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Landslide Risk Evaluation: the Mechanical Properties of Soils Sheared Undrained in a Ring Shear Apparatus

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Synopsis

The mechanism of flow slides initiation and motion is investigated. Results of the investigation show that there is a threshold state demarcating sands that contract from those that dilate. The possibility of specimens exhibiting either contractive or dilative behaviors seems to depend on whether or not the threshold pore pressure is exceeded. Results also show that whereas peak strengths are ranked as well graded > intermediately graded > narrowly graded > gap graded, steady state strengths are ranked as narrowly graded > intermediately graded > well graded > gap graded for specimens in medium and dense states.

Keywords: flow slides, friction angle, threshold state, grading, shear strength

1. Introduction

Planners and managers often desire information about hazards on a site before they can approve development. They also desire information about disasters as a way of preventing reoccurrence. Sustainable and efficient land use and development; and minimizing loss of life and property from natural disasters are the overriding reasons for this desire. One of the most important pieces of information these planners and managers want is geotechnical study related to the stability or instability of the area, and other secondary seismic hazards such as landslide, flow slide, lateral spreading and settlement.

Effective tackling of landslides, debris flow and other phenomena associated with slope failures requires constant advances in, among other things, experimental research and concepts. One area where such advances are necessary is in the prediction of flow slide initiation and mobility. The concepts of critical void ratio, steady state of deformation, phase transformation, and sliding-surface liquefaction have all shed important light on the behavior and geotechnical significance of materials associated with flow slides. The friction angle at steady state (ϕss) is the key parameter for predicting the velocity and travel distance of landslides. Accurate prediction or proper understanding of initiation and mobility of flow slides would depend on the accuracy and purity of data upon which predictions are based. The measurement of the friction angle at steady state, for instance, is often difficult and prone to serious errors. Such difficulties and their attendant errors may hinder accurate risk assessment of natural and artificial slopes. This paper introduces a new method of gaining knowledge of the friction angle at steady state of sands and explains how such knowledge could be effectively and economically gained. The new method is less prone to error, convenient, effective, and economical.

2. Method and material

2.1 Method

A newly-developed ring shear apparatus was used to simulate the initiation and mobility of flow slides. The apparatus is structured to eliminate some difficulties commonly encountered while studying the mechanism of landslide motion, and sufficiently equipped to allow speed-, and stress- controlled tests; and the measurement of very large shear displacement. The target is to determine the friction angle at a certain threshold state, the friction angle at the phase transformation, and the friction angle at the steady state of deformation using sands of varying grading. The friction angles at all three states would then be compared and assessed. The idea is to understand the mechanism of flow slide initiation and mobility and the factors that affect them by examining the relationship between these friction angles and grading.

2.2 Material

Industrial sand materials composed of sub-angular to angular quartz and small amount of feldspar were reconstituted to three uniformity coefficients – 3.3, 4.5, 9.0 and 17.5 – referred to as narrowly graded (NAG), intermediately graded (ING), well graded (WG) and gap graded (GAG).

To achieve the authors' target, a threshold state will have to be defined. The authors chose to define a threshold state as that where pore pressure at failure is equal to the shear resistance at failure such that pore pressure parameter Arf = 1 where Arf = $\Delta u/\Delta\tau$, Δu is change in pore pressure at failure while $\Delta\tau$ is change in shear resistance at failure. Soils with Arf = 1 will define the boundary between soils that undergo purely contractive behavior and those that undergo dilation, if all the soils are under the same effective normal stress. A soil that undergoes purely contractive behavior has no phase transformation line, only failure line; its Arf should be greater than unity. A soil that dilates has distinct failure line and phase transformation line located at two different points on the stress path; its Arf should be less than unity. A soil at threshold state has both lines (failure and transformation lines) located at the same point on the stress path; its Arf is unity. The objective is to determine this threshold state by carefully (starting from dense) decreasing the relative density of soils held under the same effective normal stress until loose state is attained.

3. Analysis

3.1 The boundary between contraction and dilation

Fig. 1 shows the relationship between the ratio $\Delta u/\Delta\tau$ at failure and normal stress. It may be seen that the specimens with $\Delta u/\Delta\tau$ = 1 forms a boundary between purely contractive specimens whose $\Delta u/\Delta\tau$ >1, and dilative specimens whose $\Delta u/\Delta\tau$ <1.

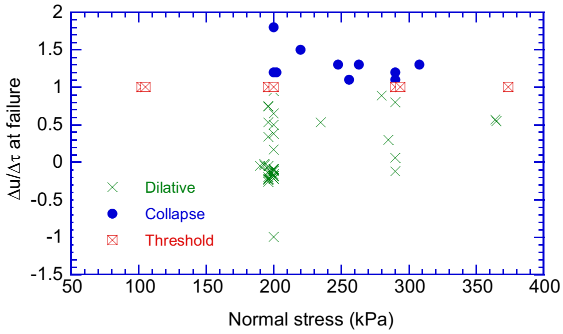

Fig. 1 The boundary between contraction and dilation

On the basis of presented evidence, a transition route for the specimens is proposed in Fig. 2. Starting from dense, Fig. 2a has a distinct peak failure and phase transformation lines. A systematic decrease in density will lead to a less dense specimen with the PT line and peak failure line getting closer to each other for every decrease in density as may be seen in Fig. 2 b. As density keeps reducing a time should come when peak failure line and PT will coincide; the specimen will experience the least dilation possible at the given effective normal stress as in Fig. 2c. The value of pore pressure (about 81 kPa) in Fig. 2c is known in this paper as threshold pore pressure. Attempts to exceed the threshold pore pressure should lead to collapse and flow behavior. Such soils will have only Mohr Coulomb failure line because there will be no phase transformation point. A direct transition from dilative behavior to a purely contractive behavior (a to c in Fig. 2) is thought to be incompatible with the present concept.

3.2 Friction angles compared

The friction angles at threshold, phase transformation, steady state, and peak are compared for the sands investigated. Fig. 3(a) shows the relationship between friction angle at phase transformation (PT) and relative density. It may be seen from the figure that although the friction angle at phase transformation (PT) might be slightly affected by relative density, all data are within 36.2° ± 0.7°. Therefore, it can be approximated to be a constant at around 36.2° in practical usage. Fig. 3(b) shows the relationship between friction angle at threshold state and relative density; where the values are in the range of 36.1° ± 0.4°. Fig. 3(c) is the relationship between friction angles at steady state (SS) and relative density. The values appear to be independent of both grading and relative density. The range of values are within 36.2° ± 0.7°

The relationship between friction angles at steady state, phase transformation, threshold state and relative density for all grading was analyzed. All values of the three friction angles (ϕ at PT, ϕ at SS, ϕ at threshold) are close to one another and exist within 36.2° ± 0.7°.

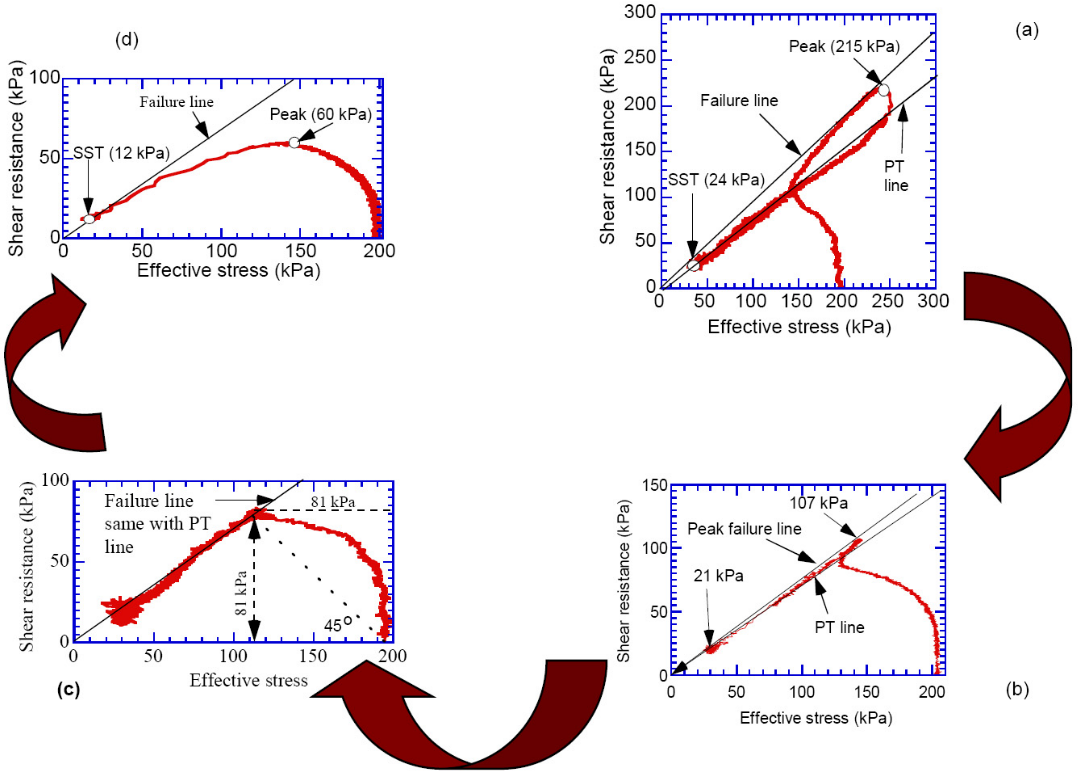

Fig. 2. The proposed soil transition route from dense to loose state: (a, b, d) well-known stress paths (c) less-known stress path

The number of tests for friction angle at the threshold state might not be enough for any conclusive interpretation. However, all plots fall in the distribution of friction angles at steady state. This is probably because grain crushing and the resulting change in grading do not affect the friction angle, which in turn might be because the minerals making up the samples are the same and angularity remains almost the same although the sizes of grains and grading might change. The friction angles at phase transformation (before peak, where grain crushing is at minimum) fall in the same range of the friction angles at the threshold state and the steady state. This is also because grain crushing and the resulting change in grading have little or no effect on the angles. It can be said that under no positive or negative pore water generation state which may occur at the phase transformation point before failure, at failure point in the threshold state, at the steady state point (after failure) where all grain crushing has been completed, the mobilized friction angles are almost same even though grading are different. Sassa (1988, 2000) proposed the friction angle during motion (ϕ_m) as the friction angle mobilized during motion of landslides. However, he did not mention that this value remains constant. This research shows that this friction angle during motion remains constant from the initial stage of movement to the steady state even though grain crushing and the resulting change in grading are associated with the movement. The stress paths of WG, ING, and NAG specimens with various relative densities at a normal stress range of 196 kPa to 204 kPa. It has been shown that the friction angle at phase transformation state and steady state are approximately equal with a value of 36.3° for WG. It has also been shown that the friction angles are approximately equal with a value of 35.7° for ING. The friction angles are also approximately equal with a value of 35.6° for NAG. This finding may broaden the range of application of the friction angle during motion. The finding that the value of friction angle during motion is almost same with the friction angle at the threshold state, and practically same with the friction angle at phase transformation will provide a significant merit in the prediction of landslide mobility and hazard area because both friction angles at threshold state and phase transformation can be obtained by the conventional shear box test. Especially, the friction angle at phase transformation can be obtained by a single test by the shear box test; which will be a very convenient process (Sassa 1988, 2000). One of the major outcomes of the investigation undertaken to discover factors initiating or triggering flow slides was the observation that by carefully altering the relative density of a material of any gradation at a given effective confining stress, that material could become dilative or contractive in behavior. Dilative specimens have distinct phase transformation and peak stress states. Contractive specimens are defined by only distinct failure state. However, in between these two fundamental behaviors is a relative density at which the phase transformation and peak stress states tend to coincide with excess pore pressure and shear resistance becoming equal at the phase transformation point and not only remained equal but essentially constant until failure, thus establishing a threshold state at a small shear displacement. The equality and subsequent constancy of excess pore pressure and shear resistance, which started at shear displacements as small as 1.5 mm and continued until the sample failed, are typical characteristics of specimens that tend to form a transition region by demarcating the contractive from the dilative behavior. It is shown that under definite conditions of loading, a threshold state, characterized by the equality and subsequent constancy of pore pressure and resistance from a few seconds after the commencement of shearing until failure, develops in the sands at a given density. It is revealed that not only do the sand samples characterized by this state sustain the unity of the ratio $\Delta u/\Delta\tau$ from a few seconds after the start of shearing until failure but that their behavior unambiguously forms a boundary between the contractive and dilative sands at a fixed confining stress. Any specimen so characterized appeared to experience the least dilation possible at a given confining stress such that at the phase transformation line $\Delta u/\Delta\tau$ was unity.

The relationship between friction angle at peak and relative density was critically examined. The friction angles at peak for all grading increase with relative density. This result is quite different from those obtained with the friction angles at steady state, phase transformation and threshold state.

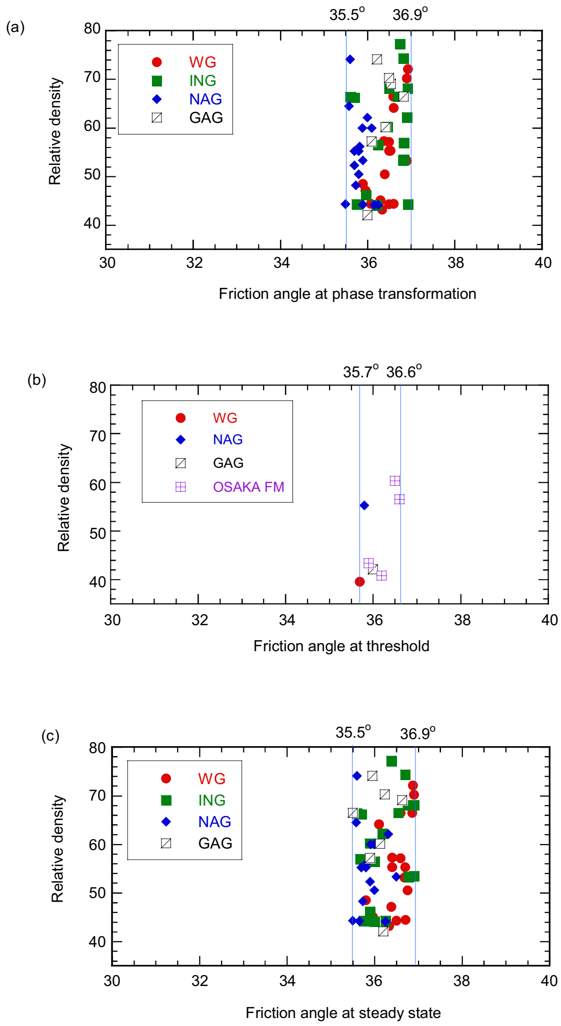

Fig. 3. Relationship between friction angle at phase transformation, threshold state, and steady state and relative density

It is interesting to note that while the friction angles of all grading at steady state are nearly equal, the steady state strengths are quite different (Fig. 4b). Results of the tests conducted on medium-dense and dense specimens are summarized in Fig. 4. The Figure, which contains only data acquired at an initial effective normal stress of about 200 kPa, shows that there are significant differences between their peaks in medium and dense states; the differences appear to widen as density increases.

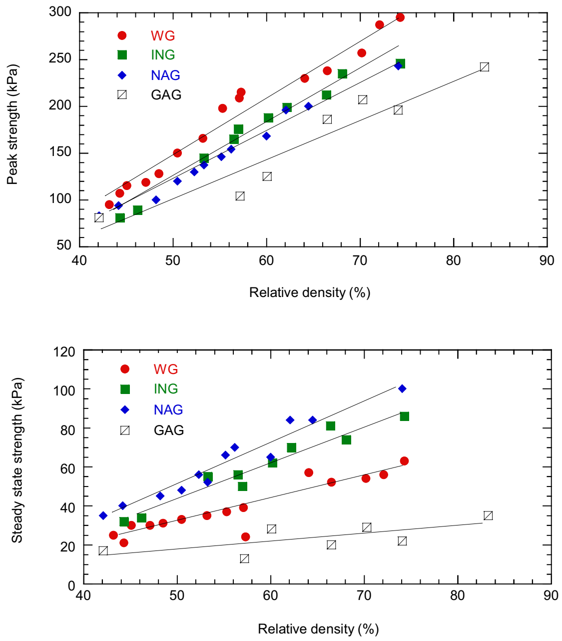

Fig. 4 The relationship between relative density and shear strength (a) peak strength (b) steady

This may imply that the effect of grading on their peak strengths may be significant for only densities above a certain limit. The proximity of peak strengths of narrowly and intermediately graded specimens may find explanation in the closeness of their uniformity coefficients.

While the peak strengths of dense intermediately graded specimens approach those of dense narrowly graded ones, the well-graded specimens at the same state move away. On the basis of the results presented in Fig. 4 (a) it is evident that at the same relative density and effective normal stress, well-graded specimens have higher peak shear strengths than the other specimens.

In what may amount to a reversal of behavior, well graded specimens, which contain some quantity of relatively large particle sizes, can be observed in Fig. 4b to have lower steady state strength than the intermediately and narrowly graded which are composed of only finer sizes. Test results seem to indicate that the degree of crushing or breakage is heavily dependent on the size of particles of which the specimens are composed (Marsal, 1967; Lee and Farhoomand, 1967; Hardin, 1985; Lade and Yamamuro, 1996; Wang, 1998)

Analyses have shown that, at the same initial state, well-graded sands have higher peak strengths than poorly graded ones, the difference increasing as the sands become denser. In an interesting reversal however, it has been shown that in medium and dense states, the steady state strengths of well-graded materials are lower than those of the ING and NAG, the difference, again, increasing with relative density (Igwe et al., 2005b)

From the view point of public safety, the better graded soils in medium and dense states are more dangerous after failure because of their potential for large post-failure travel distances than the NAG, and ING specimens. These results may aid engineers when they make decisions on what soils to use for civil engineering works; or scientists when they assess slope stability, investigate landslides, or plan prevention methods (Igwe et al., 2004a; 2004b; 2005a; 2007).

3.3 This and other pore pressure parameters

The present research compares well with other pore pressure parameters. The essence of determining the values of pore pressure parameters is to quantify the magnitude of excess pore pressure generated at a given point in the subsoil by changes in stress. Notable pore pressure parameters include, the B and A parameters defined for triaxial tests (Skempton, 1954); and the B_D parameter and A_D parameters defined for the direct shear tests (Sassa, 1988). Of particular interest to the author are the A_D and A parameters.. Sassa (1988) had defined A_D value at failure (A_{Df}) as equal to $\Delta u_f/\Delta\tau_f$ which was derived in the following way: $\Delta u_f = B_D\,(\Delta\sigma + A_D\Delta\tau)$. If the test conditions are B_D = 1.0, $\Delta\sigma$ = 0, then $A_{Df} = \Delta u_f/\Delta\tau_f$. The questions then is how do soils with A_{Df} = 1 behave and what are their geotechnical significance? The present research has shown that such soils mark the boundary between contraction and dilation.

Skempton had defined A as equal to $(\Delta u-\Delta\sigma_3)/\Delta\sigma_1-\Delta\sigma_3)$. If $\Delta\sigma_3$ = 0 in triaxial tests under constant confining pressure, the equation reduces to A $= \Delta u/\,\Delta\sigma_1$. While explaining parameter A determined from undrained triaxial tests, Skempton (1954) had noted that it was possible to associate certain effective stress paths with certain values of the pore pressure parameter A. Some of these stress paths and their A values are shown in Fig. 5. It is noted from Fig. 5-1a that an effective stress path characterized by a slope of 1:1 to the right is associated with A = 0. A vertical stress path is associated with A = 1/2. An effective stress path characterized by a slope of 1:1 to the left is associated with A = 1. Fig. 5-1 b shows that while stress paths to the right and below that for A = 0 will have A values that are negative, stress paths to the left and below that for A = 1 indicate A values greater than one. Skempton (1954) further hinted that a pore pressure parameter having a value greater than one or less than 0 are not only unusual but also deserved careful investigation. It has been reported that a pore pressure parameter greater than one is associated with loose structures (in either sand or clay) which collapses during loading. Skempton (1954) implies that soils navigating any stress path to the left and below that for A = 1 may suffer collapse and flow liquefaction, while those above the stress path for A = 1 should not suffer any collapse. This is the major background of and the primary motivation for the author's research into the dominant factors initiating collapse and flow slides. There does not seem to be any previous work using the ring shear apparatus to investigate the behavior of soils for which A = 1 and A > 1. Discovering the behavior of soils whose stress paths are at a slope of 1:1, and associating those stress paths to a critical state or condition will certainly mark an important boundary that will benefit the prediction of soil behavior.

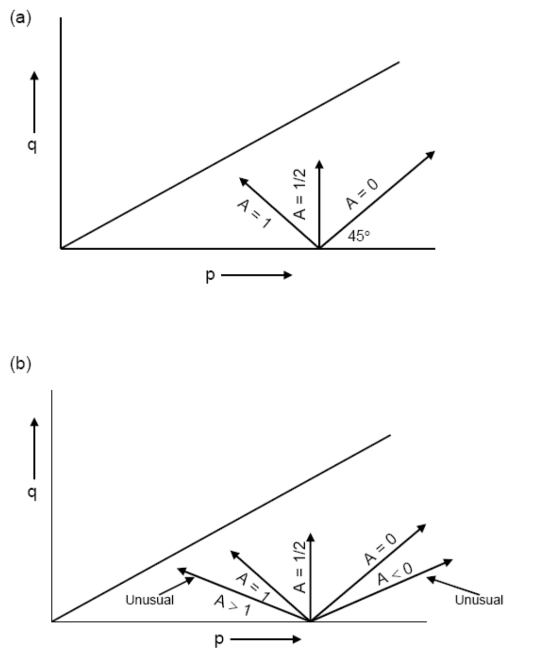

Fig . 5 Special values of A (Skempton 1954)

4 Hypothesis

4.1 Background

Normally consolidated soils (Fig. 6 a, c) at same confining stresses will follow stress paths in Fig. 6 (a) and Fig. 6(c) respectively depending on the material state of the samples. For these samples, the conditions at PT line are such that a dilation potential index, A_{rf}, ($Ar_f = \Delta u_f/\Delta \tau_f$) are < and = 1 respectively. The conditions prevailing at Fig. 6 (c) are recognized in this paper as a threshold state. If however, the soil is made in such a way that ensures the stress path follows as in Fig. 6b, the specimen may not go through the phase transformation stage because its A_{rf} would clearly be greater than one.

4.2 Significance of the hypothesis

The specimen may, instead, collapse and liquefy. It may be beneficial to note that a dilative specimen (Fig.6a) should have distinct phase transformation and peak stress states while contractive specimens (Fig. 6b) may be easily identified by just a distinct failure state. In between these two fundamental behaviors is a relative density at which the soil should neither dilate nor contract (Fig.6c). The present theory underlines the fact that the magnitude of excess pore pressure from the outset of any undrained test determines whether or not a given specimen will pass through the phase transformation stage. The fate of specimens whose excess pore pressures are not big enough to induce outright liquefaction and avoid reaching the PT line, depends on the ratio $\Delta u/\Delta \tau$ at the phase transformation point. If this ratio is unity, pore pressure and shear resistance should remain the same until failure occurs, meaning that the sample may experience the least dilation possible at a given effective stress. The PT line of such a specimen may be approximately equal to its failure line because the state of stresses at the PT point approximately coincides with those at failure. This condition will define a critical or threshold situation. All other stress paths above this critical should dilate, while other stress paths below it should show contractive behavior. To enhance comprehension, the pore pressure at which this critical is observed will be called a threshold pore pressure.

From the results of the present research, it is now known that as useful and important as Casagrande's (1936) and Castro's (1969) concepts are, they have failed not only to address questions regarding the boundary between contraction and dilation but also concerns regarding effective criteria for distinguishing, in the field, between flow-type liquefaction and non flow-type ones. Triaxial compression may simulate compression in the soil mass but it cannot reproduce direct shear state and the resulting formation of sliding surface which is a criterion for landslides. In the practical problems of landslides, post-failure behavior in large shear displacement is also very important. Those can be investigated only by ring shear tests. Undrained ring shear test were not possible before the development of undrained dynamic loading ring shear test by Sassa and his group (Sassa, 1996; 1997; and Sassa et al. 2003)

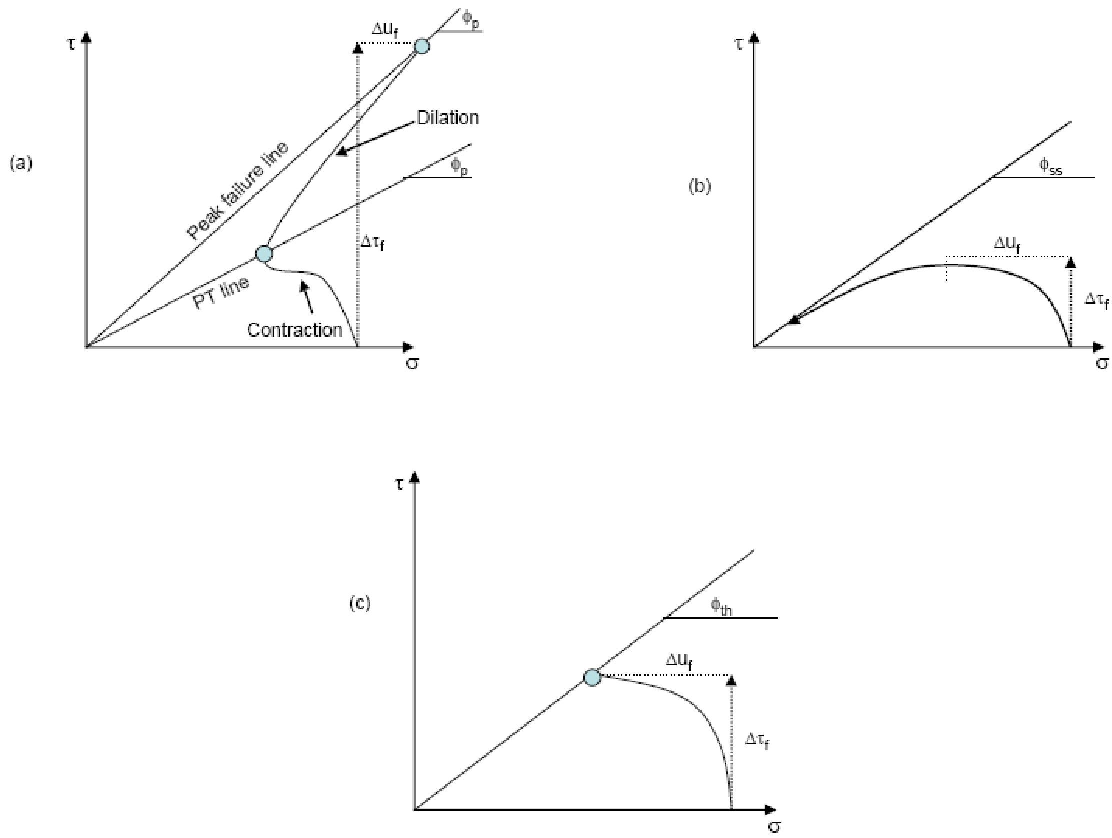

Fig. 6 The theory of threshold pore pressure

Although the drained triaxial tests of Casagrande did lead to the discovery and subsequent adoption of the critical void ratio concept as a useful tool in soil mechanics, the stress path of a soil at critical void ratio in undrained tests is still poorly understood. For the avoidance of doubt, the critical void ratio concept derived from drained tests, presupposes that, in undrained condition, a given mass of sand denser than the critical void ratio will first contract then dilate after transforming at the PT state, while a given mass of sand looser than the critical void ratio will exhibit a purely contractive behavior without experiencing any form of the dilation defined in this paper.

4.3 Characterizing soils by the new method

The concept further presupposes that a given mass of sand at a critical void ratio should, on reaching the PT state, neither dilate nor contract until failure occurs. Castro (1969) while confirming the validity of these suppositions failed to show that in undrained condition a soil at critical void ratio on reaching the PT state would neither dilate nor contract until failure took place. While it was expected that the undrained behavior of sand at a critical void ratio would conform to the supposition arising from Casagrande's drained test results, Castro's undrained tests yielded, instead, a different behavior – limited liquefaction – which has become the subject of much controversy and contention.

Castro's legacy comes with serious implications, chief of which is that if the void ratio of soils under same effective normal stress is methodically altered, the transition from dense to loose or from loose to dense would be such that the soils will either be characterized with the presence of only a failure line or the presence of a phase transformation line and a failure line located at points that are bounded by mutually exclusive conditions. In a layman's words, it means that starting from dense for instance, if the void ratio of soils held under same effective normal stress is systematically reduced, the transition from dense to loose will be in such a manner that permits soils, which initially had their phase transformation lines and failure lines located at two different points, to suddenly end up with only a failure line in loose state. Soil mechanics and geotechnical engineering have carried this implication along their many decades of crucial advances; there does not seem to have been any previous attempt to make room for a transition from dense to loose and vice versa that recognizes the coincidence of the phase transformation and the failure lines as a boundary between the contractive and dilative responses of soils in undrained condition.

It appears reasonable, from the results of the present research, to think that if increasing density leads to increasing difference between pore pressure and shear resistance at the PT point, then, the converse will also be true. A decrease in the density of a dilative specimen will decrease the difference between pore pressure and shear resistance at the PT point. As density is decreased further, a time reaches when the pore pressure and shear resistance at the PT point will have the same value; and will remain the same until failure takes place. This situation establishes a threshold state and unambiguously defines a transition condition for all specimens under the same effective normal stress. Specimens denser than that for which a critical condition was defined would dilate, while those looser than the critical would collapse and show purely contractive behavior.

5 Conclusions

1. There is a threshold state, which forms a transition region between sands that contract from those that dilate. The threshold state is characterized by the unity of the ratio $\Delta u/\Delta\tau$ at failure; and the ratio is independent of both initial confining stress and grading of specimens. The possibility of specimens exhibiting either contractive or dilative behaviors seems to depend on whether or not the threshold pore pressure is exceeded. When exceeded, specimens tend to collapse and liquefy. When the contrary is the case, specimens tend to dilate. The threshold pore pressure may be expressed as u_c = ($\sigma\tan\phi$/(1+$\tan\phi$)deduced from the relationship $\Delta u = \Delta\tau$ at failure, where u_c denotes the critical pore pressure, σ represents the total normal stress, and ϕ stands for the angle of internal friction.

2. Friction at phase transformation (PT) seems to be independent of grading. Similarly, the friction angle at the steady state (SS) appears to be independent of grading.

3. All friction angles at phase transformation, threshold state, and steady state are almost equal independent of relative density.

Grain crushing will proceed after failure resulting in change of grading during the process. However, it has been found by this research that change of grading during motion does not affect the friction angle during motion which is equal to the friction angle at phase transformation, threshold state and steady state.

4. The friction angle at steady state（ϕss）is the key parameter for predicting the velocity and travel distance of landslides. The finding that the friction angle at steady state is close to the friction angle of phase transformation and threshold state will provide a significant merit in the prediction of landslide mobility and hazard area because the friction angles at phase transformation and threshold state can be obtained by the conventional shear box test.

5. Well-graded specimens in medium-dense to dense states have higher values of peak strength than the rest of the specimens at the same condition, with the difference appearing to increase with relative density. The values are ranked as WG> ING>NAG>GAG. Better interlocking of particles at contacts achieved by mixing a wide range of particle sizes in the well-graded specimens is thought to be responsible for the higher peak strength values associated with the well-graded specimens.

6. The steady state strengths of specimens in medium to dense states have greater values in proportion to the relative densities. The values are affected by grading of sands and ranked as NAG>ING>WG>GAG . It is noted that whereas the steady state strengths are affected by grading, the friction angle at steady state is not affected by grading as stated in conclusion 8 below.

7. It is noted that widely graded specimens have greater peak shear strengths than poorly graded specimens of NG and ING. On the contrary, the steady state strengths of well graded specimens are smaller than those of

poorly graded specimens. It may be interpreted as fine particles can exist within big pores of coarse particles in well graded specimens. So the packing of grains in well graded specimens is dense, it will increase difficulty of overriding of grains in shearing. Accordingly the peak shear resistance will be increased. Difficulty of overriding is likely to result in greater possibility of grain crushing during shearing. It may generate high excess pore water pressure and result in lower steady state shear strengths.

The post-peak behaviors of these samples are important from the viewpoint of public safety. It is noted that although well-graded samples have higher peak strengths, their low steady-state strength may pose a serious safety concern because of their potential for long travel distances.

8. Gap graded specimens have the lowest peak and steady state strengths. The coarse and very fine grains in gap graded specimens may be packed in a way that permits the overriding of grains readily. The coarser grains are vulnerable to crushing, and as they crush or break, the finer particles tends to fall within the pores of coarse grains, making it denser but also resulting in a smaller steady state value due to high pore water pressure.

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地すべり危険度評価：リングせん断試験機を用いた土の非排水せん断時の力学特性についての研究

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要 旨

流動性の高い地すべりのメカニズムを解明するための基礎的研究として，粒径の異なる 5 種類の珪砂を混合させることにより粒度の異なる 4 種類の供試体を作成し，各種の条件下でトルク制御非排水リングせん断試験を実施した。概ねゆる詰め供試体は流動化し，密詰め供試体は高いせん断強度を発揮した。これを詳細に検討した結果，せん断開始時から破壊時までの過剰間隙水圧増分とせん断応力増分の比が 1.0 になる場合が臨界状態であること，及びこの臨界状態の水圧・応力増分比の値は，垂直応力，粒度分布によらず一定であることを見出した。この水圧・応力増分比の値が 1.0 より小さいほどせん断破壊強度は大きくなり，1.0 より大きくなる場合には流動性地すべりが生じることが推定された。

また，粒度分布の異なる砂試料において発揮される非排水ピーク強度は，均等係数の大きいほど大きくなり，逆に定常状態強度は，均等係数の大きいものほど小さくなることが見出された。しかし，大粒径の砂と小粒径の砂を組み合わせたギャップ配合の砂は，ピーク強度，定常状態強度とも最低の値を示した。定常状態強度は，せん断中に生じる粒子破砕とそれに伴う体積収縮の程度に支配されていると推定された。

キーワード：流動性地すべり，摩擦角，臨界状態，粒度，せん断強度