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Compressive Strength for an Aggregated and Partially Saturated Soil

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ABSTRACT

Soil strength increases with the application of soil water suctions, which impart an increasing level of effective confining stress on the soil mass. The relationship between effective confining stress and suction is influenced by soil water content. For sands, effective confining stress is approximated by the multiple of soil water suction and total saturation of the soil pores. For aggregated soils this is not true. This study was undertaken to test the hypothesis that the saturation of interaggregate pores in aggregated soils controls the level of effective stress and, subsequently, soil strength relationships. The soil used is a highly aggregated, mixed mineralogy, Paulding clay (very-fine, illitic, nonacid, mesic Typic Haplaquept) from northwestern Ohio. Unconfined compressive strength was measured for five different aggregate size groups across a range of soil water suction levels. Triaxial compression strength was measured on saturated soil samples for which effective confining stress was well defined within the triaxial test cell. Results indicated that axial stress at failure for the soil was linearly related to the product of soil water suction and the saturation level of the interaggregate soil pores. Also, the slope of the regression line was statistically equivalent to the slope of the line relating axial failure stress and effective confining stresses in the saturated triaxial tests. The experimental results support the hypothesis that interaggregate pore water controls the effective confining stress level, and hence compressive strength, for aggregated soil.

SOIL STRENGTH changes as a function of soil water suctions and soil water content (Nearing et al., 1988; Formanek et al., 1984; Panwar and Siemens, 1972; Towner, 1961; Williams and Shaykewich, 1970; Gerard, 1965; Gill, 1959; Camp and Gill, 1969). The concept most often used to describe this phenomenon is *effective stress,* according to which the compressive strength of a soil is described as being linearly proportional to the amount of confining stress placed on the soil mass (Holtz and Kovacs, 1981). The effective stress theory is well established and widely used for the design and evaluation of the strength and stability of earth structures. It has also been applied successfully for the case of partially saturated soils, with the included consideration of the effects of negative soil water potential forces, which act essentially equivalent to externally applied confining stresses in terms of increasing soil strength (Bishop and Blight, 1963; Towner, 1961; Towner and Childs, 1972).

For the case of saturated soils under negative pore water potential, the internal soil pressure is less than atmospheric and the resultant confining pressure is equal to the magnitude of the difference between that internal to the soil mass and that of the atmosphere. A resultant linear increase in shear strength with increasing negative water potential has been observed (Towner, 1961), and is attributed to the concept that the effective confining stress, σ', is equal to the magnitude of the negative soil water potential, $-\psi$:

$$\sigma' = -\psi \quad [1]$$

However, under freely drained conditions, the soil loses water under negative pore water potential and the full effect of the increase in strength with decrease in potential is not realized. Towner and Childs (1972) and Nearing et al. (1988) showed that σ' is equal to a fraction of $-\psi$ according to

$$\sigma' = -\chi^{\psi} \quad [2]$$

where χ is the fraction of the water potential that effects an increase in soil strength. Equation [2] is a general form of Eq. [1] where χ has a value of 1. For the case of nonaggregated soil, such as a sand, χ is equivalent to the degree of saturation, S, of the soil (Towner and Childs, 1972). This empirical observation implies that for the case of sand essentially all of the water in the soil pore space, and its associated negative water pressure, contributes confining pressure between soil grains, which increases the friction and required stress necessary to cause a compressive type failure. However, for the highly aggregated Paulding soil studied by Nearing et al. (1988), the value of χ as determined by both strength and volume change tests was much less than S. Nearing et al. (1988) hypothesized that the reason for this difference was due to the distribution of water in the soil. If the plane of compressive failure is located between the soil aggregates, it would be logical that the interaggregate water would control the level of soil strength to a greater extent than the intraaggregate water. Experimental results of Nearing et al. (1988), which showed that the finely aggregated soil material behaved differently than did the soil material composed of larger aggregates, were consistent with this hypothesis. However, since they did not have data on the relative amounts of interaggregate and intraaggregate water, they could not perform any quantitative analysis of the theory.

The purpose of this study was to test the hypothesis that soil water potential and interaggregate water content determine the effective confining stress and hence compressive strength of an unsaturated, aggregated soil. In other words, the χ value in Eq. [2] was evaluated in terms of its relation to soil water potential and soil water content both in and between the soil aggregates.

METHODS

Soil

The soil used in this study is a Paulding clay from Defiance, OH, with 550 g kg^{-1} clay, 350 g kg^{-1} silt, and 100 g kg^{-1} sand. This soil was chosen because of the fact that it is strongly aggregated, and thus, well suited to test the hypothesis that interaggregate water and intraaggregate water act distinctly in terms of affecting soil strength. The soil was air dried and

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passed through a series of sieves to obtain four size groups for testing: 0.0 to 0.5, 0.5 to 1.0, 1.0 to 2.0, and 2.0 to 4.0 mm. A fifth aggregate group was used that consisted of equal parts by weight of the four size fractions. The soil was then wetted by spraying to a water content of 270 g kg^{-1} and allowed to equilibrate for 3 d before use.

Unconfined Compression Tests

Soils for the unconfined compression tests were formed under uniform static compression to a density of 1100 kg/m^3 within a brass cylinder of internal radius 19.0 mm and length of 75.8 mm. The samples were removed from the cylinder and encased in cylindrical nonporous latex membranes, similar to the procedure for preparing triaxial test specimens (U.S. Army Corps of Engineers, 1970). Porous stones, similar to those used in triaxial testing, were placed on either end of the sample to allow drainage into and out of the sample. The samples were then placed in a basin of water to allow them to satiate from the bottom for 3 d. The latex membrane prevented the samples from disintegrating while being satiated. The samples were then placed on either tension tables or pressure plates at 4, 16, 32, 64, and 96 kPa negative water pressure for a period of 4 d prior to testing. The 4-d period was chosen based on prior testing of this soil. After 4 d the water content did not continue to change. Each treatment was replicated three times.

Unconfined compressive strength was measured with an Instron Universal Testing Instrument[1] (Instron Corp., Canton, MA). Latex membranes were removed before testing. There was no measurable deformation of the samples due to disturbance caused by removal of the membranes. Diameters (taken as an average of measurement at the top, middle, and bottom sections of the samples) and lengths of the samples immediately prior to testing were measured with calipers in order to compute the sample volumes. The samples were compressed at a rate of 0.05 cm min^{-1} until maximum axial compression stress was achieved. The average failure stress occurred at 3.0% strain for the samples tested. Water contents of the each sample were measured after testing.

Triaxial Compression Tests

Triaxial compression tests were performed under saturated conditions to determine the relationship for this soil between compressive strength and effective stress induced by placing a positive confining stress around the sample. Two test series were conducted, one for the 0- to 0.5-mm material and one for the 2.0- to 4.0-mm material. These two sizes represented the extremes in aggregates, and there was little difference in triaxial results for the two materials. Thus it was considered unnecessary to test intermediate size material. Samples for triaxial testing were prepared similarly to those for the unconfined compression tests, and satiated in the triaxial cell for 24 h prior to testing. The testing methods of the U.S. Army Corps of Engineers (1970) were used for the triaxial testing. Commonly stress at 15% strain is used as the failure stress for triaxial tests. In order to compare the results of the triaxial tests to those of the unconfined compression tests, however, the failure stress for the triaxial test was considered to be that associated with 3.0% strain, which was the average failure strain for the unconfined compression tests.

[1] Trade names and company names, included for the benefit of the reader, do not imply endorsement or preferential treatment of the product listed by the USDA.

Water Content and Interaggregate Pore Size Determinations

Water content was measured for each sample after the unconfined compressive strength tests to determine the total water content, w_{tot}, of each sample. Water contents, w_{agg}, of each of the five aggregate groups were determined at each water pressure, i.e., at negative 4, 16, 32, 64, and 96 kPa, by placing a sample of the aggregates onto either the porous pressure plates or a porous stone placed on a tension table, and measuring the water content by oven drying after equilibration. As mentioned above, precise measurements of the sample dimensions allowed computation of the total sample volumes, V_{tot}, and densities prior to testing. The total mass of solids, M_s, used in each sample was the amount that was packed into each core, and thus was known.

The pore-size distribution of the aggregates was measured using the Hg intrusion method (Winslow and Diamond, 1970; Danielson and Sutherland, 1986). Samples were freeze-dried prior to testing. The pore size was measured on four replicate samples of the mixed aggregate material and the results were averaged (Fig. 1). Pore distribution was used to estimate the degree of pore saturation, S_a, within the aggregates at each of the water pressure levels used in the experiment. Calculated degree of saturation ranged from 95% for the samples at 96 kPa tension to full saturation for the samples at 4 kPa tension. The Hg intrusion test also provided information on the solids density, ρ_s, which averaged 2493 kg/m^3 for the four replicates. Water density, ρ_w, was assumed to be 1000 kg/m^3.

With measured or calculated information on w_{tot}, w_{agg}, V_{tot}, S_a, ρ_s, ρ_w, and M_s, volumes were calculated for each of the five phases of the aggregated soil: (i) volume of air in the intraaggregate pores, V_{Aag}, (ii) volume of air in the interaggregate pores, V_{Ap}, (iii) volume of water in the intraaggregate pores, V_{Wag}, (iv) volume of water in the interaggregate pores, V_{Wp}, and (v) volume of solids, V_s. Equations for calculating each phase volume are:

$$V_s = M_s/\rho_s \quad [3]$$

$$V_{Wag} = w_{agg} M_s/\rho_w \quad [4]$$

$$V_{Aag} = V_{Wag}(1/S_a - 1) \quad [5]$$

$$V_{Wp} = w_{tot}M_s/\rho_w - V_{Wag} \quad [6]$$

$$V_{Ap} = V_{tot} - V_s - V_{Wag} - V_{Aag} - V_{Wp} \quad [7]$$

These volumes, and from them the saturation of the interaggregate and intraaggregate pores, were calculated for each sample.

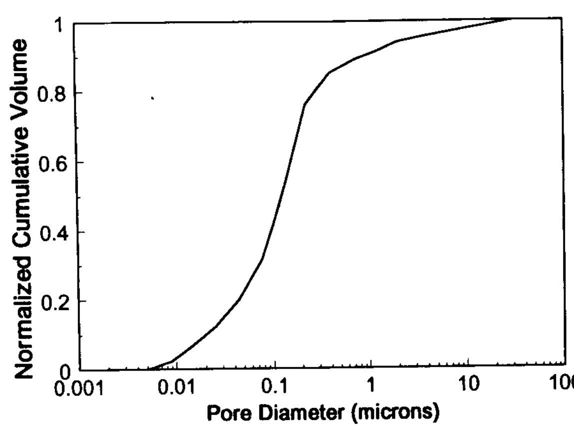

Fig. 1. Pore size distribution of the aggregates measured by the Hg intrusion method.

RESULTS AND DISCUSSION

Axial failure stress, *P*, was linearly correlated to the multiple of saturation of the interaggregate pores, S_p, and negative soil water pressure, $-\psi$, with a coefficient of determination of 0.87 (Fig. 2) and a slope of 1.25 kPa/kPa. The axial stress at 3.0% strain in the triaxial tests was linearly related to confining stress with a slope of 1.63 kPa/kPa for the 0.0- to 0.5-mm material and 1.69 kPa/kPa for the 2.0- to 4.0-mm material. This result is represented as a single line on Fig. 3, which is an average of the two triaxial test series. The slope of the line between axial failure stress and $-S_p\chi$ for the unconfined compression test was not significantly different ($\alpha = 0.05$) than the slope for the triaxial test. This result supports the hypothesis that for aggregated soils, the χ term in Eq. [2] is equivalent to the saturation of the interaggregate pores in the soil, rather than total soil saturation, which is the case for nonaggregated sands.

When either $\chi = 1$, which is representative of a saturated soil, or $\chi = S_{tot}$, which is representative of sands, is used for representing the proportion of negative pore water pressure that contributed to effective confining stress of a soil mass, the relationship between $-\chi^{\Psi}c$ and failure stress is both nonlinear and nonunique (Fig. 3). Also, the results for these two cases are very much different from the triaxial test results, for which effective confining stress is known accurately.

The results of this study explain the apparent anomoly observed by Nearing et al. (1988) in attempting to apply the concepts brought forth by Towner and Childs (1972) regarding effective stresses and shear strength in partially saturated soils. Towner and Childs could explain their shear strength results in terms of saturation levels and water pressure alone, while Nearing et al. (1988) found that: (i) the effective stress levels in the aggregated soil were apparently much lower than would be expected for similar water content levels in sands, and (ii) bulk soil compacted using different aggregate size material derived from the same initial material had different relationships between water content and strength. The concept proposed in this study explains how the results of Nearing et al. (1988) and Towner and Childs (1972) are consistent if one considers not the total saturation level of the aggregated soil bulk, but rather the saturation level of the interaggregate pore space in computing effective stresses. Also, for the soil material composed of smaller aggregates, the water content between aggregates would be greater at the same negative soil water pressure than for the larger aggregates, while water within the aggregates would be approximately equivalent. This explains the second observation of Nearing et al. (1988), that the relationship between total saturation and strength are different for the soil materials composed of the two different aggregate sizes. The basic result shown by Nearing et al. (1988) may be observed in our results in Fig. 3. The data points for the case of $\chi = S_{tot}$ do not show a unique relationship when plotted against axial failure stress. The data points for $\chi = S_p$, however, do fit a unique linear relationship with strength.

In conclusion, the results of this study indicated that the compressive strength of this aggregated, partially saturated soil could be explained adequately in terms of effective confining stress imparted to the soil mass by negative pore water potential, where the portion of the water pressure active in inducing effective stress was approximately equivalent to the saturation of interaggregate pores. It was the saturation of the pore space between

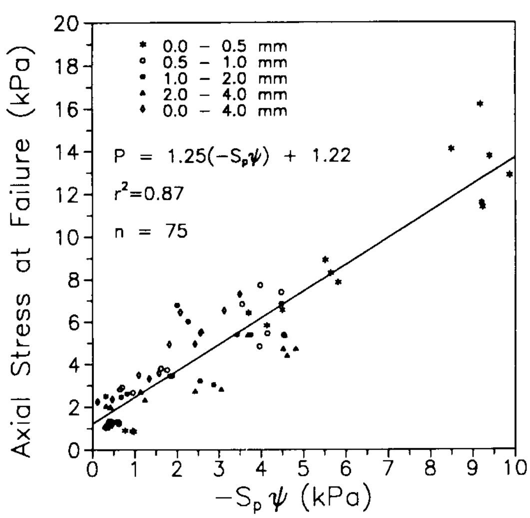

Fig. 2. Axial failure stress, *P*, of the unconfined compression tests vs. the negative soil water pressure, $-\psi$, multiplied by the saturation level of the interaggregate pore space in the soil for the five aggregate size groups, and the line of best fit through the data.

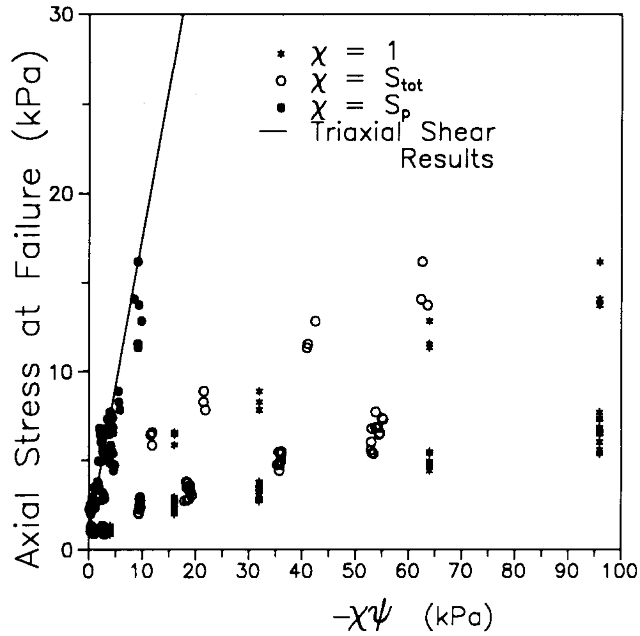

Fig. 3. Axial failure stress of the unconfined compression tests vs. the negative soil water pressure, $-\psi$, multiplied by various representations of χ, where χ is intended to be the fraction of the negative water potential that acts as a confining stress to increase shear strength. A χ value of 1 would be representative of a saturated soil, and a χ value equal to the soil saturation level would represent conditions for a pure sand. S_p is the saturation level of the interaggregate pores of an aggregated soil.

the aggregates that controlled the soil compressive strength, rather than the total soil pore saturation, as is the case with nonaggregated sand.

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