Study of the Shear Strength of a Tropical Soil with Grass Roots

M.I. Miranda Neto, C.F. Mahler

Abstract. The role of roots in shear strength has been a matter of research and also uncertainties. An investigation was conducted to identify and quantify the contribution of the roots of vetiver grass (*Chrysopogon zizanioides*) to the shear strength of soils, by means of triaxial testing. Samples were prepared from 4-inch PVC pipe molds, where tropical soil was compacted and specimens of vetiver seedlings, obtained by tillering, were grown. After 24 months of growth, from each mold of the mature vetiver grass it was possible to get at least four samples with roots inside. Similar samples of the same soil without vetiver grass were submitted to triaxial tests to determine shear strength of soil alone. Confining pressure ranging from 25 to 200 kPa was used in triaxial extension drained tests to determine a Mohr-Coulomb envelope. The shear strength parameters of soil without roots, cohesion intercept and friction angle were respectively 13 kPa and 34°, and the Mohr-Coulomb envelope of the soil with vetiver roots showed a bilinear shape with cohesion and friction angle, respectively, of 17 kPa and 59° for confining pressure below 75 kPa and 22 kPa and 33° for confining pressure above 75 kPa. So an increase in shear strength was obtained, because the roots acted to reinforce the soil mass. Triaxial compression tests were conducted in the same soil with and without roots and no significant increase in resistance was observed. The result was observed due to a vertical spread of roots, since any reinforcement in the same direction of the compressive force does not contribute to increase the strength. In conclusion, for extensional stress above 75 kPa confining pressure, the friction angle was the same as that of the soil without roots, although the intercept of cohesion was larger. Below 75 kPa, the soil showed a very large apparent friction angle due to the roots. Therefore, vetiver roots increase the shear strength in soils under extensional loadings.

Keywords: soil reinforced, soil stabilization, vetiver grass, triaxial extension test, tropical soil strength parameters, bioengineering.

1. Introduction

Many contributions have been made in recent decades to improve knowledge about the behavior of soil reinforced with metal or synthetic or natural fibers such as roots, subjected to direct shear or triaxial compression tests. One of the pioneering studies in this respect was performed by Gray & Ohashi (1983). They concluded that the main role of fibers is to increase the soil shear strength, and that a confining pressure exists below which the fiber has a tendency of be pulled out of the soil. Later, Gray & Al-Refeai (1986) indicated that rougher fibers tend to be more effective in increasing the shear strength and Maher & Gray (1990) showed that bilinear shearing envelopes of reinforced soils have a breaking point, named the critical confining pressure, below which the reinforcement tends to be pulled out.

After Gray & Ohashi (1983), other researchers (Michalowski & Zhao, 1996; Zornberg, 2002; Michalowski & Cermák, 2003; Heineck & Consoli, 2004; Gao & Zhao, 2013) have carried out theoretical studies to develop predictive models of the improvement of shear strength due to the addition of fibers in the soil. Some researchers have focused on the behavior of the addition of discrete randomly distributed synthetic fibers (Freitag, 1986; Feuerharmel, 2000; Casagrande, 2001; Casagrande & Consoli, 2002; Heineck *et al.*, 2005; Casagrande, 2005; Consoli *et al.*, 2007; Sadek *et al.*, 2010; Palacios, 2012), fibers and cement (Consoli *et al.*, 1998), or natural fibers from vetiver roots (Focks, 2006; Barbosa, 2012).

This study is another contribution to knowledge of the behavior of reinforced soil. This article examines the behavior of a tropical soil with natural inclusion of vetiver grass roots subjected to extensional forces. Since the roots' preferential direction is vertical, it is expected that soil reinforced with predominantly vertical roots subjected to triaxial compression tests should behave differently than reinforced soil under triaxial extension tests. Therefore, given that the root system is predominantly vertical, the extension test would better simulate the role of the roots in the reinforcement of the soil than compression tests since they would not be subject to buckling. In geotechnics, the axial extension would be, for example, an unloading by excavation and the lateral extension would be from passive earth pressure by jack reaction or earthquake.

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The main objective is to evaluate the contribution of vetiver roots on the shear strength of a tropical soil, especially through triaxial extension tests.

The use of natural fibers such as plant roots as reinforcement inclusions in soil, particularly for slope stabilization, has been proposed as an effective bioengineering method. However, solutions involving plants usually face problems such as the range of the root system, the low growth rate of the plant until the root system reaches maturity, the susceptibility of the plants to damage such as fire, drought and vandalism, and also the introduction of alien species in the environment, all of which can discourage their use in engineering projects.

Vetiver grass is well known for its tolerance to aggressive environments and rapid growth rate. Also, its roots can reach several meters and it is resistant to disease. These traits have caused its planting to be recommended as an effective method to stabilize slopes (Truong, 2000).

The bioengineering solutions for reestablishing vegetation using grasses are quite effective in erosion control, but some doubts still exist as to slope protection against sliding. Knowledge is incomplete about how much increase in shear strength is provided by the introduction of roots in the soil. In practical terms, if the shear strength parameters of a slope reinforced with roots are properly known, the increase in the safety factor can be assessed more accurately.

2. Materials and Methods

This study involved triaxial strength tests on soils with and without vetiver roots. The insertion of the roots as reinforcement in the soil was done by growing grass in tubular molds with 98 mm inner diameter and 1.0 m length, previously filled with compacted soil. Tubes containing compacted soil without grass were used as controls. Each tube was prepared with 14 layers of 6 cm soil at 23% average moisture using 11 blows of a 2 kilogram hammer at 50 cm of drop distance. Therefore, to prevent root growth the compaction energy used in the preparation of samples was 2.3 kg.cm/cm^3, approximately half of the normal Proctor energy.

The tubular molds made it possible to extract up to four specimens to perform triaxial tests. These molds had a 5 cm layer of granular base, which functioned as a filter, buffered with a pierced end cap to allow drainage of water. The top of the mold had a free edge of about 5 cm to allow a suitable depth of the water level in irrigation of the crop. Figure 1 shows the molds with vetiver grass.

The plants were obtained by clump division. Two clumps of grass generated 20 seedlings, each one planted in a mold with compacted soil. The molded soil void ratio ranged from 0.68 to 0.79, dry unit weight ranged from 1.63 to 1.53 g/cm^3 and average density of solid constituents was 2.75 g/cm^3. This soil was collected in the alluvial fan resulting from the gully erosion into the mantle of tropical residual/colluvium soil formed by weathering from mica-schists and gneisses, constituting the geological setting from Meso/Neoproterozoic era named the Búzios complex in

Figure 1 - Vetiver grass grown in molds with age of 18 months (near the second flowering).

Rio de Janeiro, Brazil. The particle size distribution consisted of 10% gravel, 72% sand, 10% silt and 8% clay. The cation-exchange capacity was 4.45 cmol/kg, pH = 5.1 and Ki = 2.05 suggesting a tropical soil. In terms of mineralogy, the soil contains kaolinite, muscovite, quartz and weathered feldspar. The low amount of clay is related to the leaching processes in the alluvial fan.

The molds were watered weekly, including those without vetiver grass, and they were kept in a greenhouse until the stem of the plant exceeded one meter in height. Then the molds were left outdoors exposed to the weather, with continued weekly watering. The vetiver grass was planted in October 2011 and the first flowering occurred in July 2012, indicating that the plants had reached maturity. The first triaxial test was carried out when the specimens were a little over two years old, and were repeated from January to December 2014.

The specimens for triaxial tests were extracted from the base to top of the mold, keeping the plants alive for further withdrawal of samples. Figure 2 shows the sampling sequence. The tests were performed on the same day of sampling so that the roots had some vitality when tested. At the end of the test, the roots were exhumed and their general condition, mass and moisture were measured along with the amount of roots with diameter greater than 0.4 mm. The cross sectional root area was obtained by computing all roots greater than 0.4 mm in diameter, which accounted for more than 90% of the total root mass. The fiber area ratio was defined as the ratio of fiber total cross-sectional area and sample cross-sectional area.

The drained triaxial compression tests were performed on specimens measuring 98 mm in diameter and height of approximately 18 cm. The degree of saturation was checked by pore pressure parameter, B, corresponding to 0.95, obtained by backpressures of 600 kPa. For confining pressure of 25 kPa the maximum incremental back pressure was 25 kPa to avoid over consolidation during the process. For the remaining confining pressures (50, 75,

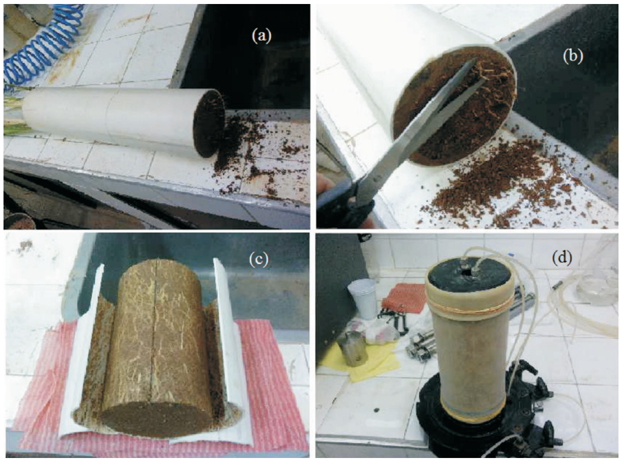

Figure 2 - The sampling: (a) cutting the sample; (b) trimming the roots left out; (c) removing the PVC layer; (d) assembling the sample in the triaxial cell.

100, 150 and 200 kPa), the counter pressure increments were 40 kPa.

The hydrostatic consolidation pressures applied were 25, 50, 75, 100, 150 or 200 kPa, depending on the stress level of the test. The shear phase consisted in applying axial loading maintaining pressure in the triaxial cell until rupture, normally characterized by no increment in deviator stress and in volume change with axial deformation. The velocity of loading was controlled by strain at a rate of 0.17%/min.

Attempts to run triaxial extension drained loading tests failed. In this test, after the hydrostatic consolidation, the loading is applied by increasing the confining pressure and maintaining the axial stress. This stress path aims to simulate the jack reaction of an earth support system. The back-pressures were high, about 600 kPa, to ensure high degree of saturation, and the long stress path was interrupted before rupture due to reaching the maximum pressure of the triaxial cell. For this reason, triaxial extension drained unloading tests in the shear phase were performed.

In the triaxial extension drained unloading test, the saturation and consolidation phase were similar to compression tests. The shear phase was run maintaining the confining pressure and reducing the axial load until failure. The sample stretched as a process of extension by excavation. The speed of unloading was initially controlled by the axial stress, at the ratio of -0.2 kPa/min until the strain rate reached -0.02%/min. From there on, the unloading control was by strain ratio of -0.02%/min until failure. The negative sign means axial discharge or extensional displacement. These lower rates in the triaxial extension unloading tests were obtained experimentally and prevented the top porous stone from detaching from the sample during the shear phase.

3. Results and Discussion

In this work, 31 triaxial extension drained unloading tests were carried out: 10 tests on samples without roots and 21 tests on samples with roots (two of the samples with roots were discarded after statistical analysis). Details of statistical treatment can be found in Miranda Neto (2015).

Also, 21 drained compression triaxial tests were performed: 12 tests on samples without roots and 9 on samples with roots. Figures 3 and 4 show respectively stress-strain curves for the extension tests on samples with roots and without roots.

In the triaxial extension tests, the major principal effective stress (σ'_1) is the effective confining stress (σ'_h), the axial strain by stretching (ε_a) is negative and the effective axial stress (σ'_v) is the minor principal effective stress (σ'_3).

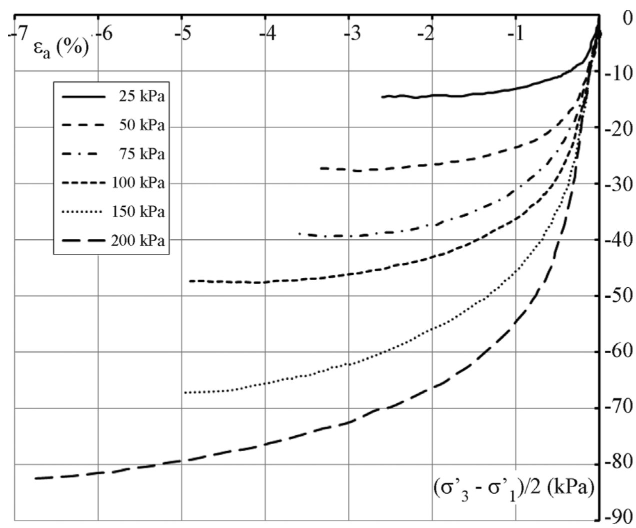

Figure 3 - Stress-strain curves of soil with roots for confining stress from 25 to 200 kPa.

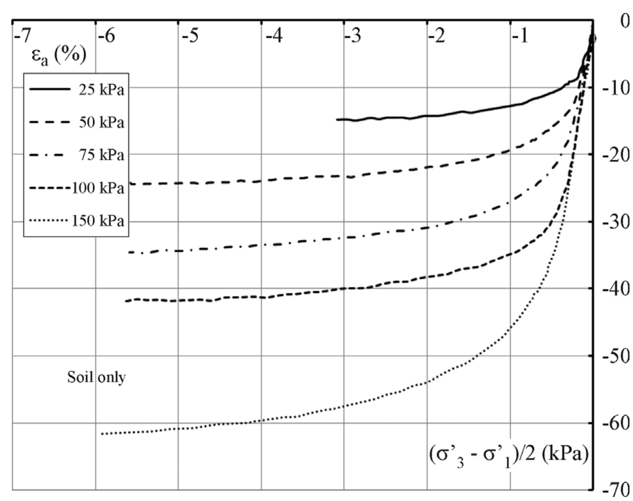

Figure 4 - Stress-strain curves of soil only for confining stress from 25 to 150 kPa.

The deviator stress (σ'_d) is negative both in the loading and unloading extension tests.

The comparison of the Figs. 3 and 4 for the same shear stress shows that the soil without roots deformed more than the soil with roots. On the other hand, for the same strain, for example 2%, the soil with roots exhibited shear strength greater than the soil alone. The exception was for the confining pressure of 25 kPa, when both soils had similar behavior.

Furthermore, for the soil with roots, when the confining pressure increased, the strain at failure became larger. The soil under low confining pressure had brittle behavior and the soil at high confining stress had ductile behavior. This behavior is similar to that of loose and dense sands described by Lee & Seed (1967), due to the critical confining pressure and critical void ratio.

The samples of soil with roots had void ratio after consolidation phase ranging between 0.69 and 0.78. This is not reason enough to determine such different behavior between samples. Nevertheless, the tests at higher confining pressures were conducted on samples with smaller void ratios, and to avoid the dilatancy effect on shear strength, the samples tested at low confining pressures had higher void ratios.

For soil without roots, with the exception of the sample tested at confining pressure of 25 kPa, whose strain at failure was 2.5%, the rupture fell between 5 and 6% of axial strain.

Figure 5 shows that only the samples tested at confining pressure below 75 kPa exhibited a smooth dilatant behavior at shear, despite higher void ratios than samples tested at confining pressure above 75 kPa.

The data from the triaxial extension drained unloading tests performed on soil samples without roots are represented in the p-q diagram of Fig. 6. Figure 7 shows the results for the soil tests with roots.

Figure 6 establishes a Mohr-Coulomb strength envelope with -12.5 kPa for cohesion intercept and -34.6° for internal friction angle in the soil without roots.

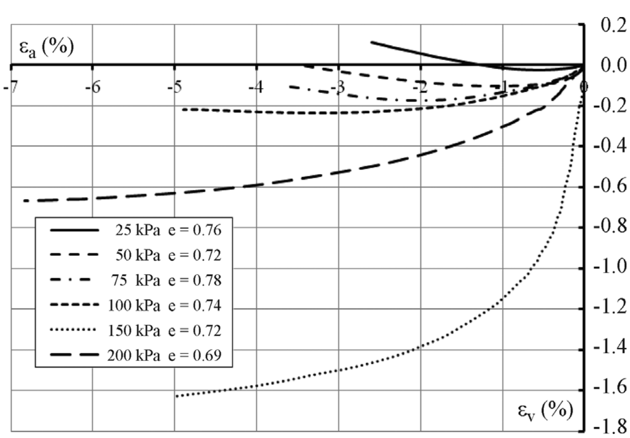

Figure 5 - Volumetric changes of soil with roots for confining stress from 25 to 200 kPa.

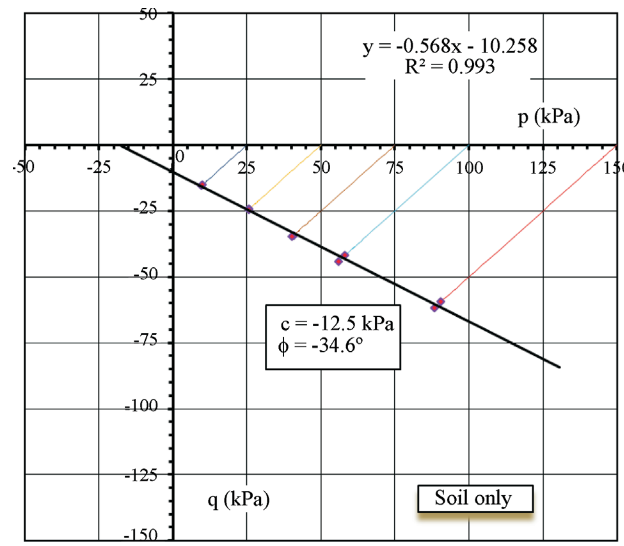

Figure 6 - Mohr-Coulomb envelope in p-q diagram for soil only.

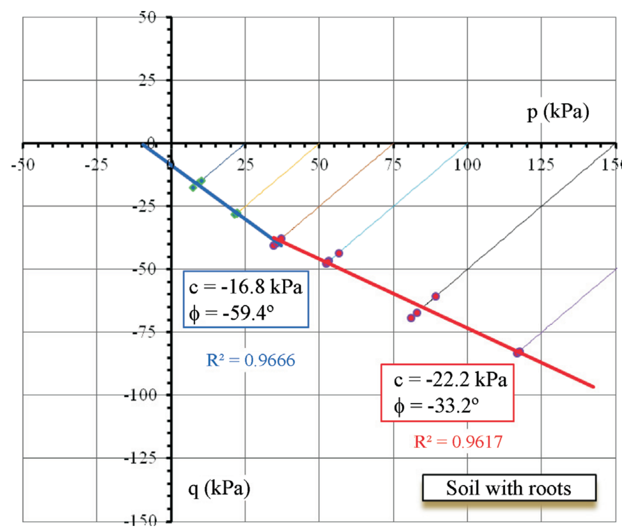

Figure 7 - Mohr-Coulomb envelope in p-q diagram for soil reinforced with roots.

In the p-q diagram of Fig. 7, the triaxial extension drained unloading tests performed on samples of soil with roots showed that above 75 kPa of confining pressure, a Mohr-Coulomb strength envelope could be modeled with similar slope to the soil without roots and with linear coefficient of -22.2 kPa, which corresponds to an extrapolation of the envelope for zero confining pressure, called cohesion intercept.

Below 75 kPa of confining pressure, an initial segment of the envelope could be modeled with the results of tests performed at confining pressures of 25, 50 and 75 kPa. This fitted branch could provide the shear strength parameters, internal friction angle of -59.4° and cohesion intercept of -16.8 kPa. Therefore, the envelope for the soil with roots was modeled as bilinear, with change of slope at the confining pressure of 75 kPa.

The bilinear envelope was reported long ago as a characteristic of reinforced soil (Gray & Ohashi, 1983, Gray & Al-Refeai, 1986 and Maher & Gray, 1990). Gray & Ohashi (1983), by performing direct shear testing on reinforced sands, concluded that the fibers under low confining pressure tend to be pulled out and the soil exhibits a higher friction angle. Above a threshold stress, the internal friction angle of the soil is not affected by the fiber and the envelope has the same internal friction angle for both the soil alone and reinforced soil.

For confining pressure above 75 kPa, the friction angle of soil reinforced with roots was the same as soil without reinforcement, although in modeling the Mohr-Coulomb envelope, the cohesion parameter, which corresponds to the linear coefficient, was different from one sample to another. In both cases, the cohesion was here taken as a cohesion intercept and not a cementation or real cohesion. This does not mean that the fibers give cohesion to the soil. There may be some aggregation, but the envelope model presented a greater intercept of cohesion for confining pressure above 75 kPa.

Also in the initial portion of the envelope, the soil with roots presented a friction angle greater than that of the soil alone. The cohesion intercept of -16.8 kPa for the reinforced soil was slightly different from the value of -12.5 kPa for soil alone. This difference, although small, might be due to a cohesive woof formed by the roots.

Miranda Neto (2015) showed that the turning point in the Mohr-Coulomb envelope for extension tests in soil with roots occurs at normal stress at failure σ_N = 5.2 kPa and shear strength τ = -25.6 kPa. For confining pressures ($\sigma'_1 = \sigma'_h = \sigma'_c$) of 25, 50 and 75 kPa, the respective minor main effective stresses ($\sigma'_v = \sigma'_3$) are negative and represent a plausible stress state in the triaxial test, but are not feasible in situ, for example, in case of slope analysis and stabilization. Figure 8 illustrates that the stress state where the minor effective principal stress (vertical) is zero corresponds to a major effective principal stress (confining pressure) of 82.2 kPa at failure and a shear strength of -34.2 kPa. The vertical effective stress equal to zero in a geostatic stress is the ground surface. Therefore, in this case, the occurrence of the first branch of the envelope is not feasible.

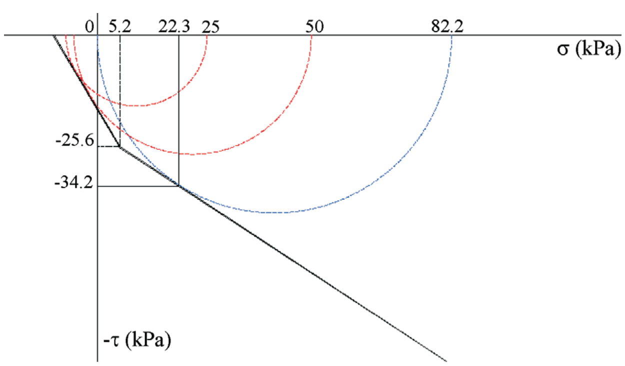

Figure 8 - Mohr-Coulomb envelope in τ - σ space for extension test with roots.

Complementing this study of shear envelope, were also performed triaxial compression drained tests on samples of soil with roots and without roots at confining stresses ranging from 25 to 150 kPa. The results are shown in Fig. 9.

There were no significant variations observed in the triaxial compression drained tests for the samples either with or without roots. The adjustments of the envelopes resulted in sufficiently straight lines to discard the bilinear model for this compression case. This can be explained by the predominantly vertical architecture of the root system, as seen in Fig. 10 (d). The samples with roots in the axial direction, the same direction as the compressive stress deviation (σ'_1 - σ'_3), did not show effective participation of roots in shear strength. Even though the stress state in the triaxial cell induced a shear plane inclined with respect to the preferred direction of the roots, the soil showed similar shear strength with and without roots in the compression tests.

In these triaxial compression tests, the peripheral roots forced the rubber membrane and buckled it even against the confining pressure in the triaxial cell, as seen in Fig. 10 (b). Wu *et al.* (1988) concluded: "In compression, the roots failed by buckling". This phenomenon seems to have occurred in these triaxial compression tests.

Figure 10 (a) illustrates a sample of soil without roots in extension for 200 kPa confining pressure, showing stretching and a set of failure planes; (b) illustrates a specimen subjected to triaxial compression with roots showing buckling; (c) shows general aspects of the roots taken from a specimen; and (d) shows a mold with partially removed soil showing the root system architecture.

Note in Fig. 10 (a) that the planes of failure for samples submitted to extension testing are close to orientation of slip lines for Rankine passive state (Lambe & Whitman,

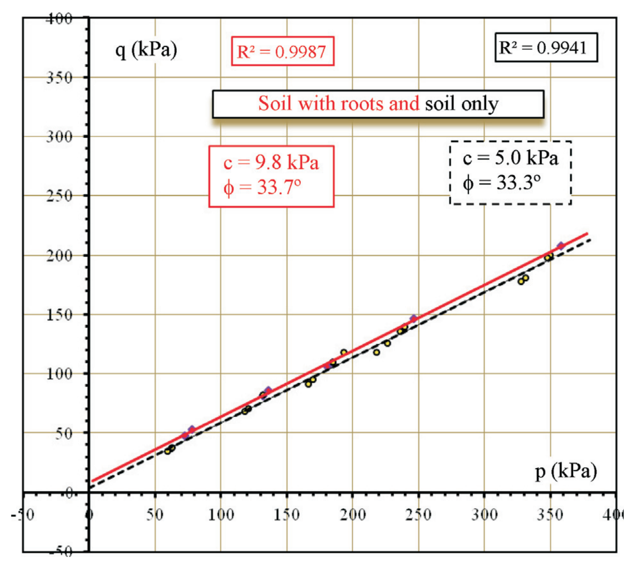

Figure 9 - Mohr-Coulomb envelope in p-q diagram for triaxial drained compression tests.

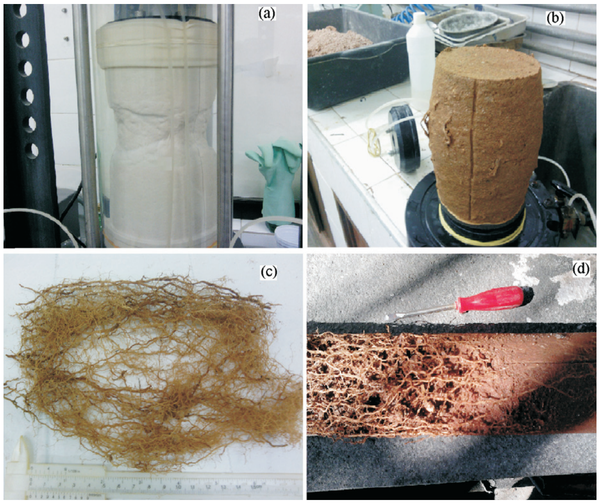

Figure 10 - (a) sample of soil at failure; (b) buckling of the roots; (c) roots of one sample; (d) root system architecture.

1969). In triaxial compression tests, these planes are steeper. The planes of failure are not orthogonal to the preferential direction of the roots.

Figure 10 (c) shows the roots from a sample subjected to extension testing at confining pressure above 75 kPa. There is no evidence that the roots were broken. Gray & Ohashi (1983) considered that the reinforcement does not break because it is more extensible than the soil: "Inclusions have rupture strains larger than the maximum tensile strains in the soil without inclusions." Therefore, assuming that the roots are more extensible than the soil, these roots cannot rupture regardless of their ultimate strength or the imposed load (Gray & Ohashi, 1983).

The fiber area ratio of the samples with roots ranged from 0.28% to 0.74%, close to the fiber area ratios tested by Gray & Ohashi (1983). Most tests in this study were run at fiber area ratio of 0.45%.

Since the fiber area rates were similar for most samples, it was not possible to analyze in this respect the most brittle behavior of the samples at confining pressures up to 75 kPa, especially for 50 and 75 kPa, which showed less strain at failure for soil with roots than soils without roots. This is a matter for more detailed research, due to issues related with the dilatancy effects at low confining stress (Lee & Seed, 1967) and the fiber aspect ratio (Maher & Gray, 1990) or fiber content (Gray & Al-Refeai, 1986).

Using the strength parameters obtained in this study, Miranda Neto (2015) performed a stability analysis by slice method on a naked slope and a slope using vetiver grass and verified that the gain in the safety factor for circular surface was of the order of 16%.

Table 1 shows an individual gain in shear strength for some levels in effective normal stress at failure (σ'_N) for extension test while Table 2 shows the same gain in shear strength for compression testing. There is a reduction in the gain with the increase of the tension level for extension testing.

4. Conclusions

Triaxial drained tests were performed on soil samples with roots and without roots of vetiver grass, grown naturally in molds. Test results showed that vetiver roots can increase shear strength of the soil used in this study and the following conclusions emerged.

The vetiver roots increased the shear strength of the soil for extension unloading up to 30% near ground surface and decreased with depth until less than 7% close to the extremity of the root system.

For compression loading, the increase in shear strength was less than 24% near the ground surface and decreased to less than 8% close to the end of the root system.

Table 1 - Gain in shear strength by stress level in extension test.

σ'_N (kPa)	Soil with roots		Soil only		τ_{root} (kPa)	τ_{soil} (kPa)	τ_{root}/τ_{soil} (%)
	c (kPa)	ϕ°	c (kPa)	ϕ°			
25	-22.2	-33.2	-12.5	-34.6	-38.5	-29.7	30
50	-22.2	-33.2	-12.5	-34.6	-54.9	-47.0	17
75	-22.2	-33.2	-12.5	-34.6	-71.3	-64.2	11
100	-22.2	-33.2	-12.5	-34.6	-87.6	-81.5	7.5

Table 2 - Gain in shear strength by stress level in compression test.

σ'_N (kPa)	Soil with roots		Soil only		τ_{root} (kPa)	τ_{soil} (kPa)	τ_{root}/τ_{soil} (%)
	c (kPa)	ϕ°	c (kPa)	ϕ°			
25	9.8	33.7	5.0	33.3	26.5	21.4	24
50	9.8	33.7	5.0	33.3	43.1	37.8	14
75	9.8	33.7	5.0	33.3	59.8	54.3	10
100	9.8	33.7	5.0	33.3	76.5	70.7	8

The contribution of vetiver roots in this soil for extension unloading increased the cohesion intercept for confining stress above 75 kPa. Below 75 kPa, despite an increase in frictional angle, no effective contribution is possible because it is not a feasible stress state.

The vetiver roots in soil acted as extensible reinforcements.

Since the length of these roots is at most 4 m, the effective improvement caused by the roots only applies to the topsoil. Nevertheless, the improvement of topsoil can cause an increase in overall slope stability of 16%.

Acknowledgments

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