# Investigation of the soil-geosynthetics interface behaviour through a modified direct shear apparatus

Minno, M.
*Department of Hydraulics Transportation and Roads, University of Rome La Sapienza, Rome 00184, Italy*
Li, L.
*Department of Civil Engineering and Engineering Mechanics, Columbia University, New York City, NY 10027, United States*
Ling, Hoe I.
*Department of Civil Engineering and Engineering Mechanics, Columbia University, New York City, NY 10027, United States*
Napoleoni, Q.
*Department of Hydraulics Transportation and Roads, University of Rome La Sapienza, Rome 00184, Italy*

Keywords: direct shear; efficiency; friction coefficient; geosynthetics; shear band; cyclic loading

ABSTRACT: The strength of a soil-geosynthetics interface plays a leading role in a reinforced soil slope design and stability evaluation, where in particular the direct sliding failure mode is mostly influenced by the friction coefficient of the interface. This paper presents the results of several tests performed using a direct shear apparatus with a shear area of 0.30x0.30 m. Nevada sand and a mixture of Nevada sand and clay were tested in combination with a double twist PVC coated steel mesh and a PET geogrid under monotonic loadings. The efficiencies, in terms of peak and residual strengths, were evaluated and interesting considerations about the shear band formation were outlined. In particular the wire mesh did not seem to affect the friction angle, while the PET geogrid reduced the efficiency up to 77%. Furthermore, cyclic loading tests were performed at different stress levels and then with the specimen loaded to failure: in the case of geogrid, efficiency was found to decrease as it did in monotonic loading tests.

## 1 INTRODUCTION

In reinforced soil structures design, the term “interface” refers to a mass zone having a certain thickness that includes both the contact surface between soil and reinforcement, where friction and interlocking effect occur, and the shear band, which represents the deformed soil mass adjacent to the reinforcement. The interaction mechanisms that occur along such interfaces are critical in stability evaluation and design of reinforced structures. Adequate safety margins must be ensured against pull-out and direct sliding failure modes.

The work as presented by the paper aimed at investigating the interface shear strength of two types of reinforcement (polymeric geogrid and steel wire mesh) embedded in two types of soil (sand and sand-clay mixture) through several direct shear tests performed in a large size apparatus under both monotonic and cyclic loading applications.

## 2 TESTING EQUIPMENT AND PROCEDURE

In order to perform such tests, a shear apparatus was fabricated at Columbia University of New York City (US). The shear box was made of two aluminium halves: the bottom half is fixed to the body of the machine, while the top half is free to move, sliding on smooth polymeric strips placed on the borders of the lower box. For the dimensions of the box, indications provided by the ASTM D 5321 were followed and a 0.30x0.30 m squared box was made (the height was 0.05 m on each half of the box).

The shear force was applied through a screw jack, while the normal pressure was applied by a lever system, where an iron bar transfers the load of slotted weights hanged on at a set distance to a thick plate laid on the upper half of the box. The friction during sliding is reduced and the residual friction is evaluated through different tests without soil specimen. The device included also a load cell to measure the shear force, two LVDTs to measure the horizontal and vertical displacements, a data acquisition system and a control system. The apparatus was not set to saturate the specimen and to keep an undrained condition during shearing, thus only CD tests in dry condition were performed. Two different types of soil, Nevada sand and Nevada sand-clay mixture, were tested in order to investigate the behaviour of sand of well known physical properties (Arumoli et al., 1992), but also to see which differences could arise adding cohesive soil to a granular one. The soil mixture used for testing was made adding to Nevada sand an amount of Kaolinite in order to represent the 15% of the whole sample (maximum dry unit weight is 16.9 kN/m3, reached at 7.2% of moisture content).

Such sand-clay mixture has been used in another series of studies (Ling et al., 2009).

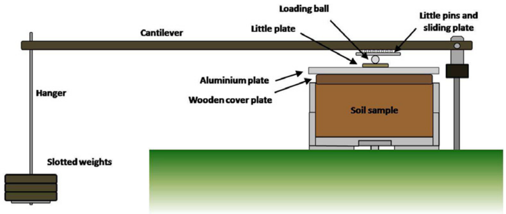


Figure 1 Sketch of the apparatus

Two reinforcing elements were tested. A double twist steel mesh (8x10 hexagonal mesh) coated by a thin PVC layer with a tensile strength of 380-550 N/mm$^2$ was used. Since it was not a bi-dimensional element, but few millimetres thick, the steel mesh sample was nailed to a wooden plate and put into the lower box at a selected height, in order to ensure that the shear plane passed over it and to guarantee the selected mesh position also at high normal pressures.

The geogrid was made in polyester (PET) and had a tensile strength of 55 kN/m in both directions. It was approximately planar and the whole sheet was fixed and set in order to keep the geogrid exactly along the gap of the boxes; by hanging a weight, a constant tensile force was applied to keep the reinforcement straight during the test to better simulate the field conditions.

## 3 TESTING SCHEDULE

The shear strengths of soil-soil and geosynthetics-soil interfaces were investigated by three tests at different normal pressures for each case. For Nevada sand the moisture content was around 5% and relative density around 60%; in sand-clay mixture the moisture content was 7.2% (the optimum determined from the results of the Proctor test) and a percentage of 95% (usually suggested for backfill compaction) of the maximum dry density representing the compaction grade. The displacement rate was 1mm/min as indicated by ASTM D5321. Furthermore the effects of cyclic loading on the shear strength of sand-clay mixture and geogrid-sand clay mixture interfaces were studied. The tests were strain-controlled, as the monotonic tests, but a displacement rate of 10mm/min was set to better simulate a dynamic loading, as suggested by ASTM D5321 for cyclic tests and in literature (O'Reilly and Brown, 1991).

To govern the cyclic testing, a stress-limit condition was used, setting the reference values as percentages of the monotonic shear peak stress, each one for a series of cycles, as described in Figure 2: thirty cycles with stress-limit of 40% of the monotonic peak stress, thirty cycles at 60%, thirty cycles at 80% and then full displacement (2.5 cm).

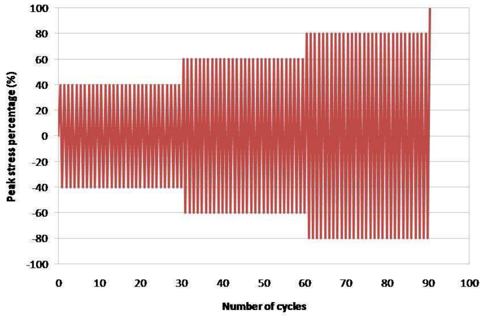


Figure 2 Cyclic loading scheme

## 4 RESULTS AND DISCUSSION

### 4.1 *Nevada sand tests*

Plotting the corrected peak shear stress (where the peak value was compensated for the device friction resistance) versus the relative normal stress as the failure envelopes, it came out that sand and sand reinforced with steel mesh have the same friction angle, 35°, while sand reinforced with a layer of PET geogrid showed a smaller value, 33° (Figure 3).

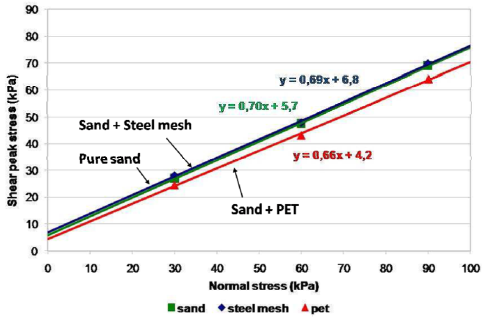


Figure 3. Failure envelopes for pure Nevada sand and reinforced Nevada sand (peak)

The steel mesh didn't seem to decrease the angle of friction, which implied that this interface is not a weaker plane for the soil mass. First of all, the steel mesh is made up of several thick PVC coated wires with double torsion nodes: the sand gets locked in the mesh openings (effect increased by the high relative density) and such 7mm reinforced layer creates a kind of barrier for the shear band, which has to develop entirely above the steel mesh (Williams & Houlihan, 1987). This happens also in most of the geogrids, but then a much smaller friction mobilizes on the geosynthetics surface and it indeed makes the reinforced plane a weaker plane. But a second factor is to be considered: the coverage ratio between the effective geosynthetics area and the whole section area, denoted in literature as $\alpha_{ds}$ (Jewell et al., 1984). It is only about 12% in the double torsion steel mesh and, even if the PVC coating is surely quite smooth, its presence along the shear surface is almost negligible and thus the shear resistance is almost totally given by the soil-soil friction. Figure 4 shows a picture of the shear surface, taken after having carefully removed the upper box and the soil inside: a well outlined, almost sharp-cut, shear surface occurred and a small coverage ratio is easily noticed.

Figure 4. Shear surface of steel mesh reinforced sand

In regard to residual shear stresses, sand and sand-steel mesh interface always behaved as one and a slight decreasing in friction angle occurred (34°). These results about the steel mesh interface lead to extremely good conclusions: the friction on the reinforcement surface was not smaller than the one of the sand itself and thus no different design parameters should be then introduced to approach the "direct sliding" failure.

In regard to the tests where the PET geogrid was used as reinforcement, different considerations were made. In fact the peak shear stress reached a smaller value than in pure sand under each normal pressure condition and the friction angle was two degrees smaller for peak strength and five degrees for residual strength. As described for the steel mesh, the geogrid didn't let the shear band expand below it because of its width, strength and axial tension. But the element had a coverage ratio of 30% and it was flat, so that the geosynthetics surface was really a weaker surface. By these tests, peak and residual efficiencies of 0.94 and 0.82 for the geogrid interface and 1.0 for the steel mesh interface were obtained. When this kind of PET geogrid is coupled with steel mesh elements, an efficiency smaller than 1.0 is suggested for the reinforced layer in design.

## 4.2 *Sand-clay tests*

Sand-clay mixture and sand-clay mixture reinforced with steel mesh had the same friction angle, 32°, while the PET geogrid-soil interface showed an angle of 28° (Figure 5). The clay addition to Nevada sand gives cohesion of few kPa to the specimen and a lower friction angle, as expected.

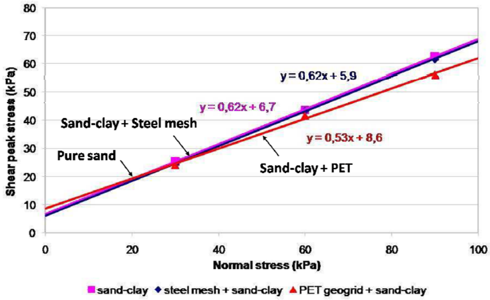


Figure 5. Failure envelopes for sand-clay and reinforced sand-clay

The efficiencies found on Nevada sand-clay mixture are 0.87 (peak) and 0.77 (residual) for geogrid interface and 1.0 for steel mesh interface. Similar considerations of the previous section can be made for both materials; the efficiency of the geogrid-sand-clay mixture interface was even smaller than the one found with sand, so more attention should be given during a reinforced soil slope design.

## 4.3 *Shear band*

To better understand how the shear band develops in a soil reinforced either with geogrids or steel mesh, the following test was performed. When the soil is placed in the box and compacted at a reference value, before setting up the bearing frame, seven vertical holes are made in the specimen along the middle line and filled up with blue coloured sand. These seven "columns", along the whole length of the box, will be partially deformed during the test (at 30 kPa as normal pressure) and their final shape will help to locate the borders of the shear band. In the unreinforced specimen, the shearing didn't deform the columns in a continuous way, but, besides a light bending, mostly through two sliding planes occurred at the top of the compaction layers, which represent weaker shear planes. In the geogrid reinforced specimen, the weakest plane didn't develop on the top of those layers, but along the geogrid surface: the sliding occurred over there, as if the two halves of the box were rigid blocks, with one sliding over the other. In the third test the soil occupied only the upper box, while the steel mesh was fixed to a wooden plate and positioned along the gap: the sand locked up between the thick wires of the mesh did not move and the surface over it became the sliding surface.

Figure 6. Shear bands in sand-clay mixture (left), sand-clay mixture with PET geogrid (middle) and sand-clay mixture with steel mesh (right)

## 4.4 *Cyclic loading*

The test results show that friction angle evaluated through cyclic loading was not lower than the monotonic friction angle for small displacement cycles (within the peak shear stress occurrence). This is consistent with observations given by Ling et al. (2008). Furthermore residual friction angle for PET geogrid-soil interface showed about the same amount of reduction from the peak value as in monotonic tests. The stress-displacement curve for cyclic loading at 30 kPa on a sand-clay mixture is represented in Figure 7. It is possible to see how the shear displacement increased with cycles, as a consequence of the decrease in shear modulus and of the plastic deformations (Fakharian & Evgin, 1997).

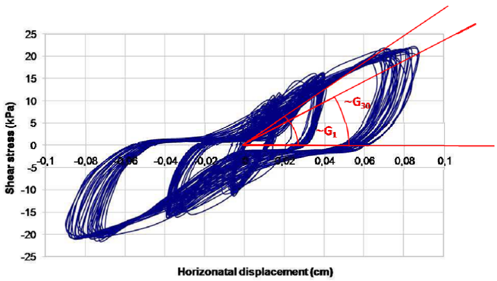


Figure 7. Cyclic loading on sand-clay mixture at 30kPa normal pressure

Moreover, this phenomenon is more noticeable at higher stress amplitudes and therefore higher strain amplitudes, confirming the non-linear behaviour of soil. In a direct shear test the shear strain is not definable, but it is still possible to have an idea of the decreasing rate of the modulus assuming a fictitious amplitude: looking at the last thirty cycles with stress-limit at 80% of the monotonic peak, a different slope between the first and the last cycle was detectable: the stiffness decreasing ratio was roughly around 25%. Comparing cycles plots from reinforced and unreinforced sample, it seems that PET geogrid-soil interface has a larger initial shear stiffness, which decreases with cyclic loading more than along the soil-soil interface, so that larger plastic deformations developed. Also in monotonic tests the peak stress along the soil-geogrid interface was reached in slightly smaller displacement amplitude, so the installation of a PET geogrid increased the shear stiffness, decreased the shear strength of the interface and made it more sensitive to a cyclic loading.

# 5 CONCLUSION

The use of steel wire mesh did not seem to affect the shear strength of the interface and thus same parameters from those of the backfill soil may be used in a reinforced soil slopes design. The direct sliding coefficients of PET geogrid-soil interfaces were determined for both peak and residual strengths; cyclic loading testing allowed to investigate the interface response to repeated loadings, showing that the shear modulus had slight lower values with cycles of small amplitude (within the peak shear stress occurrence), more noticeable than in pure soil. On the other hand, the friction angle and therefore the efficiency did not seem to decrease. In conclusion, the PET geogrid increased the shear stiffness, decreased the shear strength of the interface and made such a layer more sensitive to cyclic loading.

# REFERENCES


Arumoli, K., Muraleetharan, K. K., Hossain, M. M., and Fruth, L. S. 1992. VELACS: Verification of Liquefaction Analyses by Centrifuge Studies Laboratory Testing Program Soil Data Report. *Earth Technology Corporation*.

ASTM Standard D 5321-02: "Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method", *Annual Book of Standards*, Vol. 04.13, ASTM International, 2005.

Fakharian K. & Evgin E.. 1997. Cyclic simple-shear behaviour of sand-steel interfaces under constant normal stiffness condition. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123(12), pp. 1096-1105.

Ling H.I., Wu M.H., Leshchinsky D. and Leshchinsky B.. 2009. Centrifuge Modelling of slope instability. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135(6), pp. 758-767.

Ling H.I., Wang J-P. and Leshchinsky D.. 2008. Cyclic behavior of soil-structure interfaces in a reinforced soil wall: Experimental studies. *Geosynthetics International*, Vol. 15(1), pp. 14-21.

O'Reilly M.P.O. & Brown S.F. 1991. Cyclic Loading of Soils. *Blackie and Son Ltd*.

Jewell R.A., Milligan G.W.E, Sarsby R.W. and Dubois D.. 1984. Interaction between Soil and Geogrids. *Proc of Symposium on Polymer Grid Reinforcement*, pp.18-30.

Williams N.D. & Houlian M.F. 1987. Evaluation of interface friction properties between geosynthetics and soils. *Proc. of Geosynthetic '87 Conference, Industrial Fabrics Association International*, pp. 616-627.