SHEAR STRENGTH EVALUATION OF CLAY-ROCK MIXTURES

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ABSTRACT

At present, there is little knowledge concerning the shear strength of clays containing floating rock particles with concentrations from 0 to 30%. In practice, the effect of rock particles is typically disregarded in shear strength analysis. The two primary reasons for this are: 1) a lack of agreement concerning the influence of rock particles on material strength, and 2) the expense and difficulty of testing clay-rock mixtures with existing laboratory procedures. These factors have impeded the development of successful slope remediation design techniques for colluvium derived from resistant sedimentary rocks or spoil produced from surface mining. This study reviewed 31 technical papers which contain analysis of shear strengths for clay and sands with varying mixtures of rock particles. These technical papers, published over the last 40 years, are comprised of field case studies, laboratory evaluations, and theoretical analysis. Evaluation of this body of knowledge has shown that the shear strength gradually increases with increasing percentages of floating particles in unsaturated clays.

INTRODUCTION

One of the most important design input parameter needed for geotechnical design is soil's shear strength. The shear strength is commonly defined by the Mohr-Coulomb failure envelope:

$$\tau = \sigma_n \tan\phi + C \qquad (1)$$

where τ = Shear strength,
σ_n = Normal stress,

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ϕ = Angle of shearing resistance,
C = Cohesion.

The characteristic soil shear strength is defined by the angle of shearing resistance (ϕ) and the cohesion (C). For soils with varying percentages of rock particles, standard procedures for developing shear strength parameters are complicated and are not universally accepted in practice. In fact, examination of common design practices indicates that most slope design efforts disregard the rock particles when determining site-specific shear strength parameters for the soil (Iannacchione et al. 1994).

Soils derived from steep slopes in the central Appalachian Basin are often a clayey sand or a sandy clay. The percentage concentration of rock particles found in these slopes typically ranges from 10 to 30% (Iannacchione and Vallejo, 1995). This range produces rock particle arrangements which reduce contact to such a degree as to allow the oversized particles to "float" in the clay matrix.

Generally, a rock particle is considered oversized if it is normally discarded in a laboratory shear strength test. Numerous authors have discussed appropriate ratios among testing vessels and maximum particle size. Rathee (1981) examined several of these studies and found that recommended ranges of maximum particle size to testing vessel dimension varied from 1/5 to 1/40. Typical triaxial tests reported in most geotechnical investigations are 7.1 cm in diameter, the maximum particle size accommodated ranges from 0.8 cm to 1.4 cm. Therefore, particles within the gravel and above category are typically classified as oversized and discarded during standard laboratory testing, with smaller particles making up the soil matrix.

ROCK PARTICLES IN CONTACT

The strengths of cohesionless (granular) soils, whether wet or dry, are most dependent upon the frictional properties of the material (ϕ). Granular frictional properties are affected by the surface roughness and interlocking characteristics and the size, shape, and strength of the particles. Leps (1970) sought to explain these factors from the evaluations of a large database of triaxial tests on rock fill dam materials, and found a linear relationship between effective angle of shearing resistance (ϕ') and effective normal stress (σ_n'). These data showed that materials at low confining stresses have more strength than at high confining stresses. This was due to the dilation of the material at low effective normal stress and significant crushing of contact points with reduced dilation at high stress.

The effects of surface roughness were evaluated by Vallerga et al. (1957), with glass beads sheared under equal compactive effort. Beads with etched surfaces showed a considerable increase in the internal angle of friction. Conversely, low shear strength cohesionless soils are loose, with grains of round shape and a smooth surface. Density of these materials is affected by many factors, including gradation of the soil and confining stress.

The strength characteristics of rock particles in contact with their neighbors

have been extensively studied in conjunction with the widespread use of rock fills in dams and embankments in the 1960's and 70's (Marachi et al., 1972). Leslie (1963) reviewed a significant volume of artificially generated gradation relationships from gravelly soils and found that the highest values of the angle of shearing resistance were obtained from the densest sample with the largest maximum size particles. The authors also found that for any given porosity, the more uniform samples with smaller maximum sizes had higher values of the internal angle of friction. Marachi et al. (1972) observed that several factors influenced particle crushing, including: 1) increased water content, 2) increased uniformity, 3) increased angularity, 4) reduced particle strength, 5) increased effective confining pressure, 6) increased shear stress under a given confining pressure, 7) testing in a triaxial cell as compared to plane strain testing, and 8) increased particle sizes.

Marsal (1967) also observed that increased particle sizes reduced shear strength. However, Leussink (1965) disputed Marsal's testing approach and research conclusions, indicating that his studies had found a linear relationship between strength and porosity. In another study, Morgan and Harris (1967) concluded that there were no significant strength increases due to increased maximum particle size.

COHESIVE SOIL WITH ROCK PARTICLES

The strength of clayey soil is influenced by void ratio, composition, and angle of shearing resistance. The degree of saturation also plays a significant role in strength determination. The composition characteristics of cohesive soils are defined in terms of plasticity, where higher Plasticity generally yield lower angles of shearing resistance.

The earliest reference to laboratory-generated shear strength data from clay-rock mixtures was by Hall (1951). This study focused on the development of a triaxial apparatus for testing large soil specimens of at least 30.5 cm. Several specimens were tested which ranged from clayey sandy gravel to gravel. No conclusions were made concerning the influence of rock particles, but examination of the data clearly shows an increase in strength with increasing gravel content.

The influence of varying concentrations of rock particles on the shear strength of cohesive soil-rock mixtures was first investigated by Miller and Sowers (1957). These tests were carried out on consolidated, undrained triaxial specimens of remolded river sand and sandy clay from a decomposed gneiss. Sand versus clay mixtures ranged from 0 to 100%. Each specimen was compacted to its maximum dry density and optimum moisture contents. The experiments showed that increases in cohesionless material up to 67% had no effect upon the angle of shearing resistance but there was a gradual decrease in the cohesion of the sample (Figure 1). Between 67 and 74% cohesionless soil, the internal angle of friction increased and the cohesion decreased significantly. Beyond 74%, the internal angle of friction rose at a gradual rate. Miller and Sowers (1957) concluded that

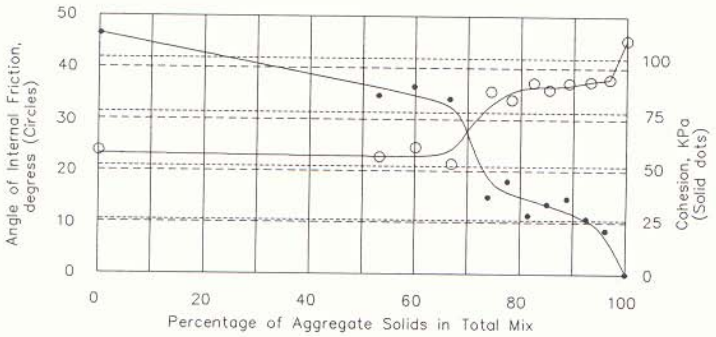

Figure 1. Relationship between aggregate percentage and angle of shearing resistance and cohesion for a cohesive soil (Miller and Sowers, 1957).

the dramatic changes in shear strength between 67 and 74% cohesionless material were a result of the granular structure controlling strength at the expense of a clayey matrix. Unfortunately, these tests were performed at relatively high effective normal stress (>200 kPa) and under optimum compaction conditions. The Mohr-Coulomb failure envelopes shown in Figure 2 indicate a reversal of trends at low confinements. However, this phenomenon seems unlikely and is probably more a result of the authors using a linear failure envelope than a physical reality.

Holtz and Ellis (1961) formulated a testing program to evaluate the effect of gravel content on shear strength for partially saturated materials containing particles up to 7.6 cm in size. This research found that shear strength did not

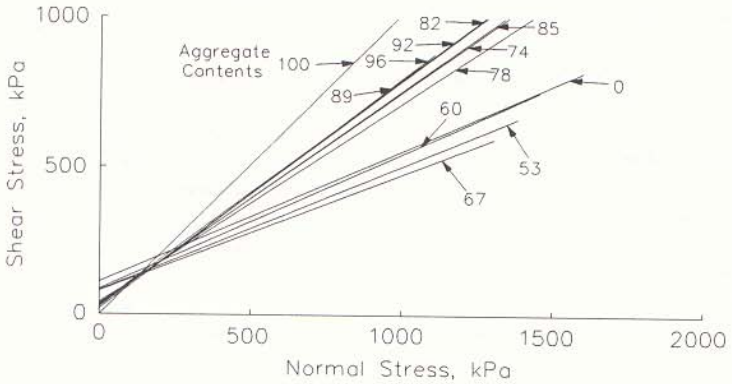

Figure 2. Shear strength data from Miller and Sowers (1957) showing the increase in angle of shearing resistance with increasing gravel content.

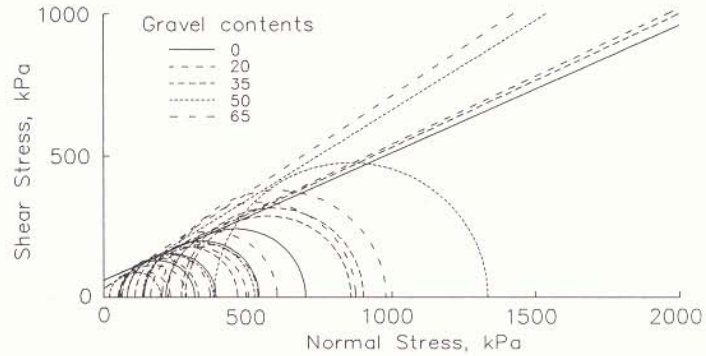

Figure 3. Shear strength data from Holtz and Ellis (1961) for cohesive soil with increasing gravel content.

significantly change for gravel contents up to about 35% (by weight). Beyond 35%, shear strengths increased significantly to about the 50% range (Figure 3). However, these data reveal an unrealistic reversal of the angle of shearing resistance at low normal stress conditions (<100 kPa).

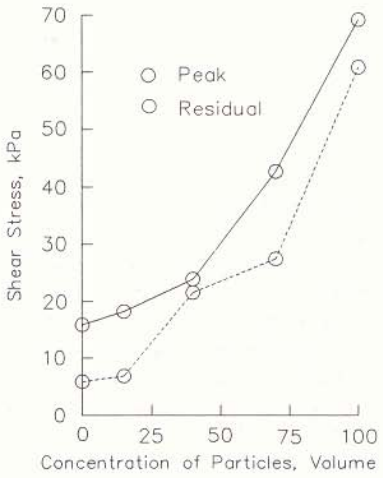

Figure 4. Shear stress values versus rock particle concentration for a clay-boulder/cobble mixture report by Patwardhan, et. al (1970).

Dobbiah et al. (1969) expanded on earlier work by examining the influence of maximum particle size on shear strength. The soil used in these tests contained mixtures of clay, silt, and sand. The authors found that increases in gravel sizes produced increases in shear strength. The density of the experimental clay-rock mixtures reached a maximum at approximately 50% gravel content, then decreased rapidly with increasing gravel concentrations. At this point, particle contact must have dominated due to the limited availability of clay within the available void spaces.

Patwardhan et al. (1970) found that considerable shearing

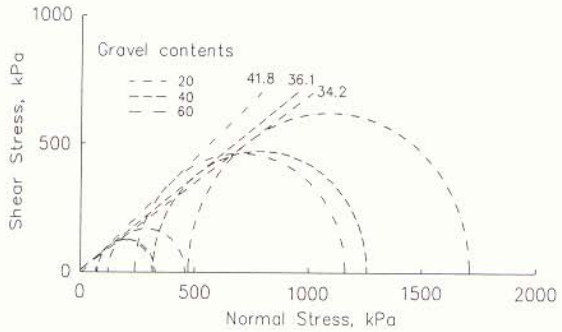

Figure 5. Shear strength data from a sand-clay soil with gravel size particles from Donaghe and Torrey (1979).

resistance was mobilized in assemblages of boulders/cobble-clay mixtures. Tests were performed in a 91-by-91-by-153 cm direct shear box using boulders with an average size of 15 cm. These test conditions produced a ratio (d_{max}/d) of about 1/6 on average, certainly close to the lower acceptable range for a direct shear test. Both the boulders and the clay came from a weathered basalt formation. Samples were saturated prior to testing and were loosely compacted, achieving pre-testing void ratios of 0.7 to 0.8. No vertical confinement was applied. The shearing resistances measured in this research were in the range of 8 to 70 kPa (Figure 4) and are much more representative of low consolidation stress conditions than the results reported above.

In another study, Rico and Orozco (1975) tested the reaction of varying concentrations of fines added to a sandy gravel material of mixed granitic and volcanic rocks. The fines material was taken from a commercial kaolinite and bentonite source and was classified as CL-ML material. Each sample was dynamically compacted at optimum water content and tested in the undrained state. In general, the undrained strength increased with increases in the fines up to about 5 to 10% depending on the type of matrix material, but then decreased sharply to a value below that of the aggregate-only soil.

The Army Corps of Engineers (Donaghe and Torrey, 1979) assessed the effects of both scalping and replacement methods (to be described later) on the shear strength of soil/rock mixtures. The tests were carried out as consolidated undrained triaxial tests on 38.1 cm specimens of gravel-sand-clay mixtures. The specimens were compacted to 95% of their standard compaction maximum dry density. Gravel sizes ranged from 0.4 to 7.6 cm and were tested at concentrations of 20, 40, and 60%. Here again, the effective angle of shearing resistance (ϕ') increased with increasing gravel contents (Figure 5).

Vallejo (1989) discussed the occurrence of large particles in rock fill dams, glacial tills, mud flows, debris flows, solifluction sheets, and residual soil deposits. Vallejo and Zhou (1994) examined consolidation and stability characteristics of simulated soil-rock mixtures. From testing mixtures of kaolinite clay with glass beads or sand, the author found that the percentage of the granular phase in the

mixtures has a marked influence on the compression index, coefficient of permeability, and shear strength of the mixtures. It was further found that when the volume of concentration of the sand varied between 80 and 100%, the shear strength of the mixtures was governed mainly by frictional resistance between the sand (Figure 6). When the concentration of sand varied between 50 and 80%, the shear strength of the mixture was provided in part by the shear strength of the kaolinite clay and in part by the frictional resistance between the sand grains. When the sand concentration was less than 50%, the shear strength of the mixture was entirely dictated by the strength of the clay.

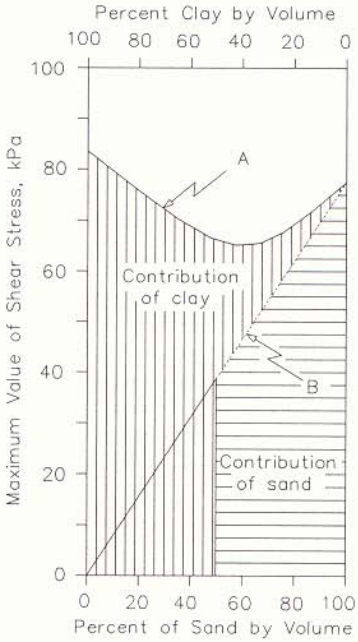

Figure 6. Shear strength of mixtures of clay and sand under a normal stress of 150 kPa (Vallejo and Zhou, 1994).

METHODS FOR DETERMINING SHEAR STRENGTH OF SOIL-ROCK MIXTURES

Several methods have been developed for determining the shear strengths of soil-rock mixtures. These methods can be grouped into four general categories: 1) Back-analysis, 2) Physical properties alteration, 3) Empirical, and 4) Analytical. The back-analysis method uses actual geometric properties of failed slopes to identify ranges of material properties which could produce these failures. The physical properties alteration method relies on adjustments to test samples to account for missing oversized rock particles. Analytical strength methods adjust the strength formulas, while empirical methods rely on past experience or large databases to assign shear strength parameters. A more detailed analysis of each method follows.

Back-analysis methods

The back-analysis method has the distinct advantage of being used with in

situ conditions. Geometric conditions--such as 1) slope angle, 2) material type, thickness, and density, and 3) location of failure plane--are known inputs into standard slope stability programs. Parametric studies are then performed by estimating the shear strength parameters and checking for a slope safety factor equal to one. This technique was demonstrated earlier to prove the inadequacy of laboratory-determined shear strength values in this study. The disadvantage of this method is the researcher's inability to 1) know the location of phreatic surfaces and, 2) deal with localized phenomena (changes in density, percentage of water, etc.).

Physical properties alteration methods

Three models of soil strength determination with the use of large particles have been proposed: 1) the parallel method; 2) the replacement method; and 3) the matrix method. The parallel modeling method of estimating the field properties of rock fill material was first suggested by Lowe (1964). In this method, specimens with parallel gradation are constructed with maximum particle sizes of 3.8 cm (1.5 in), adding fines to make up for the removal of oversized particles (Figure 7). Unfortunately, this method has proven to be unsatisfactory because of its failure to consider the shape, crushing, and surface roughness properties of the oversized material.

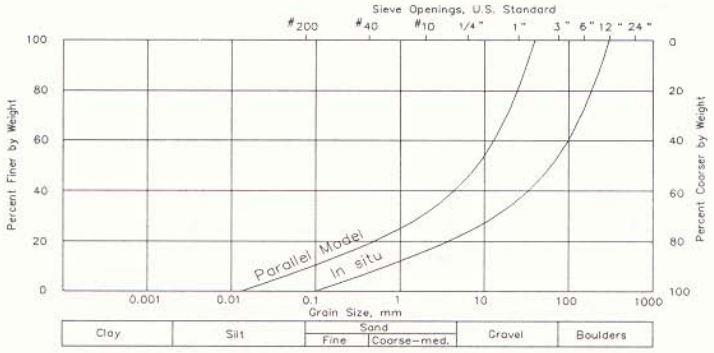

Figure 7. Parallel method illustrated with gradational analysis.

The replacement modeling method was introduced by the Army Corps of Engineers (1970) and suggests that particles larger than 1/6 of the triaxial test chamber's size be removed. If these particles compose more than 10% dry weight of sample, then an equal percentage of material retained on the #4 sieve but less than the maximum allowable sieve size should be introduced into the specimen (Figure 8). Donaghe and Torrey (1979) studied this process and found that the replacement procedures generally provided conservative strength parameters for earth-rock mixtures based on effective stresses.

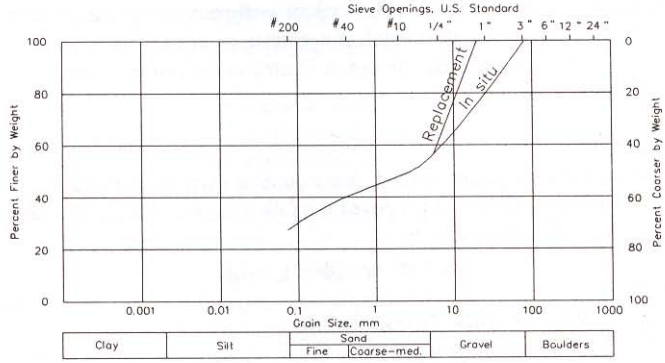

Figure 8. Replacement method illustrated with gradational analysis.

The matrix modeling method, introduced by Siddiqi (1984), removes the oversized particles from the specimen and examines the far-field soil matrix away from the particle. It is based upon the assumption that rock particles in a matrix of cohesionless material do not significantly affect the strength and deformation characteristics of the mixture. When less than 40% of the sample is composed of rock particles, there should be little contact among the particles. In this case, the far-field matrix contains a greater volume of material than the near-field; therefore it is the dominant strength member. Conversely, when the rock particles compose greater than say 65% of the sample, particle contact dominates. The soil matrix material simply fills voids created by the bridging action between non-floating particles.

More recent work by Su (1989) and Fragaszy et al. (1990) has shown that rock particles affect the density of the near-field matrix material. The authors determined that for state conditions in cohesionless material, the void ratio should increase around rock particles. This is a result of rock particles promoting void development based upon packing arrangements. The studies showed that average matrix density measurements lead to strengths that are too low. A method was proposed to determine the density of the far-field soil matrix, which according to the authors' findings controls the static strength of the material.

Empirical method

An interesting modeling technique has been proposed by Barton and Kjaernsli (1981) for estimating the shear strength of rockfill dams. This procedure requires the following input data: 1) the uniaxial compressive strength of the rock material, 2) the particle size (D_{50}), 3) the degree of particle roughness, 4) the porosity following compaction, and 5) normal stress of interest. This method serves to obtain preliminary estimates of the peak drained friction angle of rock fill,

whether it consists of angular quarried rock or well-rounded gravel. Although this example is restricted to soils with high concentrations of rock particles in contact and is specifically useful for dam design, it illustrates the character and practical nature of this approach.

Analytical method

As with the empirical method, few examples exist for the analytical method. Hencher et al. (1984) proposed the following formula to calculate the shear strength of boulder colluvium:

$$\tau' = \sigma' \tan (\phi' + i_m + i_f) \qquad (2)$$

where τ' = Effective shear stress,
σ' = Effective normal stress,
ϕ' = Effective angle of shearing resistance for the matrix (corrected for dilation),
i_m = Stress-dependent dilation angle for the matrix,
i_f = Dilation angle for the overall shear plane, taking boulder interference into account.

The values of the effective angle of shearing resistance (ϕ') and the stress-dependent dilation angle for the matrix (i_m) were obtained from the direct shear test results. The value of the dilation angle for the shear plane (i_f) was taken to be the averaged value of deviations of slip surface from the main direction of failure. Irfan and Tang (1993) indicated that this method gave an estimate of the upper-bound shear strength for the boulder colluvium in the Hong Kong area.

In a second example of the analytical methods, Vallejo (1979, 1989) examined the Skempton and DeLory (1957) approach for analysis of infinite slopes and found it inadequate for mud flows and debris flows that had a mixture of clay and rock lumps in a soft mud matrix. Vallejo examined the ratio of the volume occupied by the large particles and the volume of the whole mass, C, and determined the following relationships:

- C greater than 0.8 => frictional shear resistance between the large particles dominates,
- C less than 0.55 => shear strength for the soil dominates,
- C between 0.55 and 0.8 => shear strength of clayey matrix and frictional shear resistance of the large particles interact.

When the intermediate condition exists, Vallejo recommended that the following formula be used:

$$\tau = C\, \sigma \tan \phi + [1 - C]\, c \qquad (3)$$

where ϕ' is the effective angle of shearing resistance between the large particles and c_u is the undrained shear strength of the matrix (mud).

CURRENT THEORY EXPLAINING THE BEHAVIOR OF A COHESIVE SOIL-ROCK MIXTURE

The effect rock particles have on cohesive soil matrix systems is significantly different in relation to previously explained theories for cohesionless soil matrix systems. The major expression of this difference is found in the way stiff particles attract stress, alter strain patterns, and affect density. In a cohesionless soil matrix, a high concentration of rock particles (>40%) produces grain-to-grain contact and high angles of internal friction with little cohesion (Figure 9a). The addition of small amounts of cohesive soil matrix produces a shape drop in the angle of shearing resistance (Miller and Sowers, 1957) and a rapid increase in cohesion. This indicates that some of the clay is trapped between rock particles, preventing particle-to-particle contact (Figure 9b). The soil matrix between the rock particle contact points is highly compacted, while in other areas some open voids and grain-to-grain contact persists. As the clay content increases,

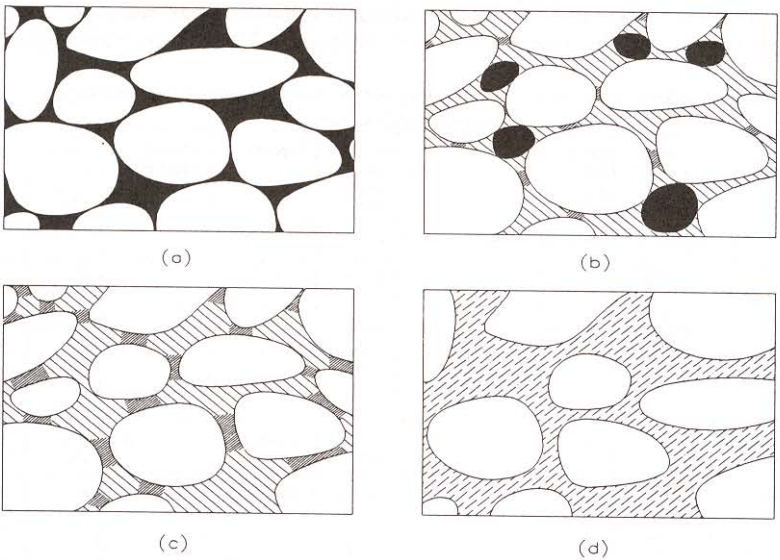

Figure 9. Compaction characteristics of soil with different aggregate mixtures (Miller and Sowers, 1957).

the cohesion increases, but at a decreasing rate. This reflects increasing clay compaction and a greater degree of void-filling by the clay (Figure 9c). At this point, there is a sufficient soil matrix to fill the voids loosely. At some point, enough clay exists in the system to cause the particles to float in the matrix of compacted clay (Figure 9d).

VARIATION IN CLAY SHEAR STRENGTH WITH CHANGING ROCK PARTICLE CONCENTRATIONS

Previously presented studies by Hall (1951), Miller and Sowers (1957), Holtz and Ellis (1961), Dobbiah et al. (1969), Patwardhan et al. (1970), and Donaghe and Torrey (1979) provided shear strength parameter data for varying concentrations of rock particles (Table 1). Most of the soil matrix material consisted of sandy clay with varying plasticity characteristics. Generally these test samples were wet but unsaturated material compacted close to maximum dry density. In many cases only limited tests were performed on particle concentrations where floating rocks would dominate (<40%). Confining pressures in the form of normal or lateral stresses ranged from 29 to 1379 kPa.

An evaluation of the effects rock particles have on cohesive soils in the unsaturated state provides some relevant data. Data contained within the above reports were utilized to evaluate the shear strength characteristics of material confined at approximately 200 kPa. This pressure simulates approximately 10 m of overburden. Examination of eastern Kentucky colluvium landslides showed that most failures occur between 5 and 15 m of overburden. However the Patwardhan

Table 1. Characteristics of previous laboratory test performed on cohesive soils with varying concentrations of rock particles.

Author(s), date	Soil matrix type	Distribution, C_c	Plastic Index, PI	Max. Particle size, cm	Compaction characteristic	Type of test	Moisture condition	Confining stress, kPa	Concentration of rock particles, %
Hall, 1951	Clayey (27) sand (73)	0.01	12	7.6	Close to MDD	Triaxial, CU	Unsaturated	104 to 414	53,85
Miller and Sowers, 1957	Sandy clay	0.01	6	0.5	Close to MDD	Triaxial, NA	Unsaturated	35 to 207	0,53,60, 67,74,78, 82,89,92, 96,100
Holtz and Ellis, 1961	SC-CL	0.8	28	7.6	Close to MDD	Triaxial, CU	Unsaturated	29 to 215	0,20,35, 50,65
Dobbiah et al. 1969	Clayey (36) sand (64)	0.6	17	2.5	Close to MDD	Triaxial, NA	Unsaturated	69 to 414	10,20,30, 40,50,60, 70,80
Patwardhan, et al. 1970	Clay	NA	7.3	15 (avg.)	Initial VR = 0.8	Direct Shear	Unsaturated	0	0,15,40, 70,100
Donaghe and Torrey, 1979	Clay and sand	0.6	21	7.6	Close to MDD	Triaxial, CU	Unsaturated	414 to 1379	20,40,60

Max. = Maximum dry density and optimum moisture content
UU = Unconsolidated-undrained
CU = Consolidated-undrained
CD = Consolidated-drained
MDD = Maximum dry density
VR = Void ratio
NA = Not Available

et al. (1970) data were not normalized because these tests were performed without confining pressure.

Figure 10 shows how shear strength of clay-rock mixtures is affected by varying rock particle concentration. In general, a gradual increase in strength is recognized as particle concentrations increase. In several of the tests, there is a marked increase in shear strength at a particle content of approximately 50%. This is undoubtably in response to significant particle-to- particle interaction occurring at and above this concentration. In general, high concentrations of rock particles (rock fills) have higher shear strengths. Leps (1970) examined 18 laboratory tests where the strengths of rock fills were determined. He found that the average rock fill had an internal angle of about 45° at 200 kPa normal pressure, which yields a shear strength of approximately 200 kPa.

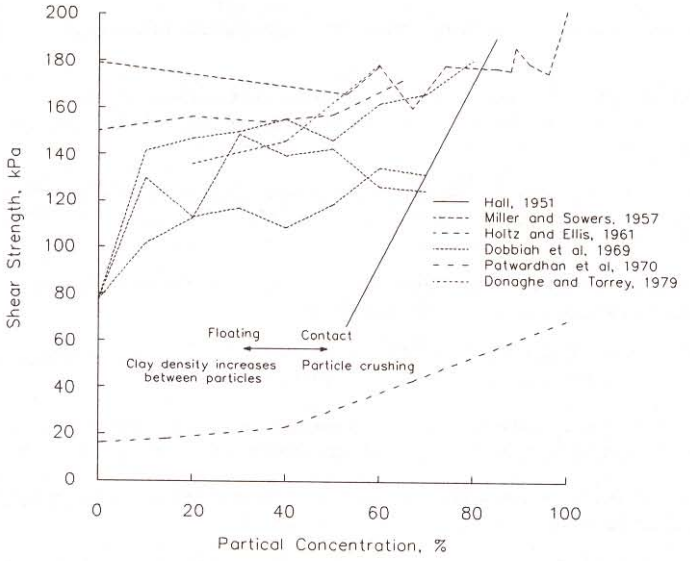

Figure 10. Relationship between shear strength and particle concentration for six past studies where the matrix material contained unsaturated clay.

SUMMARY AND CONCLUSION

This study has established that the shear strength of unsaturated clay can be affected by floating, oversized rock particles. This has significant practical implications because many colluvium soils have oversized particles within their matrix. Historically, design of slope remediation projects has been hampered by two factors: 1) an inability to test material with oversized particles, and 2) a lack of

knowledge about the relationship between particle concentration and shear strength. Miller and Sowers (1957) first proposed a theory to explain the changes in behavior of cohesive material as floating particle concentrations increased. In this theory, floating particles compacted the soil matrix between the rock particle contact points and also changed void ratios. Miller and Sowers (1957) did not indicate that a relationship between shear strength and floating rock particle concentration exists; however, this study found six laboratory cases where shear strength was shown to gradually increase with increasing floating particle concentrations. This implies that slope remediation efforts based solely upon shear strengths developed from standard laboratory tests, with all oversized particles removed, could produce conservative designs.

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