ON BEHAVIORAL THRESHOLDS IN MIXTURES OF SAND AND KAOLINITE CLAY

D.C. Simpson¹ and T.M. Evans², A.M.ASCE

ABSTRACT

Nearly all soils are comprised of mixtures of coarse and fine particles. Behavior under mechanical and thermal loading of soil is strongly influenced and in some cases, governed by the ratio of coarse to fine particles. A better understanding of the fundamental behavior of soil mixtures will provide insight to design decisions for new and emerging geotechnologies. In this work, behavioral threshold fines fractions were identified by experimental methods, where the threshold was defined as the point where changes in coarse/fine mixture ratio result in abrupt behavior changes. Binary mixtures of sand and kaolinite clay ranging from 0 to 100% fines content were subjected to consistency and undrained shear strength testing with the fall cone apparatus, compressibility tests using an oedometric cell, thermal conductivity tests with a thermal needle probe, and stress-strain-strength testing in undrained triaxial shear. Results indicate that behavioral thresholds exist at a critical fines content where a minimum void ratio occurs and at a percolation threshold where continuous force chains are present. The behavior changes are explained using theories of effective properties and percolation. Interpretations of these results lead to a clearer understanding of soil behavior.

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**INTRODUCTION**

Nearly all soils are mixtures of coarse and fine particles whose behavior under mechanical, thermal, and hydraulic loading is strongly influenced by the ratio of fine to coarse particles. Through understanding the physical cause of behavior changes, soil behavior can be robustly predicted. Work has been performed in the past investigating the influence of fines content on various behaviors of coarse-fine mixtures, but few researchers have performed a large suite of tests on a full range of mixture ratios.

The influence of mixture ratio on the behavior of soils has been shown to have significant effects on many different behavior types. Thevanayagam (1998), Yamamuro and Lade (1998), and Thevanayagam et al. (2002) showed that mixture ratio had measurable effects on undrained strength and static liquefaction susceptibility. Cubrinovski and Rees (2008) were able to show how mixture ratio affected the position of the critical state line and a mixture’s susceptibility to dynamic liquefaction. Pandian et al. (1995) were looking for optimal seepage barrier material and showed that the compressibility and conductivity of mixtures was strongly influenced by mixture ratio. These results comprise an important contribution to the literature, but the current work seeks to expand on these studies by considering mixing fractions from 0% to 100% fines using a comprehensive suite of laboratory tests, notably including thermal conductivity.

**REVIEW OF THEORETICAL BACKGROUND**

We define a behavioral threshold as a small change in mixture ratio which results in a significant change in behavior or response trend to some perturbation. Previously, generalized state parameters, effective properties, and percolation theory have been used to explain the behavior of particulate mixtures. These theories are generally based on what will be referred to as a four-phase...
soil model (Figure 1), which explicitly accounts for the presence of coarse and fine particles in a binary mixture.

Generalized state parameters attempt to describe how properties of binary mixtures change with fines fraction by considering the solid and void fractions of both mixture constituents. Several generalized state parameters have been proposed in the literature; for example, Westman and Hugill (1930) and Lade et al. (1998) demonstrate how apparent volume \( V_{\text{apparent}} = V_{\text{total}} / V_{\text{solids}} \) changes with fines fraction, as shown in Figure 2. Line segment GR in Figure 2 shows how the apparent volume (or void ratio) of a binary mixture decreases with increasing fines fraction. This behavior occurs because fine particles are filling the coarse-fraction void space without displacing coarse particles in a unit volume. Segment RF demonstrates how as the fines fraction increases beyond some critical fines content at point R, the apparent volume increases as fine particles displace coarse particles in a unit volume.

Yang et al. (2006) and Choo and Burns (2014) define a critical fines content \( f^* \) at point R as a function of constituent void ratio and specific gravity.

\[
f^* = \frac{e_c G_{sf}}{G_{sc} (1 + e_f) + e_c G_{sf}}
\]

where \( e_c \) is the void ratio of the coarse fraction, \( e_f \) is the void ratio of the fine fraction, \( G_{sf} \) is the specific gravity of the fine particles, and \( G_{sc} \) is the specific gravity of the coarse particles.

Generalized state parameters can be used to describe the behavior of a mixture by considering the state of one mixture component. For example, Thevanayagam (1998) explains undrained shear strength variation with changing silt content in mixtures of sand and silt by comparing the interfine void ratio \( e_f = e / fc \) where \( e \) is the global void ratio of the mixture and \( fc \)
is the fines content) and the intergranular void ratio \( e_s = \frac{e + f_c}{1 - f_c} \) to the minimum and maximum global void ratios of the mixtures.

Effective properties offer a mathematically robust way to bound or average behavior moduli (e.g. elastic modulus, thermal conductivity) of mixtures. The Hashin-Shtrikman variational approach (Hashin and Shtrikman 1963) can be used to determine upper and lower effective bounds on mixture moduli for mixtures of \( n \) constituents based on each constituent’s volume fraction and modulus. The self-consistent approach predicts an average response of a binary mixture by assuming that one constituent (the inclusions) is embedded in the other (the matrix) and that the matrix has the properties of the mixture. Self-consistency is the preferred method when interactions of inclusions are non-negligible or the volume fraction of inclusions is large (Guéguen and Palciauskas 1994). Solutions to the self-consistent set of equations are presented by Hill (1965) and Tarnawski et al. (2002).

Percolation theory may be used to describe the occurrence of particles with neighboring (directly adjacent) or nearest-neighboring (diagonally adjacent) particles in a continuum, forming clusters of connected particles. As the concentration of particles increases, the frequency and size of the clusters increases until a dominant system-wide cluster emerges. The concentration where the cluster size is as large as the system is the percolation threshold. Percolation theory can be used to determine if a system is macroscopically connected, e.g. if microscopic inclusions in a continuum are connected on the system-scale (Sahimi 1994). In soil mechanics, percolation theory has been used to describe the behavior of a binary system as the mixture ratio of coarse to fine particles changes (Peters and Berney 2009). Percolation of real three-dimensional systems of binary mixtures (such as coarse particles in a fine matrix) is not an exactly solved problem. Peters and Berney (2009) suggest that changes in observed volume change tendency in mixtures of sand.
and clay subjected to undrained triaxial shear were a result of sand fraction percolation. The transition observed by Peters and Berney (2009) occurred between 40 and 48% sand fraction, which roughly corresponds at an effective porosity $n_{eff} = 0.72 \left( V_T - V_{SS} \right) / V_T$ where $V_T$ is total volume and $V_{SS}$ is the volume of sand solids), which was reported as the percolation threshold by King et al. (2002) for rounded spheres in a continuum. 

Depending on the theoretical framework employed and the mixing ratio, the mineralogy of the fines fraction may or may not be predicted to affect the behavior of the mixture. For example, at high fines contents (i.e., when the sand grains can be considered as a dilute suspension within a clay matrix), self-consistency and effective properties theories both predict that clay mineralogy will impact system response. Percolation theory, on the other hand, implies that the only thing that matters is the continuity of the respective phases. With regards to behavioral thresholds, the four-phase model implies that only mixing fraction is important. In the current work, pure kaolinite was the only fine material considered. If different fines were used (e.g., silt, clay of a different composition), then clearly material behavior would be different than that observed here. What is less clear is whether changing the chemistry of the fines would change the location of the behavioral thresholds and thus, the results and conclusions presented below should be considered only to apply to mixtures of sand and kaolinite.

**MATERIAL AND MIXTURE PROPERTIES**

The coarse material used in this study was Ottawa 50/70 sand supplied by US Silica; this is a poorly-graded, rounded to well-rounded silica sand. The grain size distribution (GSD) of the sand was measured in accordance with ASTM D 422, using sieve numbers 40, 50, 60, 70, and 80.
and fit with a unimodal probability density function (Fredlund et al. 2000) to determine descriptive sizes (e.g. \( d_{10} \)). These results are presented in Figure 3.

Since the Ottawa 50/70 sand is very uniform, only a few mechanical sieves can be used to measure the GSD. To supplement the GSD information and confirm the shape of the fitted Fredlund et al. (2000) curve shown in Figure 3, microscope images of 24 individual sand grains were used. The sand grains were imaged at 200x magnification. The shape of each grain was approximated by a planar ellipse. The best-fit ellipse was determined by minimizing the Euclidian distance between the points defining the particle perimeter and a general quadratic curve (Fitzgibbon et al. 1999) constrained in such a manner to ensure that the resulting shape was elliptic rather than say, hyperbolic (cf. Weisstein 2014). From these ellipses, a volume can be determined by rotating the ellipses about their minor axis, forming an oblate spheroid. Using the measured specific gravity and the volume of the spheroids, mass fractions can be determined. The equivalent sieve opening for each ellipsoid is calculated as shown in Figure 4 and Equation 2, where \( r_1 \) and \( r_2 \) are semi-axis lengths. Figure 3 shows that the assumption of oblate spheroids agrees well with the best fit curve for the mechanically measured data, indicating that the sand grains are indeed similar in shape to slightly flattened spheres (i.e., oblate spheroids).

\[
Sieve \ size = \sqrt{2(r_1^2 + r_2^2)}
\]  

The fine material used in this study is kaolinite clay manufactured by Unimin Corporation marketed as “Sno-Brite Industrial Kaolin.” The measured properties of the clay are shown in Table 1.

For mixtures of Ottawa 50/70 sand and kaolinite clay, there exists a critical fines content specific to these materials where a minimum void ratio occurs because void spaces between the
sand particles are filled by clay without displacing sand particles from a unit volume (i.e. the
densest possible packing). The void ratio of the mixture may also be theoretically bound by
considering the minimum and maximum void ratios of the constituents; an upper bound is
formulated by considering maximum void ratios of both constituents, and a lower bound is
formulated by considering the minimum void ratios. Figure 5 demonstrates these bounds and
shows the critical fines content occurring at approximately 20% fines.

EXPERIMENTAL INVESTIGATION

Fall Cone Testing

The fall cone operates by allowing a cone of known geometry and weight placed at the
surface of a level soil specimen to free-fall. The penetration depth of the cone and water content
of the sample are measured and plotted with the water content on the ordinate and penetration
depth on the abscissa. Interpolation is used to determine the water content at a specific penetration
depth (depending on cone weight and geometry) corresponding to the liquid limit. Details of the
testing procedure can be found in BS 1377-2 (British Standards Institute 1990) and a detailed
discussion of the mechanics of the fall cone test is presented by Houlsby (1982).

Mixtures of sand and clay ranging from 0% to 100% fines were tested. First, consistency
testing was performed using a fall cone device customized with a linear variable differential
transformer (LVDT). The LVDT method allows for time-displacement data acquisition and the
precise determination of penetration depth after a specific desired time interval. The modification,
as described by Simpson (2014) and Evans and Simpson (2015), alters the falling mass of the cone
resulting in a redefinition of the liquid limit at 24.1 mm of penetration (cf. Wood and Wroth 1978).
All mixtures were tested using a light and a heavy cone to determine the liquid limit, plastic limit,
and plastic index (Wroth and Wood 1978). Specifically, Wood and Wroth (1978) provide a method to determine the plastic limit of a soil by performing fall cone tests using two different weight cones. Noting that for remolded soils the undrained shear strength at the plastic limit is 100 times the undrained shear strength at the liquid limit, they showed that:

\[
PI = \frac{2\Delta w}{\log_{10}(W_1/W_2)}
\]

where \(\Delta w\) is the mean distance between the two flow curves as measured on the water content \(w\) axis, and \(W_1\) and \(W_2\) are the weights of the two cones. Figure 6(a) shows the typical results of a fall cone test.

Feng (2000, 2001) proposed a different procedure for fall cone data interpretation where the water content and penetration are both plotted on log-scales. The liquid limit and plastic limit are then the water contents corresponding to 20 mm and 2 mm of penetration (for a 30° apex 80-g cone), respectively (Feng 2000, 2001). A typical flow curve used in the Feng (2000, 2001) method is shown in Figure 6(b). This formulation is also based on the fact that \(s_u^{PL} = 100s_u^{LL}\) (Wood and Wroth 1978), but requires extrapolation because tests are not performed at water contents that result in penetration depths as low as 2 mm. Equation 4 shows the Feng (2000, 2001) formulation of the water content-penetration depth relationship, where \(w\) is water content, \(c\) is the water content at penetration \(d = 1\) mm, and \(m\) is the slope of the power law line.

\[
\log w = \log c + m\log d
\]

Mixtures were prepared with four different water contents to bracket the assumed liquid limit, mixed thoroughly using a metal spatula, and allowed to hydrate for at least 16 hours prior to
testing. Testing procedures generally followed BS 1377-2 (British Standards Institute 1990) and the LVDT method was used for all mixtures (Evans and Simpson 2015; Simpson 2014).

Oedometric Testing

Consolidation testing was performed on all mixtures using a fixed-ring oedometer cell. Mixtures with more than 20% fines were mixed at a water content of approximately two times the liquid limit in a planetary mixer, preconsolidated to 100 kPa, and then trimmed from a large cake with the specimen ring. Mixtures with 20% fines or less were mixed wet (damp, without free liquids) using a spatula, spooned into the oedometer ring and preconsolidated to 100 kPa in the oedometer cell prior to load incrementing. Specimens were tested with a load increment ratio (LIR) of 0.25 up to a maximum vertical effective stress of 2314 kPa with one unload-reload cycle. Typical $e$-$\log(\sigma'_v)$ data are shown in Figure 7 for select mixture ratios.

Stress-Strain-Strength Testing

Consolidated undrained triaxial tests were performed on mixtures of 0, 10, 20, 30, 70, 80, and 100% fines. Specimens with 30% or less fines were dry-tamped in 6 approximately 25 mm lifts into a specimen mold with a dry drainage system (cf. ASTM D 7181), de-aired water was allowed to flow through the specimen under vacuum, and then specimen size was determined after 16 hours of curing. For mixtures with more than 30% fines, specimens were mixed and preconsolidated to 100 kPa as described for oedometric testing and then trimmed using a vertical specimen lathe. Specimens were saturated with backpressure and then consolidated to 150 kPa and sheared. The resulting stress paths are shown in Figure 8 with the critical state envelope bound by upper and lower bound critical state lines, $q_f = M \cdot p_f'$. 
Thermal Conductivity Testing

The thermal conductivity of the mixtures was measured using a needle probe (ASTM D 5334). Mixtures were prepared dry in a loose state by thoroughly mixing dry constituents in a mixing container and then air pluviating with a minimal fall height, similar to dry tubing. Mixtures were prepared dry in a dense state by tamping instead of pluviating, and in a wet state, prepared by mixing wet in a planetary mixer and tamping in equal lifts, referred to as “wet at the clay liquid limit” \( (W_{Lc}) \) where the water content, described by Equation 5, was such that the clay fraction was at the liquid limit for all mixtures:

\[
W_{Lc} = \frac{M_w}{M_s} = \frac{W_{L,kaol}M_c}{M_s} = fW_{L,kaol}
\]

where \( M_w \) is the mass of water, \( M_s \) is the mass of solids, \( W_{L,kaol} \) is the liquid limit of pure kaolinite \( (W_{L,kaol} = 49) \), \( M_c \) is the mass of clay solids, and \( f \) is the clay mass fraction. The typical thermal excitation applied to the specimens to measure conductivity (cf. ASTM D 5334) is shown in Figure 9.

DISCUSSION OF BEHAVIORAL THRESHOLDS

Experimental Results

Consistency testing results are presented in Figure 10 and indicate a transition to non-plastic behavior at approximately 20% fines. Non-plasticity below about 20% fines is indicated by the apparent increase in LL, also observed by Monkul and Ozden (2007) for non-plastic mixtures of sand and clay, and by significant scatter in the penetration-water content data, resulting in severely non-parallel flow curves or decreasing penetration with increasing water content for mixtures with less than 20% fines. Likos and Jaafar (2014) performed fall cone testing on dry,
partially, and fully saturated clean sands and found that the presence of suction stress caused by partial saturation reduced penetration, resulting in observed atypical flow curve behavior. Note that this is a particular hazard of using the fall cone to measure consistency limits – because bearing capacity (which is a function of suction stress) is actually being measured and then correlated to consistency, it is possible to, e.g., measure a ‘liquid limit’ for a clean sand. However, careful observation during testing and evaluation of the flow curves reveals an obvious transition from plastic to non-plastic behavior.

The liquid limit was calculated using the procedures outlined in BS 1377-2, Wood and Wroth (1978), and Feng (2000) and the maximum calculated difference across the three methods was 2.2%. Variation in the Wood and Wroth (1978) method for determining plastic limit and plasticity index is a result of using a cone weight ratio of \( R = \frac{W_1}{W_2} = 1.9 \) [see Equation 3, Wood and Wroth (1978) use \( R = 3 \)], which increases sensitivity of the calculated plastic index to inaccurate flow curves. As \( R \) decreases, the calculated \( PI \) increases for a constant \( \Delta w \). If the flow curves are not perfectly parallel, the average \( \Delta w \) may fluctuate irregularly, and the low \( R \) magnifies the fluctuation in calculated \( PI \).

The compression index, \( C_c \), was determined from the virgin compression load increments. The swell (recompression) index, \( C_s \), was determined from the unload-reload cycle. The compression index, shown in Figure 11 remains relatively unchanged until the fines content increases beyond about 20% because stress-strain response is dominated by the relatively rigid continuous network of coarse particles (cf. Evans and Valdes 2011). When the fines fraction exceeds the critical fines content (about 20% fines), the relatively compressible fine material begins displacing coarse particles, increasing the compressibility of the mixture. This continues until the coarse particles become so dilute in the fine matrix (at about 90% fines) that further
increase in fine material does not increase compressibility. Recompression behavior exhibits a
similar trend, but lags the compression index in fines content (see inset in Figure 11). We
hypothesize that this is a result of the virgin loading preferentially reorienting the soil mixture
fabric. This loading has squeezed some of the fine particles from between coarse particles, thus
forming a rigid sand skeleton and causing recompression behavior similar to lower fines content
mixtures. When fines content exceeds 30%, enough fine particles are present to prevent
development of a completely rigid sand skeleton, resulting in softer recompression behavior.

Either Taylor’s method (square root time) or Casagrande’s method (log time) is typically
used to determine the coefficient of consolidation, \( c_v \), from measured load-deflection data at a
given load increment. However, these methods are not robust because they rely on operator
judgment and they use an ad hoc approach to manually fit only a portion of the measured data. In
the current work, \( c_v \) for each mixture was calculated at every load increment by fitting Terzaghi’s
depth-averaged solution for the diffusion equation (e.g., Terzaghi et al. 1996) to the entire range
of measured time-deformation data at each loading stage:

\[
U = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp(-M^2 T_v) \tag{6}
\]

where \( U \) is average degree of consolidation, \( M = \pi(2m + 1)/2, T_v = c_v t/H^2 \), and \( H \) is the drainage
path length. While the sum in Equation 6 is infinite, precluding effective curve-fitting, the authors
note that the average difference between the calculated degrees of consolidation obtained by
summing the first 10 terms and those obtained by summing the first 10,000 terms is on the order
of 10^{-6} when input values representative of the current work are used for calculation (i.e., \( c_v = 1 \)
\text{mm}^2/\text{s}, H = 25 \text{ mm}, t = [0, 120] \text{ min}). Indeed, double-precision calculations can only discern the
difference between the two solutions at times less than approximately 35 s. The first 100 terms of
the summation were used for the measurements described herein. Figure 12 shows typical fits of measured data with the Terzaghi solution.

The average coefficient of consolidation, \( \langle c_v \rangle \), was calculated for each mixture by averaging the coefficients of consolidation measured on the virgin compression line of each mixture and is shown in Figure 13 as a function of fines fraction. Coefficient of consolidation has three well-defined behavioral regimes as a function of clay content: (i) at fines fractions of approximately 30\% or less, \( \langle c_v \rangle \) is essentially constant around 5 cm\(^2\)/min; (ii) at fines fractions of 60\% and above, \( \langle c_v \rangle \) is essentially constant at about 0.19 cm\(^2\)/min; and (iii) at fines fractions between 30\% and 60\%, \( \langle c_v \rangle \) exhibits a decreasing behavior.

The coefficient of consolidation quantifies complex hydromechanical behavior that is a function of both the solid skeleton and the void space geometry. Clearly, both the soil skeleton and the void space topology at a given stress state are strongly dependent upon mixture ratio, so to better understand the behavior observed in Figure 13, the mechanical and hydraulic response are decoupled using Terzaghi’s classical definition of the coefficient of consolidation, shown as Equation 7:

\[
c_v = \frac{k(1 + e_0)}{\gamma_w a_v}
\]  

(7)

where \( k \) is the hydraulic conductivity, \( e_0 \) is the void ratio at the beginning of a given load increment, and \( a_v \) is the coefficient of compressibility.

The coefficient of compressibility was determined by first fitting a logarithmic equation (Equation 8) to the \( e-\log(\sigma'_v) \) data (omitting the unload-reload cycle):
\[ f(\sigma'_v, \bar{x}) = x_1 \ln(\sigma'_v + x_2) + x_3 \] (8)

where \( \sigma'_v \) is applied vertical effective stress and \( \bar{x} \) are the fitting parameters. See Figure 14 for typical function fits. The slope of the \( e - \log(\sigma'_v) \) curve may be determined at any point by using the derivative of Equation 8:

\[ \frac{df(\sigma'_v, \bar{x})}{d\sigma'_v} = \frac{x_1}{\sigma'_v + x_2} \] (9)

Typical results for select load increments are shown in Figure 15. By inspection of Figure 15, it can be seen that the coefficient of compressibility follows the same trend as the compression index: relatively unchanged until a critical fines content, then a steady increase with increasing fines until a dilute suspension of coarse particles is reached.

The hydraulic conductivities of the mixtures can be calculated from the measured coefficients of consolidation and compressibility using Equation 7 and are shown in Figure 16 for three select load increments. The calculated hydraulic conductivity generally exhibits the same binary behavior as the coefficient of consolidation, implying that the same physical mechanism is causing the behavioral transition.

Hydraulic conductivity is inferred from the consolidation data rather than being explicitly measured. Hydraulic conductivity inferred from consolidation testing is historically unreliable (Olson 1985), but is still useful for identifying behavioral trends. Additionally, insight may be gained from considering the measured thermal conductivity of the mixtures. Both conductivity phenomena are functions of the soil fabric. At low fines content, hydraulic conductivity increases as pores become more interconnected and create flow paths (Beven and Germann 1982). Interconnected pores develop at low fines content when mixtures are dominated by the coarse
fraction. Interconnected pores also result in larger interconnected contact areas. Previous researchers (Yun and Santamarina 2008; Evans et al 2011) show that greater contact area results in higher thermal conductivity, which explains the similar behavior between measured thermal and inferred hydraulic conductivity. Figure 17 shows that the conductivity of the dry loose specimens exhibits an asymptotic response to changes in fines. We hypothesize that this is a result of the use of air pluviation for specimen preparation. Building clayey specimens with air pluviation creates highly flocculated specimens dominated by a highly porous matrix, which is why the conductivity approaches the conductivity of air ($\lambda_{\text{air}} \approx 0.03 \text{ W/mK}^{-1}$) for increasing clay fractions. The conductivity of the dry dense mixtures is roughly characterized by a binary response, with a wider transition range from 40-70% fines. Thermal conductivity is influenced by nearest-neighbor and next-nearest-neighbor percolation (Ewing and Horton 2007), resulting in the behavior transition occurring over a wider range. The wet specimens exhibit similar behavior, but are influenced by the presence of water. The abrupt change in conductivity from 10% to 20% fines likely corresponds to the addition of enough water to form capillary bridges (Ewing and Horton 2007).

Critical state friction angles determined from consolidated undrained (CU) triaxial tests are presented in Figure 18. Mixture shear strength exhibits an approximately binary trend, similar to that observed for thermal and hydraulic conductivities. In this case, we hypothesize that the marked decrease in shear strength with increasing fines fraction is due to a breakdown of the coarse particle skeleton as fine materials begin to relieve the normal forces at coarse particle contacts.

**On Behavioral Thresholds**

The experimental results indicate that three behavior thresholds exist, which are summarized in Table 2. The observed thresholds depend on the behavior in question, and the
applicable behaviors to each threshold are also summarized in Table 2. Observed thresholds
demarcate four behavior regimes, summarized in Table 3.

Behavioral thresholds depend on the behavior in question, and can be explained physically
with existing mixture behavior theories. Behavior threshold $t^{*1}$ is synonymous with the critical
fines content, and demarcates a transition from a state where all coarse particles are touching each
other (coarse percolated) and fine particles are confined to the interstitials of the coarse particles
up until a state where clay begins displacing sand particles (coarse and fine particles). These
behavior regimes are manifested in consistency and compressibility behavior because both
consistency and compressibility depend on the availability for either rigid coarse particles to
interact or active fine particles to interact. Compressibility and consistency have a long history of
interaction in the literature (e.g. Skempton and Jones 1944), so their consistent behavior is not
surprising.

The transition from coarse and fine percolation to only fine percolation behavior occurs at
$t^{*2}$, corresponding to the percolation threshold of coarse material in the fine matrix. This fines
content is not well known, but the percolation threshold estimation by Peters and Berney (2009)
corresponds to a fines content range of about 40 – 60% depending on the density of the mixtures
for mixture constituents considered here. In this study, $t^{*2}$ affects critical state strength parameters
and hydraulic and thermal conductivities of the mixtures. This behavior threshold can be explained
by percolation because critical state strength and conductivity phenomena depend on the presence
of system-wide connected coarse particle clusters. Coarse particle clusters create force chains
which affect the critical state strength (Rechenmacher et al. 2010), so higher critical state strength
parameters at lower fines content are a result of coarse particle percolation. Coarse particle clusters
also create interconnected pores (Beven and Germann 1982) and large interconnected contact areas, resulting in higher conductivities at lower fines content.

Compressibility behavior depends on the availability of the relatively compressible fine particles. As fines contents increase beyond $t_{*1}$ and for strain levels observed in typical one-dimensional compression, decreasing coarse particles has little effect on compressibility. Coarse particles are suspended in the fine particle matrix and unable to contact one another and provide rigidity to the overall soil fabric.

**SUMMARY AND CONCLUSIONS**

Percolation theory and generalized state parameters are used to explain observed behavioral thresholds in mixtures of sand and clay. Compressibility behavior evaluated by oedometric loading and consistency behavior measured with the fall cone device show a behavior threshold at $t_{*1}$, the critical fines content. This behavior threshold is a result of mixtures transitioning to a regime where both fine and coarse material are percolated. Hydraulic conductivity calculated from oedometric loading, thermal conductivity measured with a needle probe, and critical state strength determined from undrained triaxial loading imply a behavioral threshold at $t_{*2}$, the percolation threshold of the coarse fraction. This behavioral threshold occurs when mixtures transition from a state with system-wide coarse particle clusters to a state without system-wide clusters of coarse particles.

This research has shown that for a given mixture ratio of coarse and fine particles, behavior can be considered “sand-like” or “clay-like” depending on the behavior in consideration. Furthermore, this research has also shown that altering a binary mixture ratio of coarse and fine particles may produce little to no change in observed behavior. An understanding of these
behavioral thresholds by practitioners and researchers alike may allow for more robust predictions of soil behavior in the absence of more comprehensive data, such as during the early stages of projects when a full laboratory material characterization has not yet been performed.

ACKNOWLEDGEMENTS

The Oregon State University School of Civil and Construction Engineering provided financial support for the first author during the course of this research. Many of the thermal conductivity measurements were performed by Vicente Coria. The authors gratefully acknowledge this assistance and support.
Table 1 Properties of kaolinite clay.

<table>
<thead>
<tr>
<th>Specific Gravity, $G_s^*$</th>
<th>Liquid Limit$^+$</th>
<th>Plastic Limit$^+$</th>
<th>Specific Surface Area, $S_a$ (m$^2$/g)$^\dagger$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.60</td>
<td>49</td>
<td>24</td>
<td>10 – 20</td>
</tr>
</tbody>
</table>

$^*$ASTM D 854

$^+$BS 1377-2

$^\dagger$After Mitchell and Soga (2005)

Table 2 Observed behavioral thresholds

<table>
<thead>
<tr>
<th>Threshold</th>
<th>$t^{*1}$</th>
<th>$t^{*2}$</th>
<th>$t^{*3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approximate fines content</td>
<td>0.2</td>
<td>0.4 – 0.6</td>
<td>0.9</td>
</tr>
<tr>
<td>Applicable Behaviors</td>
<td>Consistency, Compressibility</td>
<td>Critical State Strength, Conductivity</td>
<td>Compressibility</td>
</tr>
</tbody>
</table>

Table 3 Behavior Regimes

<table>
<thead>
<tr>
<th>Behavior Regime</th>
<th>Coarse Percolated</th>
<th>Coarse and Fines percolated</th>
<th>Fines Percolated</th>
<th>Dilute Suspension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applicable fines content $f$</td>
<td>$0 \leq f \leq t^{*1}$</td>
<td>$t^{*1} &lt; f \leq t^{*2}$</td>
<td>$t^{*2} &lt; f \leq t^{*3}$</td>
<td>$t^{*3} &lt; f \leq 1$</td>
</tr>
</tbody>
</table>
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limit of soils.” Ground Engineering, 11(3).


Granular Matter, 10(3), 197–207.
FIGURE CAPTIONS

Figure 1. The four-phase soil model.

Figure 2. Apparent Volume [after Westman and Hugill (1930) and Lade et al. (1998)]

Figure 3. Grain size distribution of Ottawa 50/70 sand; oblate spheroid data points are approximated grain sizes based on optical microscopy. *After Hryciw and Thomann (1993)
†ASTM D 854

Figure 4. Conceptualization of obtaining sieve sizes from planar ellipses fit to irregular particle shape

Figure 5. Void ratio bound and the critical fines content

Figure 6. Flow curves for 50% clay mixtures (a) demonstrates Wood and Wroth (1978) method with two different weight cones and (b) demonstrates Feng (2000, 2001) method. *Penetration depths corresponding to LL and PL defined for the LVDT Method (Evans and Simpson, 2015).

Figure 7. Typical oedometric stress-strain response for select mixtures: 100%, 60%, and 30% fines

Figure 8. Triaxial stress paths with inferred steady-state stress ratio envelope shown, where $M = q/p_f'$, the slope of the critical state line.

Figure 9. Typical thermal excitation.

Figure 10. Measured liquid and plastic limits; non-plastic mixtures are identified as mixtures with less than approximately 20% fines.

Figure 11. Compression and recompression indices for the soil mixtures.

Figure 12. Typical fits of Terzaghi’s diffusion equation for determination of $c_v$.

Figure 13. Variation of average coefficient of consolidation with fines fraction in the mixture. Dashed line indicates inferred behavioral trend.
Figure 14. Typical logarithmic fits for determining coefficient of compressibility for 100%, 70%, and 40% fines

Figure 15. Coefficient of compressibility for two select load increments

Figure 16. Hydraulic conductivity for three select load increments; dashed line indicates inferred behavioral trend. Permeability normalized to calculated value at $\sigma'_v = 2314$ kPa and $f = 1.0$.

Figure 17. Thermal conductivity measurements of (a) dry and (b) wet sand-clay mixtures. (Note change of vertical scale between plots.)

Figure 18. Critical state friction angle. Inferred behavioral trend indicated by dashed line.
Particle size (mm)

Fraction passing

Fredlund et al. (2000)
Oblate Spheroid

emin = 0.48*
emax = 0.71*
C_i = 1.07
C_c = 1.02
Gs = 2.65†

d_{60} = 0.266 mm
d_{30} = 0.259 mm
d_{50} = 0.264 mm
d_{10} = 0.248 mm

Click here to download Figure: Figure_03.pdf
Particle
Planar ellipse
Sieve opening

Major semi-axis, \( r_1 \)
Minor semi-axis, \( r_2 \)

Particle
Planar ellipse
Sieve opening

Click here to download Figure: Figure_04.pdf
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Inferred region of non-plasticity
Clay Fraction $f$

Index

$C_c$ / $C_r$

$0 \quad 0.1 \quad 0.2 \quad 0.3 \quad 0.4 \quad 0.5 \quad 0.6 \quad 0.7 \quad 0.8 \quad 0.9 \quad 1$

$0 \quad 0.05 \quad 0.1 \quad 0.15 \quad 0.2 \quad 0.25 \quad 0.3 \quad 0.35$

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$f = 1.0, \sigma_v' = 48 \text{ kPa}$

$f = 0.7, \sigma_v' = 116 \text{ kPa}$

$f = 0.4, \sigma_v' = 75 \text{ kPa}$
Vertical Effective Stress $\sigma_v$ (kPa)

Void Ratio $e$

$0.2 \quad 0.3 \quad 0.4 \quad 0.5 \quad 0.6 \quad 0.7 \quad 0.8 \quad 0.9 \quad 1$

$f = 1$
$f = 0.7$
$f = 0.4$

Logarithmic Fits

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Clay Fraction $f$

$a_v (\text{MPa})$

$
\sigma_v' = 554 \text{kPa} \\
\sigma_v' = 2314 \text{kPa}
$
Clay Fraction $f$

Normalized Permeability

$\sigma_v' = 31$ kPa
$\sigma_v' = 554$ kPa
$\sigma_v' = 2314$ kPa

Inferred Trend