“Cohesive Soil”: A Dangerous Oxymoron[1]

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Note:
Located at URL
http://geosystems.gatech.edu/Faculty/Santamarina/General/Publications/Electronics/Dange_Oxy/Dangeoxi.zip

INTRODUCTION

Oxymorons or self-contradictions are often used in our daily lives. An oxymoron is also said to be "a wittily paradoxical turn of phrase which appeals to unconscious responses instead of rational examinations“ (Robertson, 1997). Consider for example, “organized chaos” and “incomplete solution”. Oxymorons often express a wish or an assumption, yet, they may also capture a misconception.

The geotechnical literature is not exempt from oxymorons. For example, analytical solutions for interparticle contact clearly show that soils and all particulate materials are inherently non-linear -Hertz theory- and non-elastic -Mindlin’s theory (See Richart, Hall and Woods, 1970; Cascante and Santamarina, 1996). Yet, “linear-elastic soil behavior” is a common expression. In most cases it refers to the model selected to interpret test results or to the assumption made in the design of geosystems subjected to small strains. For many reasons, linear elasticity has proven quite useful (insensitivity of the induced field of stress to material parameters and difficulty in gathering parameters for more sophisticated and complex constitutive models).

The purpose of this electronic note is to argue against the use of “cohesive soil”, probably the most pervasive oxymoron in the geotechnical field today. Indeed, the terms “cohesive soil” and “cohesionless soil” are almost equivalent to soil classification. The use of these terms creates a confusing framework for teaching purposes, and pre-sets the engineering mind with the wrong model of behavior. Ultimately, it leads to less reliable geotechnical systems when fine-grained soils are involved.

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SOURCE OF CONFUSION

Partial Saturation. The misleading observation that a ball of mud “holds together” can be readily explained by the negative pore pressure due to capillary forces and the consequent effective stress which conveys strength. This phenomenon is similar to a suction cup stuck onto a glass window. The magnitude of the interparticle force caused by the meniscus around the contact between two particles can be readily computed and compared with the weight of the particles (Mathcad file #1 - Equations can be found in Adamson, 1990). Results show the importance of partial saturation in the effective stress felt by the granular skeleton. The effective stress principle was modified to accommodate the effect of partial saturation (Bishop and Blight 1963; Fredlund and Rahardjo, 1993).

Undrained Shear. When the rate of loading exceeds the rate of pore-water pressure dissipation (controlled by the permeability k - see sequel), the dilative or contractive tendency of a soil does not manifest, and shear takes place at constant global volume. The volume-confinement-shear domain of soil behavior prescribes that shear at constant volume can only occur if the effective confinement changes, therefore, there will be pore pressure changes. To avoid the complexity of measuring and predicting pore pressure changes, the undrained shear strength is often used within the framework of a total stress analysis, where the dissimilar behavior of the fluid and the particulate skeleton are grouped together. The phi=0 concept was included in Terzaghi’s “Theoretical Soil Mechanics” published in 1943.

Dilatancy. The concept of dilatancy was understood by grain dealers and was studied by Reynolds before this century (in 1885 Reynolds demonstrated the concept with rubber bags filled with sand and water). Today, we know that the tendency of a soil to dilate decreases with increasing confinement. Hence, the envelope of failure states is curved. If the straight line Coulomb failure criterion y=A+Bx is imposed to the data, there will be fitting parameters for the intercept A and the slope B, which are valid for the stress range of the tests. Yet, it is unwarranted to call them cohesion and friction. In Taylor’s words, “It may well be claimed that it is poor policy to use the terms cohesion and friction angle in this empirical sense for the [fitting] coefficients A and B” (Taylor, 1948 - page 402). He goes further to say, “However, this procedure is in such common use that it must be accepted”. Today, this practice seems inappropriate.

Particle Eccentricity. The geometry of fine particles deviates from sphericity and approaches a plate-like geometry. Eccentric particles require a higher number of interparticle contacts to form a stable packing, and magnify the mechanical anisotropy of the medium when subjected to anisotropic loading, such as ko-preloading (Rothenburg and Bathurst, 1992). The dilatancy angle for shearing transversely to the depositional plane increases almost proportional to the slenderness of particles; in the extreme case of a stack of coins, the dilatancy angle is virtually 90 deg. The failure of particulate materials made of eccentric particles may show post peak behavior and shear banding even in loose specimens (Aloufi and Santamarina, 1995. Simple experiments with rice can readily help gain additional insight into the effects of particle eccentricity on global behavior).

Electrical Forces. In 1926, Goldschmidt showed that clays mixed with non-polar fluids result in a non-plastic mix. The DLVO theory (Derjaguin-Landau-Vervey-Overbeek) can be used to evaluate the balance between van der Waals attraction and double layer repulsion (Mathcad file #2. Equations can be found in Israelachvili, 1992; parameters and applications to soil behavior can be found in Mitchell, 1993, Santamarina and Fam 1995, and Fam and Santamarina, 1996). Attraction prevails when the interparticle distance is very small, i.e. high confinement, or when the ionic concentration of the pore fluid is very high. In general, for near surface soil deposits, it is reasonable to conclude that interparticle attraction cannot be the cause of any significant cohesion in the medium. (Born repulsion and hydration forces must be considered if particles are closer than ~20 Angstroms. Other complications may develop; for example, the edge charge of kaolinites becomes positive at low pH).

IMPLICATIONS TO ENGINEERING DESIGN

Curve fitting with an improper model leads to incorrect interpretation of results. Thus, it is not surprising that the coefficient of variation for “cohesion” is one of the largest values documented for geotechnical parameters (The coefficient of variation is the ratio of the standard deviation over the mean - Harr, 1987 lists a value of 0.4; see also Kulhawy, 1992, and Lacasse and Nadim, 1996). The immediate consequence of a high coefficient of variation is high probability of failure. Kezdi (1975 - page 210) reports the statistics of retaining wall failures compiled by H. O. Ireland. The data persuasively suggest the prevalence of failures when the retained fill and/or the foundation soil are clay. Unfortunately, no information is provided on the frequency of each type of wall which is needed to compute the historical probability of failure in each case.

Even if the material has some initial cementation (dry soils and soils undergoing diagenetic changes), compatibility of deformations and progressive failure dictate that large strain critical state soil parameters should be used in the analysis of ultimate capacity. Due to the brittleness of cementation, only friction remains in the critical state (see recommendations for design parameters in Lambe and Whitman 1969, Atkinson 1993, and Wood, 1990).

From a teaching point of view, let us stop presenting the three terms of bearing capacity equations together. Either a total stress analysis is conducted using the undrained shear strength Su within the Nc and Nq terms, or an effective stress analysis is used considering a frictional medium and the Nq and $N\gamma$ terms.

FINAL COMMENTS: FINENESS

The terms “cohesive soils” and the associated redundancy “cohesionless soil” should be avoided.

On the other hand, greater emphasis should be placed on fineness and specific surface. Indeed, the balance between gravimetric forces and surface-related forces is directly related to specific surface, for a given mineral composition.

Fineness controls capillarity, permeability, and the consequent generation of pore pressure relative to the rate of loading. The link between fineness and permeability was recognized by Hazen in 1911 (Note: Hazen’s equation $k=C.D10^2$ applies to sands).

Note that the importance of D10 and specific surface were well understood by Casagrande and other researchers during the early developments of classification systems such as the USCS. Indeed, permeability is the main reason why we are concerned with the percent of fines filling the voids in coarse-grained soils (The USCS distinguishes between <5% and >12% fines). On the other hand, the liquid limit test is excellent diagnostic procedure to assess the importance of surface related phenomena in soil behavior

Finally, the micro-level physics needed to interpret observed macro behavior also changes with fineness: if particles are fine, the model of “solid particles” and Newtonian interparticle forces does not apply anymore, and the interparticle behavior must be studied at the level of molecules and electrical forces.

Let us conclude with a few other oxymorons... The “common sense” behind “cohesive soils” biases the “perceived reality” so that the “known uncertainty” “almost always” becomes “pretty ugly”. Of course, this note is open to “constructive criticisms”...

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