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Shear Strength Behavior of a Residual Soil of Gneiss Compacted at Metastable-Structured Conditions

J.H.F. Pereira
University of Brasília, Departament of Civil Engineering, Brasília, Brazil

D.G. Fredlund
University of Saskatchewan, Department of Civil Engineering, Saskatoon, Saskatchewan, Canada

ABSTRACT: The influence of wetting-induced collapse on the shear strength envelope of a compacted metastable-structured residual gneiss soil was experimentally investigated. Shear strength was investigated using a modified direct shear box equipment wherein the stress state variables were independently controlled. The compacted specimens were consolidated at the shearing box at specified net normal stresses. Measurements of total volume change and water content change were made at specified matric suction values following a wetting stress path. The experimental data were analyzed to define the shear strength envelope constitutive relationship for the metastable-structured soil. A constitutive model is proposed for predicting the shear strength behavior of the compacted metastable-structured residual soil which undergoes wetting-induced collapse. The proposed constitutive model is applied to the prediction of the shear strength behavior of a compacted metastable-structured soil under different external loading conditions. Prediction is compared to experimental results.

KEYWORDS: Shear Strength, Unsaturated Soil, Collapsible Soil, Modeling.

1. INTRODUCTION

The mechanical behavior of soils in their unsaturated condition, where only a fraction of the pore-voids is filled by water, can not be satisfactorily described by using classical soil mechanics principles for saturated soils. The collapsing behavior of soils has been used as an indication of the limitation of a generalized single effective stress principle (Bishop et al. 1960) embracing both saturated and unsaturated soils (Matyas and Radhakrishna, 1968).

A collapsible soil is commonly referred to as a metastable-structured soil. An increase in pore-water pressure results in swelling for a stable-structured soil, whereas an increase in pore-water pressure may cause a volume decrease for a metastable-structured soil (Barden et al. 1973). The definition of a failure criterion establishes the conditions at which the shear strength of the soil is reached. Post-failure behavior must define the new volume change behavior of the failed material as a function of the stress state variables and/or deformation variables.

The shear strength of an unsaturated soil can be formulated in terms of independent stress state variables (Fredlund et al. 1987). The stress state variables, $(\sigma - u_a)$ and $(u_a - u_w)$, have been shown to be the most advantageous combination for practice. Using these stress variables, the shear strength equation is written as follows:

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan\phi' + (u_a - u_w)_f \tan\phi^b \quad (1)$$

where: c' = effective cohesion, $(\sigma_f - u_a)_f$ = net normal stress state on the failure plane, u_{af} = pore-air pressure at failure, ϕ' = angle of internal friction related to the net normal stress, $(u_a - u_w)_f$ = matric suction on the failure plane at failure e ϕ^b = angle indicating the rate of increase of shear strength relative to the matric suction.

The above equation includes in an elementary manner the effect of shear strength increasing with matric suction. The required parameters, (i.e., c', ϕ' and ϕ^b) can be determined from laboratory tests. Alternatively, for a non-deformable soil the ϕ^b parameter can be estimated from the soil-water characteristic curve (Fredlund et al. 1995).

Equation 1 defines a plane. The failure plane envelope can be modified to accommodate non-linearity associated with the soil parameters involved, c', ϕ', and ϕ^b. Experimental evidence (Fredlund et al. 1987) has demonstrated that the shear parameters c' and ϕ' are relatively constant for stable-structured soils. In turn, ϕ^b changes as a function of the matric suction and ϕ' is a maximum value for ϕ^b (Escário et al. 1973; Fredlund et al. 1987).

Previous studies (El-Sohby et. al. 1987, Pereira 1996) alerted to the high influence of the microstructure on the shear strength. The amount of clay fraction and arrangement of clay bonds at interparticle contacts can render a variety of shear strength behavior for a compacted metastable soil structure.

It is now known that a collapsible clayey soil, when loaded at its natural water content, can maintain its original structure with no significant volume change. This fact is attributed to action of strong bonds between the coarse particles which prevent relative movements, (Jennings and Knight 1957, Alonso et al. 1985). At this natural water content the small changes in the soil volume, due to external loading, could be attributed to the compression of the fine soil, (i.e., clay bonds, between the coarser material). In general, a clayey soil shows a high dry shear strength that is reduced when saturation takes place. This reduction in strength can also be related in a general way to changes in matric suction, but on a microscopic level the van der Waals attraction, the double layer repulsion, and the adsorbed water should be taken into account.

There is a gradual increase in compressibility of a collapsible soil during the saturation process (Alonso et al. 1985, Fredlund and Rahardjo, 1993). After the collapse, the soil possesses a different structure from the original clayey bonds controlling the soil behavior at the pre-collapse condition.

The definition of the soil strength parameters c', ϕ ' and ϕ^b becomes more complex for a collapsible soil. Depending upon the total stress applied to the soil, a metastable structure can change to a stable structure when collapse occurs. A typical collapsible soil usually has a fraction of clay between 10% and 20%, while the remainder is composed of coarser material; mainly silt and fine sand. The clay particles may be present in different forms in the soil structure. Collapse occurs as a result of shear failures at water-softened interaggregate or intergranular contacts, or the softening and distortion of clay aggregates.

Depending on the matric suction and the vertical stress applied, the shearing process can induce a progressive and gradual transference of the shearing forces from the clay bonds and/or clay aggregations to the coarser particles at the failure plane. For high values of matric suction clay aggregations may remain intact after shearing and behave similar to sand. For low values of matric suction, the clay bonds and/or aggregations may collapse, depending on the net normal stress. The shearing of clay bonds and/or aggregations means a decrease in the soil cohesion and might mean an increase in the frictional strength on the failure plane. The magnitude and rate of this transference depends on factors such as: applied stress state, initial condition of the specimen, and the rate of shearing.

The influence of soil collapse on the shear strength still needs additional research (Schmertmann 1976, Alonso et. al. 1985, El-Sohby et. al. 1987). From previous studies it seems reasonable to evaluate the shear strength of a collapsible soil in an experimental manner. This implies that the shear parameters c', ϕ' and ϕ^b must be obtained using a range of stress state variables and stress paths compatible with the problem being analyzed. In this research program a series of modified direct shear tests were carried out in order to define a failure envelope for an collapsing soil.. The laboratory program carried out for the present study is later discussed in this paper. Details are reported in Pereira (1996).

2. LABORATORY TESTING PROGRAM

A laboratory program was designed with the main objective of defining the shear strength mechanical behavior of a compacted collapsing soil gradually saturated by water. The saturating of the collapsing soil would simulate the transient unsaturated-saturated water flow during the saturating of an small collapsing dam according to previous studies (Miranda, 1988; Pereira, 1996). The range of stress state variables and stress paths used were defined considering the following factors:

a.) the initial condition of the compacted soil in terms of both net normal stress and matric suction.

b.) the gradual decreases in matric suction during the development of the transient water flow process into the collapsing dam before steady-state conditions are reached;

c.) a range of net normal stress which correspond to an small earth dam structure;

2.1. Testing material

The soil used in the present research study is a residual soil derived from a granitic gneiss of the Ceara' group and is representative of a typical soil used in the construction of small dams in Northeast Brazil. Miranda (1988) and Pereira (1996) used this material in the analysis of failure mechanisms of "Alka-Seltzer" dams during first reservoir filling.

2.1.1. Characterization of the Testing material

The soil is a residual silty sand. The index properties are presented in Table 1.

Table 1. Index properties of the soil (Pereira, 1996).

Soil	Residual silty sand derived from gneiss
Location	County of Pacatuba in the State of Ceara'/Brazil
Natural water Content	2
Grain size Distribution	Sand - 52 %; Silt - 35 %; Clay - 13%; D10 - 0.0006 mm; D30 - 0.016; D60 - 0.22 mm
Atterberg limits	Liquid limit, w_l = 29; Plastic limit, w_p = 17; Plasticity index, PI = 12;
Specific Gravity	G_s = 2.64;
Unified Soil Classif. System	SW-SM; well-graded sand with silt

Laboratory tests to define the shear strength envelope for the collapsing soil was performed on statically compacted specimens using material passing the 2.0 mm sieve. The amount of material larger than 2 mm is less than 2 %.

2.1.2. Properties of the compacted testing soil

Specimens extracted from "Alka-Seltzer" dams in northeast Brazil demonstrate that the average dry density is about 14.75 kN/m^3 (Miranda, 1988). An initial water content was selected such that the initial matric suction could be measured and controlled using equipment available in the laboratory at the University of Saskatchewan.

The matric suction of 370 kPa was measured for the specimen compacted at a water content of 10.5%. At this compaction condition the soil specimen presented a void ratio of about 0,754 and a degree of saturation of about 37%. The collapsing behavior of the soil compacted at 10.5 % was checked by using double oedometer tests (Jennings and Knight, 1957), according to the procedure ASTM-D5333 - 92. The verification of a significant amount of collapse, the initial measured matric suction, and the practical aspect of workability of the specimen, led to the choice of a gravimetric water content of 10.5 %. The initial condition of the compacted collapsing soil corresponds to a water content of minus 4% dry of optimum conditions and a density of 90% of the corresponding point on the AASHTO standard compaction energy curve. The relative density of 90% reflects the actual conditions, in terms of equipment availability, at the site of the construction of Alka-Seltzer dams (Miranda, 1988; Pereira, 1996).

Figure 1 shows the variation of void ratio with respect to the applied vertical stress (i.e., collapse potential of the compacted soil tested) obtained from the double oedometer tests. The range of vertical stresses used for the study was 0 to 800 kPa. However, the range of interest for practical conditions is lower than 200 kPa (i.e., based on the geometry of Alka-Seltzer dams and according to previous studies (Miranda 1988). Figure 1 also shows that the soil specimen did not present any collapsing behavior for applied vertical stresses lower than 50 kPa. A vertical stress of 100 kPa produced soil collapse amounting to 3.0% which increased to about 7.2% at a vertical stress of 200 kPa.

The soil presented low compressibility when loaded under unsaturated, or as-compacted, conditions. Upon saturation, by inundation at a vertical stress of 400 kPa, the measured soil collapse was greater than 11% and closely coincided with the saturated stress path curve. An increase to 800 kPa in the vertical stress resulted in a similar response for both saturated stress path curves. When unloaded to 25 kPa of vertical stress there was essentially no strain recovery of the specimen volume.

Figure 2 shows a conventional drying soil-water characteristic curve for the compacted collapsing soil. This information was used to provide a preliminary estimate of the range of matric suction to be used in the subsequent phase of the laboratory program. The tests were performed using Tempe-cell and pressure-plate apparatus, and based on the ASTM D 3152-72.

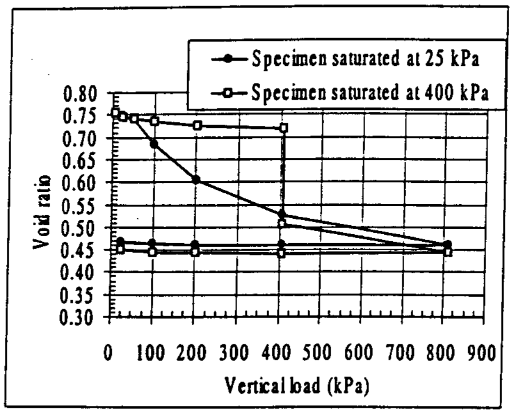


Figure 1. Double oedometer test results.

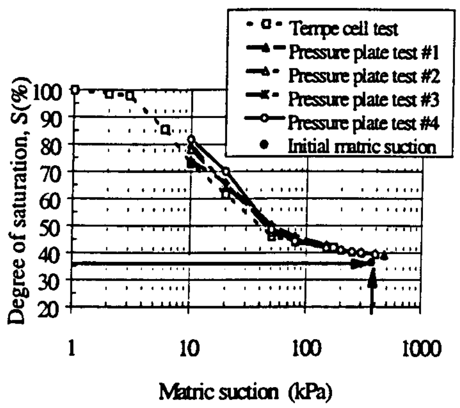


Figure 2. Drying soil-water characteristic.

The drying soil-water characteristic curve showed that the soil specimen started desaturating at a matric suction of about 3.0 kPa. The degree of saturation of the specimens dropped to values of less than 50% for suction values of about 60 kPa. The results indicate a residual degree of saturation around 40%. Figure 2 also shows the initial matric suction (i.e., 370 kPa) of the collapsing compacted soil measured by using null-type axis-translation tests.

From the drying soil-water characteristic curve, it can be inferred that the soil coefficient of permeability would become extremely small at a matric suction around 100 kPa. Under a wetting stress path, a very small increase in the volumetric water content from the initial conditions in the soil (i.e., matric suction equals to 370 kPa) up to an applied matric suction of 80 kPa was expected.

2.2 . Evaluation of the shear strength behavior of the collapsing soil behavior during saturation

The modified direct shear apparatus for testing unsaturated soils was designed and built by Fredlund et. al. (1987). This equipment has been used to define the shear strength envelope of a variety of different soils (Fredlund et al. 1987; Pereira, 1996). For the modified direct shear tests, a range from 0 to 100 kPa of matric suction values was chosen. This range was mainly based on the fact that after the residual degree of saturation has been reached, there is no significant increase in shear strength (Escario et al. 1986; Fredlund et al. 1987). The range of vertical net normal stress was defined from 20 to 200 kPa based on the geometry of a typical "Alka-Seltzer" dam. Wetting stress paths were also used in these tests. The steps for the stress state variables are later detailed in this paper.

The axis-translation technique was used in all tests of the laboratory program. The tests were performed in both saturated and unsaturated conditions using the modified direct shear box.

Each 51 x 51 x 21 mm soil specimen was extruded from a 100 mm diameter by 25 mm thick statically compacted specimen. The tests were performed according to the procedures and recommendations suggested by Gan (1986). After assembling the specimen in the shear box, the following procedure was performed:

a.) The vertical deflection of the specimen was monitored using a LVDT adjusted on the loading system. A predetermined vertical load was applied to the specimen and the desired matric suction was applied.

b.) The specimen was then allowed to equilibrate under the predetermined vertical net normal stress and the applied matric suction. Changes in total volume (i.e., LVDT readings), were periodically recorded by means of a data acquisition system. Consolidation was allowed until the complete equilibration of both total volume changes and volumetric water content of the specimen was achieved.

c.) After equilibration (an average of 5 days), the specimen was sheared at a constant rate under drained conditions.). A shear rate of 2.4 mm/day was found to be satisfactory (Escario et al. 1986, Fredlund et. al. 1993. All the tests were performed in a single stage. The matric suction range selected was from zero to 100 kPa.

The shear tests were performed according to the program presented in Table 2. A wetting path was followed for each test. On average, one week was required to test each sample.

Table 2. Stress state variables for the modified direct shear tests on the collapsing soil.

Stress State Variable (kPa)	Path 1 (kPa)	Path 2 (kPa)	Path 3 (kPa)	Path 4 (kPa)
$(\sigma - u_a)$	25	50	100	200
(u_a-u_w)	0	0	0	0
(u_a-u_w)	25	25	25	25
(u_a-u_w)	50	50	50	50
(u_a-u_w)	100	100	100	100

3. TESTING RESULTS

Pereira(1996) presents details of the collapsing behavior of the dry of optimum compacted soil during the consolidation phase in terms of both total volume change (i.e., in the form of vertical deformation) and water phase volume change versus time. These results correspond to the four applied vertical stresses (i.e., 25, 50, 100 and 200 kPa).

On a qualitative basis, the results show that the measured volumetric changes corresponding to the consolidation of the soil specimens in the direct shear box were similar to the measured results obtained from the double oedometer and triaxial permeameter tests (Pereira, 1996). Despite the inherent differences in confining conditions, as compared to the triaxial permeameter system, the k_0-consolidation in the shear box could be utilized to reinforce the following characteristics of the collapsing soil behavior for the soil tested:

i.) when saturated under a vertical stress of 25 kPa, the dry of optimum compacted soil does not present a collapsing behavior;

i.i) there is a nonlinear soil collapse behavior with respect to a given vertical load for decreasing matric suctions;

iii.) the soil collapse was completed before the soil specimen reached complete saturation.

Pereira (1996) shows the shear stress versus shear displacement relationships for the collapsing soil under vertical stresses of 25, 50, 100 and 200 kPa, respectively. The shear tests were performed under drained conditions, at applied matric suctions of 100, 50, 25 and 0 kPa for each vertical stress. Figure 3 illustrates the shear strength versus shear displacement relationships for the collapsing soil under a verticalstress of 100 kPa.

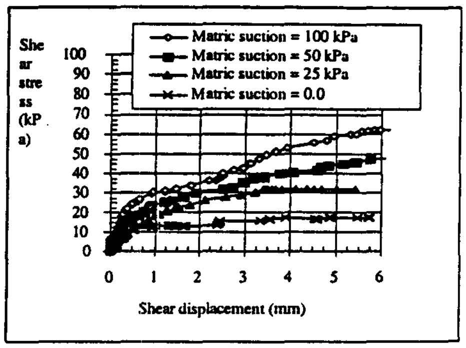


Figure 3. Direct shear tests forthe collapsing soil under a vertical estress of 100 kpa.

During shearing the soil specimens behave either as a loose sand (i.e., compresses during shear) or as compact sand (i.e., dilates during shear) depending upon the applied stress state(i.e., net normal stress and matric suction). saturated conditions (i.e., at a matric suction of 0 kPa) for all the applied vertical stresses, the soil compresses during shear. Under vertical stresses of 25 and 50 kPa and at matric suctions ranging from 25 to 100 kPa, the soil specimens dilate during shear. The same behavior was observed for soil specimen under a vertical stress of 100 kPa and at a matric suction of 100 kPa. Under a vertical stress of 200 kPa, the soil compresses during shear for matric suctions ranging from 25 up to 100 kPa.

The results indicates that, irrespective of the volume changes induced in either the consolidation and shear phases, the shear strength of the collapsing soil increases as a combined function of the vertical stress and the applied matric suction.

Despite the fact that this conclusion reflects the anticipated behavior of any unsaturated soil, some comments are warranted to explain the combined results of volumetric changes and shear strength for the collapsing soil. The following comments attempt to explain the shear strength behavior of a compacted collapsing soil with low clay content, which is compacted at dry of optimum water content:

i.) The structure of an uncemented compacted collapsing soil with a low clay content has been described (Barden et al. 1973; Miranda, 1988) as constituted of clay aggregations (i.e., macropeds) and coarser grains (i.e., sand and silt) connected by bonds of finer particles (i.e., clay and silt particles). The strength of such bonds and clay aggregations is highly dependent upon capillary action. Any external loading generates shear stresses at the connecting bonds. The unsaturated soil does not collapse as long as the local shear strengths of the connecting bonds are higher than the acting shear stresses. During saturation, both the clay aggregations and the connecting bonds soften and weaken due to the reduction in matric suction. The soil collapses as the local shear strength at the connecting bonds is overcome by the shear stresses induced by the applied loading. The soil microstructure is complex in terms of pore-size distribution (Matyas and Radhakrishna, 1968; Lawton et. Al. 1991; Alonso et. al. 1985), and each connecting bond is sheared according to its local shear strength. The breaking of connecting bonds and/or clay aggregations implies the local redistribution of stresses among the remaining clay aggregations and coarser particles. The soil skeleton can reach a new equilibrium depending on the external load and the remaining strength of the connecting bonds and clay aggregations. The breaking of connecting bonds and clay aggregations occurs in a progressive manner as the matric suction decreases and/or the acting external load increases.

ii.) The double oedometer test performed on the collapsing soil (see Figure 1) illustrates that there is a gradual increase in soil collapse when the vertical stress is increased from 25 to 800 kPa. This suggests that the metastable structure (i.e., soil skeleton supported by clay aggregations and connecting bonds) of the collapsing soil remains partially intact even under vertical stresses as high as 400 kPa. At a vertical stress of 800 kPa, the void ratio of the collapsing soil is about 0.45, which is close to the void ratio of the residual soil of gneiss compacted at optimum conditions.

iii.) In the consolidation phase, each soil specimen was brought to equilibrium conditions under a defined stress state following a wetting stress path. The behavior of the collapsing soil during consolidation(Pereira, 1996) shows that there is a progressive increase in the total soil collapse (i.e., at matric suction of 0 kPa) when the vertical stress applied to the soil specimens is increased from 25 to 200 kPa. The saturation of the specimen, even under an applied vertical stress of 200 kPa, is not enough to destroy the structure of the collapsing soil. Under a given net vertical stress, the higher the applied matric suction, the higher the amount of remaining bonds and clay aggregations binding together the structure of the collapsing soil. This fact leads to the conclusion that each soil specimen (i.e., possessing a specific metastable structure after consolidation) will present a specific behavior during the shearing phase.

iv.) The shearing phase induces shear stresses and consequently alters the existing equilibrium stress state in the connecting bonds and clay aggregations along the failure surface. Despite the well known limitations of direct shear tests to provide quantitative information on either stress distribution or stress-strain characteristics of a soil, a qualitative analysis can be done on the test results. The imposed stress state around the failure surface can induce local shearing of connecting bonds and/or clay aggregations. Additional soil collapse occurs when the induced local shear stress overcomes the available local shear strength. As the connecting bonds and/or clay aggregations are sheared, the remaining clay aggregations and coarser particles behave as a granular soil along the failure plane. Depending on the net confining stress (i.e., a direct function of the applied stress state) and the interlocking of the granular structure (i.e., remaining from the consolidation phase), a soil specimen can undergo either dilation or compression during shearing.

v.) Additional collapse of the soil structure implies a gradual transfer of shear stresses from connecting bonds and clay aggregations on the failure plane, to coarser particles and remaining clay aggregations around this failure plane. Continuous breaking of connecting bonds and clay aggregations will result in a continuous decrease in void ratio which generally results in an increase in shear strength. The shearing of the connecting bonds should predominate at low shear deformations, since the shearing of the soil skeleton is necessary in order for shear displacement to take place along the failure plane.

vi.) Under unsaturated conditions, the collapsing soil specimens present a stiff soil skeleton that behaves like an elastic rigid body at low shear deformations. The shear displacements start as the soil skeleton strength (i.e., the connecting bonds and/or clay aggregations along the failure plane) is overcome by the induced shear stress. The mobilized failure surface might present an irregular shape and the soil specimen will tend to dilate as shear displacement continues. The dilating behavior is a combined function of the vertical stress and the strength of the soil skeleton along the irregular failure surface.

The results, corresponding to vertical stresses of 25, 50 and 100 kPa, illustrate a well-defined primary shear strength peak at low shear deformation for all the specimens tested, irrespective of the applied matric suctions. Such a behavior, although not so well-defined, was also observed for the soil specimens under a vertical stress of 200 kPa. Similar behavior has been observed in artificially cemented sand specimens (Lefebvre 1995) and in naturally cemented loess specimens (Lin 1995). In the present study, the cementing effect represents the breaking of connecting bonds holding the metastable soil skeleton. The results also illustrate that the shear behavior of the soil specimens after the primary peak, is a function of the combined effects of the applied vertical stress and the matric suction. The continuous contracting behavior observed on the saturated soil specimens, as opposed to the equivalent strain-hardening stress-strain curves, is related to a continuous decrease in void ratios during shearing, leading to a continuous increase of shear stress in these high initial void ratio collapsing soils. The strain-hardening behavior of the non-collapsing soil specimens (i.e., tested under vertical stresses of 25 and 50 kPa and at matric suctions higher than 20 kPa) is related to the interlocking effects in the soil. In general, the direct shear tests show the role of matric suction on the shear strength of soil specimens compacted dry of optimum water content, at low dry densities.

4. SHEAR STRENGTH ENVELOPE SURFACE

The soil model was defined by using best-fit analyses in the search for functional relationships that could capture the essential characteristics of the behavior of the soil as observed from the available experiments. Using the software SigmaPlot, Pereira (1996) performed the best-fit analysis.

The available shear strength data for the collapsing soil (Pereira, 1996) illustrate the predominant strain-hardening behavior of the collapsing soil when it is sheared at unsaturated conditions under a net vertical stress higher than 25 kPa. The shear strength of the soil specimen allows the definition of an extended Mohr-Coulomb shear strength envelope for a collapsing soil. There appears to be no well-established criteria in the literature, for the definition of failure for strain-hardening soil behavior. The conventional approach is to define failure based on a strain criteria. Such an approach may lead to the definition of shear strength parameters that increase with increasing shear displacement (Terzaghi and Peck 1967). The choice of a deformation criteria compatible with the field problem at hand minimizes inaccuracies from the use of the strain criteria approach. Based on the fact that the shear tests with a well-defined peak strength maintained their strength for large shear displacements and on the deformations expected to occur in "Alka-Seltzer" dams, it was assumed that: *In the absence of a well-defined peak strength, failure would occur when the shear stress versus shear displacement curve reached a shear displacement of 5 mm.* Similar procedures were used by others researchers (Campos et al. 1995).

Figure 4 shows the shear strength versus matric suction relationships under various net vertical stresses for the collapsing soil. Figure 5 shows the shear strength of the collapsing soil plotted against the net vertical stress.From a phenomenological standpoint, figures 4 and 5 can be used to define the extended Mohr-Coulomb envelope (i.e., shear strength surface) for the collapsing soil. The software SigmaPlot was used to define the best-fit coefficients for the extended Mohr-Coulomb failure envelope.

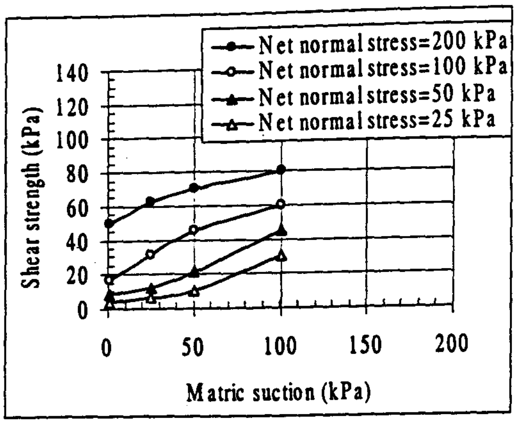


Figure 4. Shear strength versus matric suction for the collapsing soil

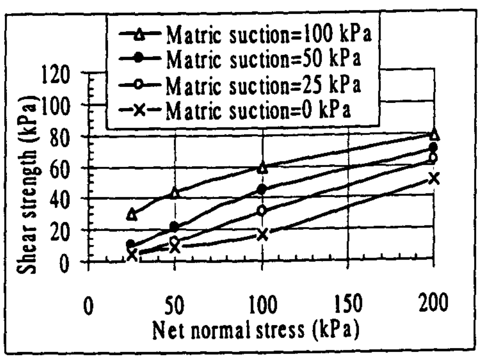


Figure 5. Shear strength versus net vertical stress for the collapsing soil

Equation 2 expresses the resulting best-fit mathematical model for the extended Mohr-Coulomb failure envelope for the collapsing soil.

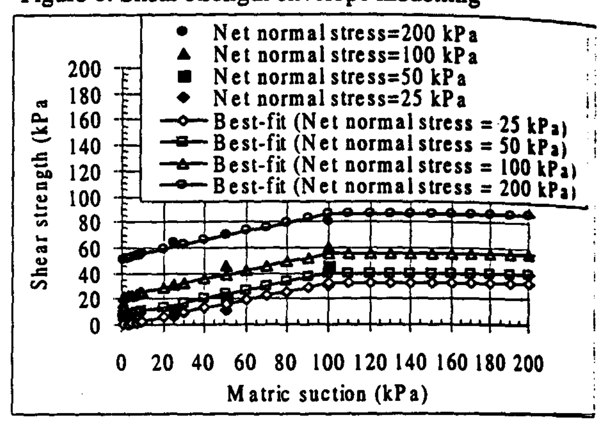
$$\tau_{ff} = a_1 + b_1(\sigma - u_a) + c_1(u_a - u_w) + d_1(\sigma - u_a)(u_a - u_w)^p \quad (2)$$

where: $a_1 = -7.893$, $b_1 = 0.1944$, $c_1 = 0.3238$, $d_1 = 0.09319$, $p = 0.04307$.

Equation 2 can be seen as a phenomenological prediction of the shear strength envelope of the collapsing soil for the range of matric suction from 0 to 100 kPa. Additionally, the study assumes that at a given net normal stress, the shear strength of the collapsing soil remains constant for the range of matric suctions from 100 to 370 kPa. Such assumption is based on the fact that for matric suctions higher than the corresponding to the residual degree of saturation an unsaturated soil presents an angle ϕ^b which tends to zero (Vannapali, 1994). Figure 6 illustrates the comparison between the best-fit results and the experimental data in terms of shear strength versus matric suction for various net vertical stresses..

Figure 6 suggests that best-fit modeling of experimental data might be a useful tool to the definition of the shear strength behavior of a metastable-structured compacted soil. However, a best fit modeling always requires a search for the physical meaning of the parameters involved.

Figure 6. Shear strength envelope modelling



5. CONCLUSIONS

The shear strength versus matric suction relationships for a metastable-structured soil is complex and highly affected by the wetting-induced soil collapse. Available data suggests that at high matric suctions (i.e., when there is no collapse) a metastable-structured soil can maintain its opened structure intact. In turn, the available data suggests that when collapse occurs the metastable soil structure changes due to the breakage of bonds and aggregations along the shearing plane.

Modified direct shear tests can be a useful tool to define in an experimental way the shear strength behavior of a metastable-structured soil. The shearing tests must be defined for the range of stresses expected in the earth structure under analyses.

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