined as the ratio of the resistance between soil and reinforcement to the resistance between soil and soil. For the soil/reinforcement direct – shear – interaction mechanism, the resistance between soil and soil is the direct shear resistance of the soil with the same shear area as that of the soil/reinforcement interface. Bond coefficient is the parameter that

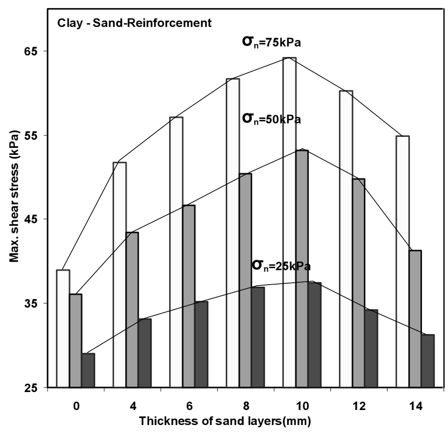

Fig. 10. Maximum shear stress versus thickness of sand layers for clay reinforced with thin layers of sand and reinforcement

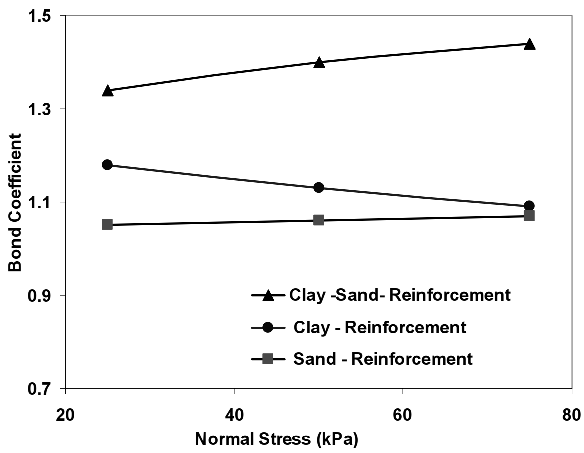

Fig. 11. Bond coefficient versus normal stress

expresses the efficiency of the grid reinforcement for providing shear resistance. Figure 11 shows the variation of the bond coefficient as function of normal stress for the clay, clay-sand and clay-sand-geogrid systems. The corresponding values vary between 1.0 and 1.4. It is indicated that the shear resistance between clay and the geogrid was higher than that of the clay soil. In a situation where the location of the shear surface is constrained to pass along the soil-reinforcement interface, this is possibly due to the influence of the apertures on the geogrid, which may provide some amount of bearing resistance during shear. However, it is difficult to measure the bearing effect of the apertures on the geogrid quantitatively. In real situations, the shear plane will pass through the plane with lowest resistance, so that the bond coefficient cannot exceed unity. This would indicate that the soil/geogrid reinforcement can provide the same shear strength as the soil itself. The range of bond coefficients determined in the present study is slightly wider than the range of 1.0 to 1.2 reported by Bergado et al. [15] whom investigated the interaction between cohesive-frictional soil and various grid reinforcements. This difference is attributed to the different grids used.

9. Effect of transverse members

To investigate the effects of passive resistance provided by the transverse members of the geogrid, several tests were conducted with these members removed. All the samples had the same number of longitudinal members as the geogrid with transverse members. A comparison of the total resistance of the reinforcement with and without transverse members was made and the results are shown in Figures 12 and 13. The shape of the shear stress – shear displacement curves for both sets of samples are similar. Results show that removal of the transverse members slightly reduces the shear resistance between the soil and the reinforcement. It was found that the direct shear resistance of the geogrid without transverse members was approximately 90% of the direct shear resistance of the geogrid with transverse members. Pullout tests conducted by Bergado et al. [15] on Tensar geogrids and bamboo grids with transverse members removed respectively showed pullout resistances equal to 90-100% and 80-90% of the grids with transverse members. They reported that the total resistance of the grids with and without transverse members is very close which they attributed to the small spacing between the longitudinal grid members and three dimensional effects.

10. Conclusions

A large number of 300x300mm direct shear tests were carried out to investigate the behaviour of reinforced clays with geogrids encapsulated in thin sand layers (i.e. sandwich technique). It was observed that using thin layers of sand to cover

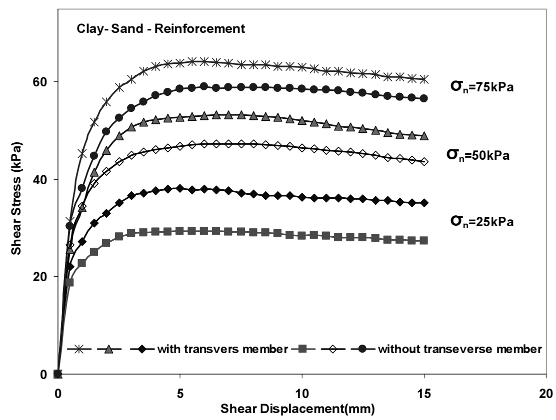

Fig. 12. Shear stress versus shear displacement curves for clay – sand – reinforcement with and without transverse members

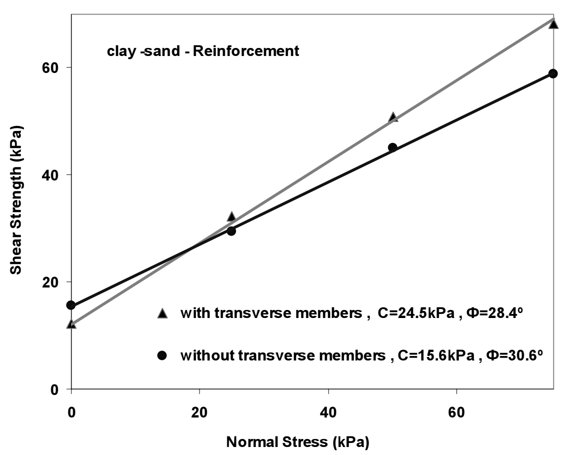

Fig. 13. Failure envelopes for clay – sand - reinforcement samples with and without transverse members

the geogrids significantly improves the response of clay soils through interfacial enhancement. The improvement is the result of more effective interlocking of sand within the geogrid openings. Owing to the influence of the apertures on the geogrid, the shear resistance between the geogrid and the soil can be equal to or larger than the shear resistance of the soil itself.

For the soil, geogrid and normal pressures used, an optimum sand layer thickness was determined which resulted in most improvement. The provision of thicker sand layers did not lead to further improvement in the behaviour of the composite system. Combined effects of sand layers and geogrid reinforcement increased with increase in normal pressures. The change was more pronounced at higher normal pressures. By increasing sand layer thickness up to 10mm, maximum shear stresses displayed by samples increased and thicker sand layers caused reduction in maximum shear stresses. The optimum sand layer thickness (i.e. 10mm) seemed to be independent of the normal pressure used.

Shear resistance provided by the geogrids without the transverse members was determined to be approximately 10% less than the shear resistance of geogrids with the transverse members.

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