Excess pore water pressure: a major factor for catastrophic landslides

OGBONNAYA IGWE[1], KYOJI SASSA[2] & HIROSHI FUKUOKA[3]

[1] Research Centre on Landslides, Kyoto University. (e-mail: igwe@landslide.dpri.kyoto-u.ac.jp)
[2] Research Centre on Landslides, Kyoto University. (e-mail: sassa@landslide.dpri.kyoto-u.ac.jp)
[3] Research Centre on Landslides, Kyoto University. (e-mail: Fukuoka@landslide.dpri.kyoto-u.ac.jp)

Abstract: A theoretical model predicting the pore pressure change necessary for liquefaction failure of saturated soil masses in undrained conditions is assessed. It is shown that a threshold pore pressure, u_t, derived from the Mohr Coulomb failure criterion when pore pressure at failure is equal to the corresponding shear resistance, is enough to initiate liquefaction type of failure in sandy masses. Loading tests to failure on source-area sandy soils from a catastrophic landslide location, undertaken to verify the model, show that under definite conditions of loading, a threshold state, characterized by the equality and subsequent constancy of pore pressure and shear resistance from a few seconds after the commencement of shearing until failure, develops in the sands at a given density. It is shown that not only do the sand samples characterized by this state sustain this unique equality and constancy from a few seconds after the start of shearing until failure but that their behaviour unambiguously forms a boundary between the contractive and dilative sands at a fixed confining stress. Samples in which the threshold pore pressure was exceeded readily liquefied while those in which the pore pressure built-up was below the limit gained strength by tendencies to dilate. This paper demonstrates that while the stability of a slope founded on sandy soils may be breached when the pore pressure exceeds a certain limit, it is possible to make estimates of the limit. It is shown that where such estimates are accompanied with adequate field measurements of pore pressure, the efficiency of landslide prevention projects may be enhanced because only slopes whose stability is proven to constitute a real public threat are reinforced and reinforced adequately.

Résumé: Un modèle théorique prédisant que le changement de pression de pore nécessaire pour l'échec de liquéfaction de masses de sol saturées dans undrained la condition est évalué. Il est montré qu'une pression de pore de seuil, ut, dérivé du critère d'échec de Coulomb de Mohr quand la pression de pore à l'échec est égale à la résistance de cisailles qui correspond, sera obligé à inaugurer assez le type de liquéfaction d'échec dans les masses sableuses. Les tests de chargement à l'échec sur le source-secteur sols sableux d'un emplacement écrasant catastrophique, entrepris de vérifier le modèle, le spectacle que sous les conditions définies de chargement, un état de seuil, caractérisé par l'égalité et la constance subséquente de pression de pore et la résistance de cisailles de quelques secondes après le début de tondre jusqu' à l'échec, développe dans les sables à une densité donnée. Il est montré que fait non seulement les échantillons de sable caractérisés par cet état soutiennent cette égalité et cette constance unique de quelques secondes après le début de tondre jusqu'à ce que l'échec mais que leur comportement forme non ambigu une frontière entre les contractifs et sables de dilative à un limiter la tension fixe. Les échantillons dans lequel la pression de pore de seuil ont été facilement dépassés a été liquéfié pendant que ceux-là dans lequel l'accumulation de pression de pore était au dessous de la limite la force gagnée par les tendances pour dilater. Ce papier démontre que pendant que la stabilité d'une pente fondée sur les sols sableux peut être violée quand la pression de pore dépasse une certaine limite, c'est possible de faire des estimations de la limite. Il est montré qu'où telles estimations sont accompagnées avec les mesures de champ suffisantes de pression de pore, l'efficacité de projets d'empêchement écrasants peut être améliorée parce que dont l'incline seulement la stabilité est prouvée pour constituer une vraie menace publique est renforcée et est renforcée suffisamment

Keywords: shear stress, saturated materials, liquefaction, landslides, collapse, load tests

INTRODUCTION

Landslides are vicious slope movements accounting for inestimable amount of loss, waste and damage in virtually every part of the world. Triggered by earthquake, volcanic eruption, intense rainfall, rapid snowmelt, changes in water level, and even by the activities of man himself, it is very difficult, if not impossible, to overestimate their threat to public safety. They are known to have frequently breached the peace of cities and towns, sacked communities and villages, buried the wealth of rural and urban dwellers, wrecked countless hopes and dreams, harshly punished some sloppy structure designing, defied some inadequate preventive measures, and produced endless catalogues of carnage. Landslides do not only destroy homes and hopes, they also deface and devalue historical, cultural, and entertainment facilities so dear to man. Taming their aggression and ruinous impacts, thereby rescuing the environment from a potential crisis, should, in point of fact, become a priority. Liquefaction of saturated soils, often regarded as the fundamental cause of flow slides, has been responsible for many of the tragedies resulting from slope failures. The intense mobility of liquefied soils, which permits movements that range from several tens of meters to several thousands of meters, almost always ensures that huge amount of resources is lost in the wake of a landslide disaster. Sound knowledge of the mechanism of liquefaction, the factors that influence the liquefaction potential of a mass of soil, and the characteristics of liquefiable soils, is a potent tool not only in landslide investigation and mitigation but also in the civil engineering industry. For indeed careful and rigorous assessment of the liquefaction potential of sands when selecting them for embankments, dams, foundations, and roads is a tradition of immense importance in the construction industry. And because a great deal of failures of earth structures, foundations, and slopes founded on sands have been attributed to the liquefaction of the sands, stakeholders in environmental protection and urban development seem to have elevated the importance of liquefaction-evaluation by placing it at the heart of their management policies. This elevation of importance has, in part, inspired intense research leading to, for instance, better knowledge of the factors and dynamics behind the failure of Fort Peck Dam in Montana in 1938, Calaveras Dam in California in 1920, the Lower Lan Norman Dam, the foundation failures induced by the 1964 earthquake in Alaska, USA, and Niigata, Japan, and the flow slides in the province of Zealand in Holland and Mississippi River. Ever since the widespread destruction arising from the 1964 earthquakes in Alaska, U.S.A. and Niigata, Japan dramatically brought the subject of soil liquefaction to public awareness, considerable amount of research has been undertaken by several researchers, including Sassa, and colleagues at the Disaster Prevention Research Institute, Kyoto University, Japan, who have used one of the most refined ring shear apparatuses to simulate, as closely as possible, the stress-strain conditions that develop on a mass of soil when it is subject to conditions capable of triggering liquefaction.

The problem

Although liquefaction phenomenon has been the subject of a barrage of investigations and publications for decades now, its mechanism leading to large lateral displacements has yet to be fully understood. Questions such as – why do some soils collapse and liquefy whereas others, under identical stress conditions, dilate and gain some measure of strength; and what are the primary factors triggering liquefaction and flow failures, especially, in loose cohesionless soils, only serve to underscore the incompleteness of what researchers as yet know about soils liquefaction. Finding perfect answers has led to the emergence of a good number of beneficial concepts including Casagrande's critical voids ratio concept. In spite of the emergence of these concepts, questions still remain, especially as to effective ways of relating the critical state of soils with essential soil parameters, such as pore pressure and shear resistance; there does not seem to have been any previous attempt to relate collapse and liquefaction to an experimentally-verifiable limit or critical value of pore pressure, above which collapse occurs and below which it does not. In this paper, two new concepts – the concepts of least dilation, and threshold pore pressure – are introduced to interpret the undrained shear behavior of granular soils at a threshold density. It is shown that the characteristics of the soils so interpreted tend to define the boundary between contraction in loose, and dilation in dense soils held under same effective normal stress. This paper also demonstrates that while the stability of slopes founded on sandy soils may be breached when the pore pressure exceeds a certain limit it is possible to estimate the limit. Where such estimates are accompanied with adequate field measurements the efficiency of landslide prevention projects may be enhanced because only slopes whose stability is proven to constitute a real public threat are reinforced and reinforced adequately.

Liquefaction and limited liquefaction

Since not all slope failures are due to liquefaction, establishing a standard that enhances the prediction and identification of flow-type failures in the field will not only shed more light on the mechanism of soil liquefaction but will also improve the efficiency of slope stability analysis. Literature is replete with studies, including that by Ishihara (1993), attempting to establish such a standard. Following a summary of a good number of field and laboratory data, Ishihara (1993) proposed a threshold SPT N-value to distinguish between flow-type and non-flow type failure. Although researchers have made quality efforts at drawing a boundary between liquefaction and non-liquefaction, they have yet to find a common ground over what behaviors of sand, as observed in the laboratory, should be recognized as an important mechanism determining the occurrence or otherwise of flow-type failures in the field. It may be important to note that even though beneficial concepts, hypotheses, and postulates explaining the undrained behaviors of sands whose voids ratio exceed or fall below the critical density exist in the literature, there has yet to be a distinctive behavior associated with sand at critical density. Determining how sand at a critical density behaves during undrained loading may be important in understanding more about soil liquefaction.

Three basic undrained behaviors of granular materials are very commonly referred to in geotechnical discourse: dilation, limited or partial liquefaction, and liquefaction, Fig. 1. The phase transformation line (PT line) as recognized by Ishihara 1993, is a line passing through points where contractive behaviors terminate and dilative behaviors begin, in specimens that first contract, and then dilate. Although the validity of limited liquefaction as a true soil behavior has been subjected to a considerable amount of doubt, debate and controversy, the three basic behaviors sketched above are not only a very useful means of characterizing granular soils but also an effective means of understanding the mechanism of slope failures. The occurrence of, and practical implication of the so-called limited liquefaction have been a contentious issue with two opposite views increasingly gaining currency. Sutter and Smith (1980) have reported that the occurrence of limited liquefaction is a function of how close the void ratio of a given material is to a critical void ratio. They have noted that whereas specimens with voids ratio considerably higher than the critical would almost certainly suffer complete liquefaction, those whose voids ratio are marginally higher or nearly equal to the critical would experience limited liquefaction. Sutter and Smith's results are supported by those of Castro and Poulos (1977), and Poulos et al. (1985) who, while assessing procedures for evaluating the undrained steady-state strength of sands with results of undrained triaxial tests, have reported that the undrained strength of sands was dependent on only insitu void ratio; and independent of either soil fabrics or loading methods. They conclusively showed that sands whose voids ratio exceeded a certain threshold value suffered liquefaction instead of the so-called partial liquefaction.

Evidences from other works, however, seem to indicate that the occurrence of limited liquefaction does not depend wholly on the proximity of material density to the critical, but in part on the constraints offered by the testing apparatus, and test conditions (Mathew and John, 1991; Jude, 1998). It is this partial dependence on apparatus constraints that has compelled some (like Jude 1997, Love 2000) to question the validity of limited liquefaction as a true soil behavior. In their elaborate argument, doubts have been raised over the possibility of observing, in the field, a material flowing and at the same time undergoing hardening. Those who support limited liquefaction as a true soil behavior have, however, tried to make sense out of the frequency at which the behavior is observed. Relying heavily on the rate at which the behavior is observed on loose specimens during testing, they have vigorously demonstrated that the behavior is indeed a true characteristic associated with the deformation of granular materials in undrained shear. In spite of these divergent views, however, there seems to be a consensus that there exists a boundary between a purely dilative behavior and liquefaction, whether complete or limited liquefaction. But, that boundary has yet to be clearly assessed.

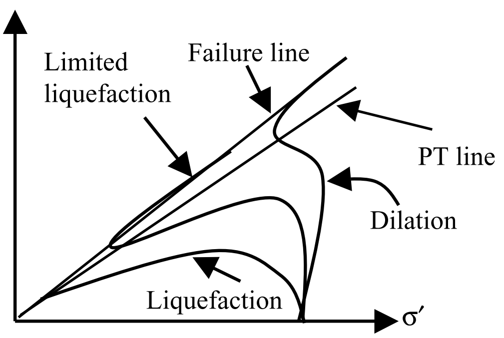


Figure 1. A sketch of the three basic behaviors of granular materials (after Castro 1969)

Objective and methodology

In the light of the above, it is possible then to ask whether or not there should be a boundary between dilation and liquefaction for material under same confining stress – limited liquefaction or complete liquefaction; and what the defining parameters of such a boundary should be. The approach employed in this paper was to carefully alter the void ratio of specimens held under same confining stress in attempts to identify stress paths whose peak strengths would nearly coincide with their strength values at the PT line. Any specimen whose peak strength equals its strength values at the PT line will be identified as the least dilating at a given normal stress because its phase transformation line will be the same as its failure line. The characters of such a specimen will then be used to define the boundary between dilation and liquefaction. Such a definition will permit adequate and logical interpretation of the behavior of soils as density is varied from dense to loose. It may instantly become obvious that if the density of a given mass of sand is gradually decreased, a density reaches where failure line and phase transformation line will coincide. Further decrease in density may lead to flow liquefaction behavior; with the ultimate consequence of having only a failure line as its prominent feature.

THE SOLUTION: HYPOTHESIS

The first step to solving the problem would be to design a strong theoretical foundation, which may include some hypothetical and idealistic boundary conditions, as a prerequisite to any laboratory test. In keeping with this, the author thinks that the best approach will be to first theorize that since there exist undrained stress paths (of medium-dense to dense sands) in which a PT line and failure line are located on points that are mutually exclusive, there might as well be a stress path in which the PT line and the failure line coincide. Such a stress path, if discovered, might not only be a new addition to the geotechnical engineering profession but might also define the boundary between contraction and dilation. To advance the cause of a fresh concept like this one, the next step should consist of hypothesizing on the possible boundary conditions that might compel a given mass of soil to neither contract nor dilate at the phase transformation state until failure takes place; and explaining the possible outcomes of such hypothetic assertions. For the sake of relevance, the hypotheses should draw from well-known facts and concepts while at the same time seeking to enthrone a new idea. This is exactly what the author has done by drawing from the wealth of Mohr Coulomb Failure Criterion and other beneficial ideas while creating a new concept, as the section below will show.

Normally consolidated soils (Figure 2a, b) at same confining stresses will follow stress paths WX and WZ respectively depending on the material state of the samples. For these samples, the conditions at PT line are such that a dilation potential index, r_f, ($r_f = \Delta u_p/\Delta\tau_p$) are < and = 1 respectively. The conditions prevailing at 2b are recognized in this paper as critical. If however, the soil is made in such a way that ensures the stress path follows WY as in Fig. 2c, the specimen will not go through the phase transformation stage because its r_f would clearly be greater than one. The specimen will, instead, collapse and liquefy. It may be beneficial to note that a dilative specimen (Fig.2a) should have distinct phase transformation and peak stress states while contractive specimens (Fig. 2c) may be easily identified by just a distinct failure state. In between these two fundamental behaviours is a relative density at which the phase transformation and peak stress states should coincide to form a threshold state (Fig.2b). 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 $\Delta u_p/\Delta\tau_p$ 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 will experience the least dilation possible at a given effective stress. The PT line of such a specimen will 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 situation. All other stress paths above this critical should dilate, while other stress paths below it should show contractive behaviour. To enhance comprehension, the pore pressure at which this critical is observed will be called a threshold pore pressure. If the ratio as seen above is less than one at the phase transformation line, the material will dilate significantly and its PT line will be different from its failure line.

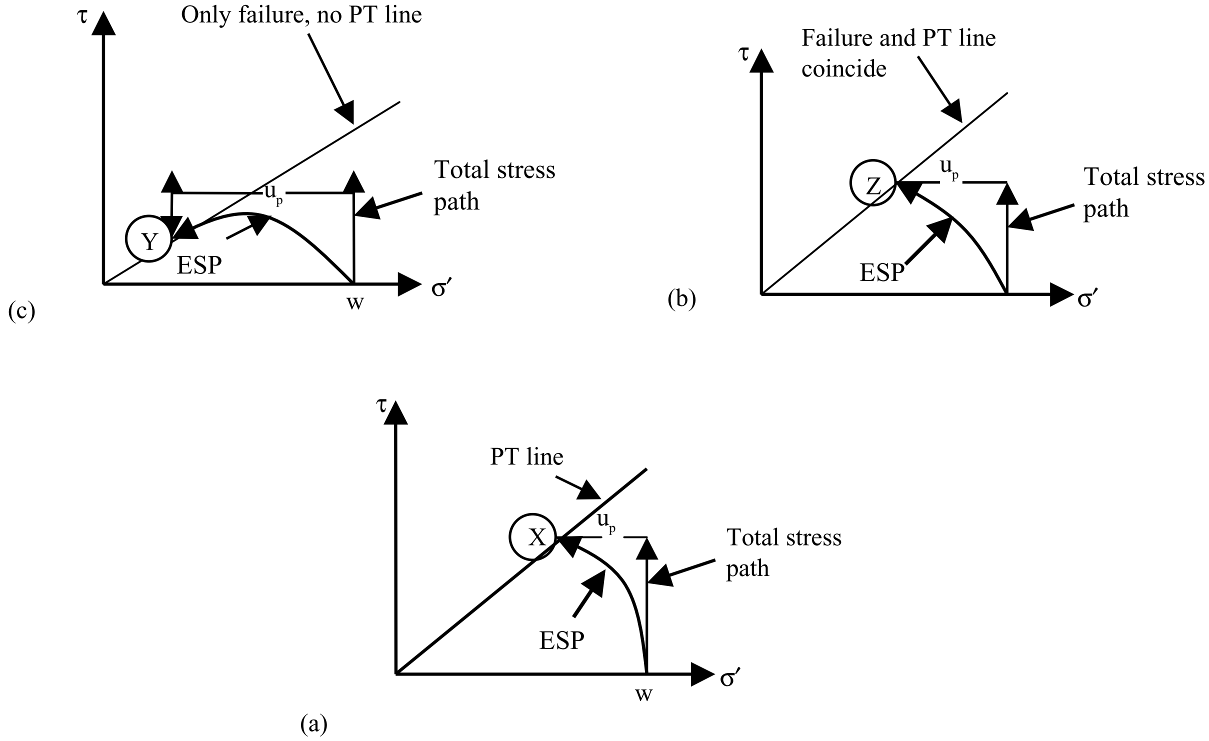


Figure 2. Schematic diagrams illustrating the concepts of least dilation and critical pore pressure **(a)** dilation **(b)** critical **(c)** liquefaction

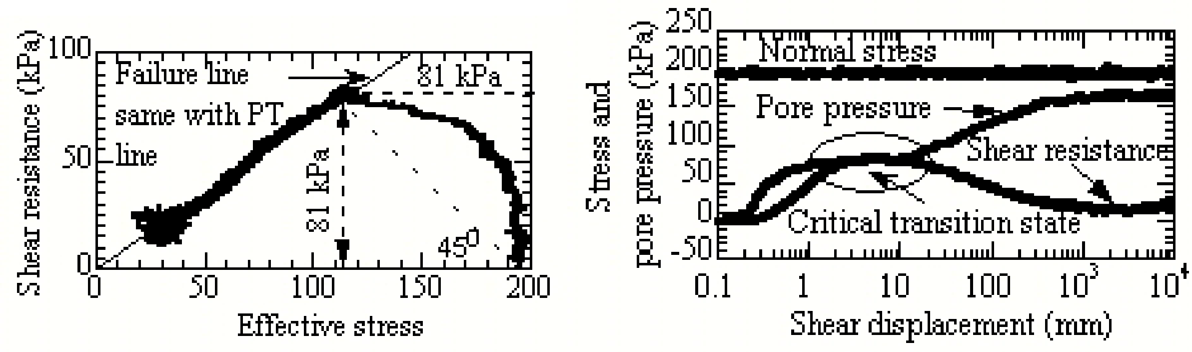
Experimental verification

When failure and phase transformation lines coincide

Artificially constituted silica sands and natural samples taken from the 1995 Takarazuka landslide (Fig. 3) that killed 34 people in Kobe, Japan were used to verify the concepts. Both the artificial and natural samples have similar physical properties, and indeed have almost the same friction angle. Figures 4a and b are stress path and stress versus shear displacement respectively of a normally consolidated gap graded silica sand material confined at 196 kPa with a void ratio of 0.89. The figures illustrate what happens whenever pore pressure at failure is equal to the corresponding shear resistance such that there is no distinction between the phase transformation stage and failure state because the specimen appeared to have experienced the least dilation possible at the given confining stress. Excess pore pressure and shear resistance became 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 (Fig. 4b). The equality and subsequent constancy of excess pore pressure and shear resistance, which started at about 2 mm and continued until the sample failed at 10 mm shear displacement, are typical characteristics of specimens that tend to form a transition region by demarcating the contractive from the dilative behaviour. Theoretically, it may be easy to see that any stress path below this critical will liquefy while any above will dilate. It may be noticed from Figure 4b that on becoming equal at the point that would have marked the phase transformation, pore pressure and shear resistance remained the same value until failure because dilation was obviously suppressed. If the specimen had dilated significantly, pore pressure and shear resistance would not have remained same value until failure because while the former would have decreased, the latter would have increased making it impossible for the values to remain the same. The same behaviour was found to be true in the Takarazuka specimens confined at 372 kPa and consolidated to a void ratio of 0.77, Fig. 5a and b.

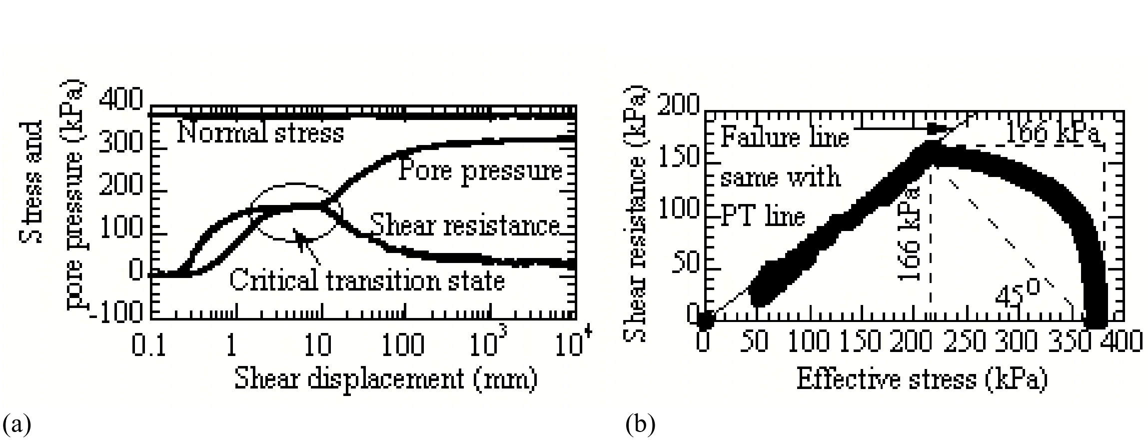
Although the iconic drained triaxial tests of Cassagrande did lead to the discovery and subsequent adoption of the critical void ratio concept as a useful tool in soil mechanics, there is no evidence that it has been show what the behavior or the stress path of a soil at critical void ratio in undrained tests would look like. 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 behaviour without experiencing any form of the dilation defined in this paper. The concept further presupposes that a given mass of sand at the critical void ratio should, on reaching the PT state, neither dilate nor contract until failure occurs. Castro (1966) 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 the scientific world wanted the undrained behavior of sand at a critical void ratio to conform to the supposition arising from Cassagrande's drained tests results, Castro's undrained tests yielded, instead, a different behaviour – limited liquefaction – which has become the subject of much controversy and contention.

Figure 3. Picture of the Takarazuka landslide that followed the Great Hanshin earthquake of 1995.



(a) (b)

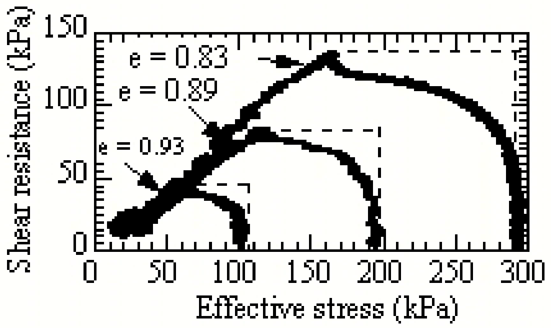
Figure 4. (a) Stress path defining a critical condition (b) Pore pressure and shear resistance behavior of silica sands



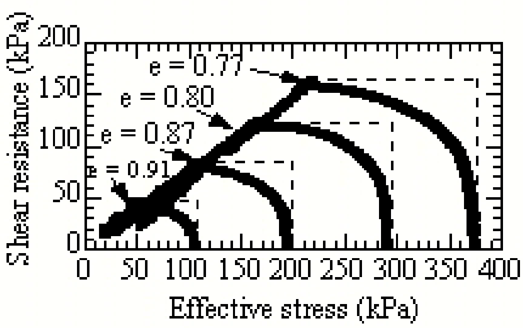
(a) (b)

Figure 5. (a) Pore pressure and shear resistance behavior of Takarazuka (b) Stress path defining a critical condition

The validity of the concepts under consideration was also tested at other normal stresses and found to be true in all cases (Figure 6a, b). Within the range of confining stresses used in the study, it was found that whenever pore pressure and shear resistance became equal at the phase transformation point the specimens dilated the least because the parameters (pore pressure and shear resistance) remained the same until failure. The specimens appeared to experience the least dilation possible at the given normal stresses, whenever pore pressure equalled shear resistance at the phase transformation line. This process ensures that their failure lines coincided with their phase transformation lines. Experimental results show that all other stress paths below these critical ones will collapse and liquefy, while stress paths above them will dilate and gain a good measure of stability



(a)

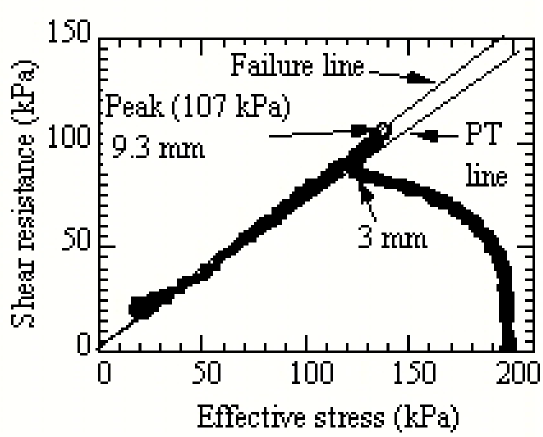


(b)

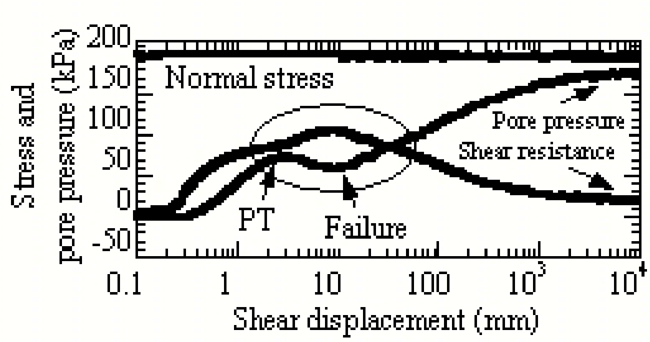
Figure 6. (a) Critical stress paths of silica sands (b) critical stress paths of the Takarazuka landslide sandy specimens

When failure and phase transformation lines do not coincide

One case, among many cases, that typifies situations where pore pressure at the phase transformation stage is not equal to the corresponding shear resistance is illustrated in Fig. 7a and b. The consequence of this situation is that the specimens dilated and ensured that the phase transformation line remained different from the failure line. It may be noticed that the important condition for dilation is for the pore pressure and shear resistance at PT to be different, no matter how small the difference might be. Fig 8b illustrates the mechanism of dilation in a silica sand specimen consolidated to a void ratio of 0.82. The figure shows that because pore pressure at PT line is different from the corresponding shear resistance, the specimen dilated, expressed as a decrease in pore pressure and a corresponding increasing in shear resistance (highlighted in the circle). These changes continued until failure occurred. For a denser specimen, the changes would even be more remarkable although they follow the same pattern. Because the difference between pore pressure and shear resistance at the PT line would be greater in a denser specimen, the dilation would also be higher than in Fig. 7a and b. As density increases, the difference increases too. Consequently, the dilation gets higher with peak strengths becoming increasingly greater than the strength values at the PT stage. 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)



(b)

Figure 7. (a) Typical stress path showing significant dilation because pore pressure and shear resistance at PT are not equal (b) Stress versus shear displacement

A decrease in the density of a material 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 (Fig. 8) 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 Figure 9. At any effective normal stress, there is only one stress path that will define this critical condition; meaning that only one specimen will dilate the least and as a result have its pore pressure and shear resistance equal from the PT point until failure. It is the opinion of this paper that this is one of the conditions that can lead to a failure line coinciding with the PT line. The coincidence of the PT line with the failure line is a new phenomenon that might become very useful in predicting and characterizing the behavior of granular materials held under same confining stress. Liquefaction occurs in loose soils because pore pressures generated in them during static loading tend to exceed the critical.

Dense soils are difficult, not impossible, to liquefy because pore pressures generated in them during static loading might be below the critical value at a given effective normal stress. One of the gains of the present analytical method is that it provides a logical and smooth transition of sands from dense to loose, and vice versa, without leaving a gap behind. Starting from dense, it may be seen in Figures 8 and 9 that sample A with void ratio 0.77 has distinct failure and phase transformation lines. This is similar to sample B with void ratio 0.78. As the sample is made looser, a point reaches when, as in sample C, failure and phase transformation lines coincide because the sample experienced very little dilation. After this particular sample, all other samples can only posses a failure line with a complete absence of a phase transformation line. On the strength of laboratory evidences, it is possible to state that whenever the PT line coincides with the failure line of the granular materials at a given effective normal stress, the material dilates the least. And the shear properties of such a material will define the boundary between two important soil behaviors: dilation and liquefaction. The beauty of the new concepts lies in the fact that the defining parameters considered critical may be adequately represented in a stress-strain-void ratio space, and interpreted with references to some experimentally measurable quantities. Unlike abstract analogies, the reference parameters in the concepts under consideration may be directly observed and measured. Such a quantitative analytical procedure may be easily verified by competent colleagues elsewhere.

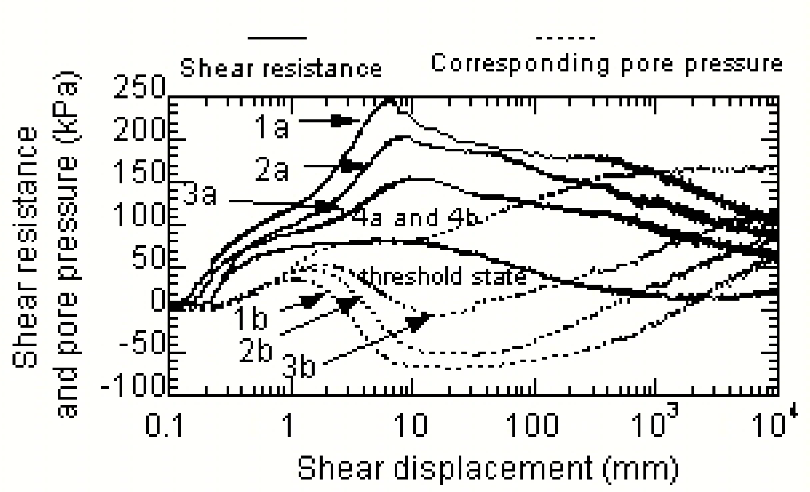


Figure 8. Shear resistances and the corresponding pore pressures of some sands as density changes (specimens at 196kPa confining stress)

Equally important is the fact that the critical pore pressure (the pore pressure required at the phase transformation line to ensure that a material dilates the least) can be quickly but conditionally calculated with as small as one good laboratory test at a known confining stress. The conditions for a reliable calculation include: 1) the specimen must be fully saturated; 2) the test must be monotonically loaded undrained; 3) Mohr Coulomb failure criterion must be applied. If, and when these conditions are satisfied, critical pore pressure, u_c, as used in this paper may be calculated from the following equation:

$$u_c = \sigma \tan\phi / (1 + \tan\phi)$$

Where u_c is the critical pore pressure, σ is the normal stress used in the undrained test, and ϕ is the friction angle of the material at a given normal stress σ. This is the amount of excess pore pressure that must be generated before liquefaction failure of the sands can be expected. It may be seen, from the equation, that the essential parameters required are normal stress on the material, and friction angle of the material only. Assuming (this is an important assumption that must be made) the friction angle of specimens under same confining stress do not vary, or vary only slightly, then the equation is capable of predicting the stress path which will have its pore pressure at failure equal to its corresponding shear resistance. It can predict the behaviour of the stress path which will dilate the least among any number of stress paths reaching a failure line from the same confining stress.

It may be interesting to note that with the behaviour of a critical stress path (the stress path of a sample at a critical density) now established, interpreting or predicting, with a fair measure of reliability and accuracy, other stress paths on the same confining stress as the critical would be less difficult. For instance, if the critical pore pressure of a specimen confined at 196 kPa is found, from calculation, to be 81 kPa, then the stress path of that specimen can be easily sketched in a stress-strain space. Consequently, it will be easy to see, at least in theory, that any stress path going over the critical may dilate, and any going below will collapse. This method is reliable because no two stress paths emanating from the same confining stress can reach the failure line at equal pore pressure values. In this light therefore, no other stress path can have the same value of pore pressure as the critical for indeed they cannot be two critical stress paths under one confining stress. The concept of critical pore pressure may derive its relevance from the fact that there exists an invaluable relationship between pore water pressure at failure and shear displacement. Although this is a well-known fact that does not need further commentary, it may be important to confirm that indeed the critical pore pressure as used in this paper was found to mark the boundary between small and large displacements. All specimens in which the critical pore pressure was exceeded were all found to suffer very large displacements with stunning rapidity.

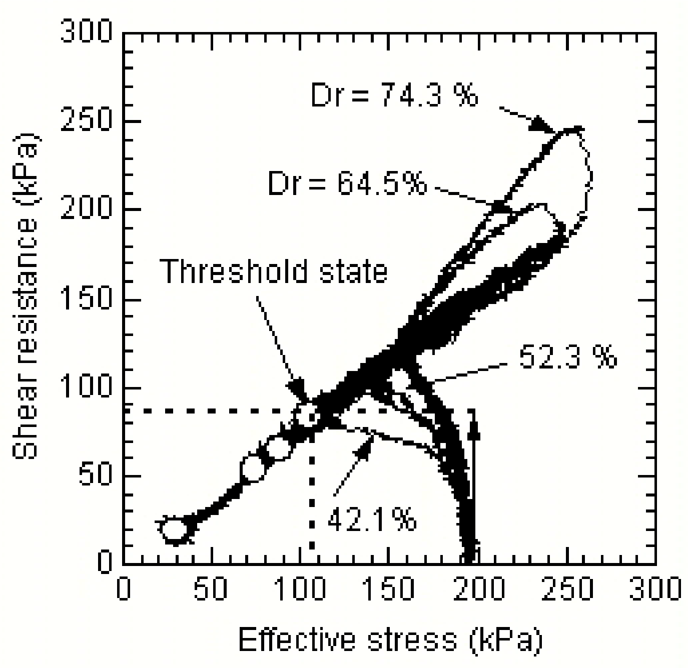


Figure 9. Stress paths illustrating the influence of density on the behavior of silica sand materials

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$$u_c = \sigma \tan\phi / (1 + \tan\phi)$$

Where u_c is the critical pore pressure, σ is the normal stress used in the undrained test, and ϕ is the friction angle of the material at a given normal stress σ. This is the amount of excess pore pressure that must be generated before liquefaction failure of the sands can be expected. It may be seen, from the equation, that the essential parameters required are normal stress on the material, and friction angle of the material only. Assuming (this is an important assumption that must be made) the friction angle of specimens under same confining stress do not vary, or vary only slightly, then the equation is capable of predicting the stress path which will have its pore pressure at failure equal to its corresponding shear resistance. It can predict the behavior of the stress path which will dilate the least among any number of stress paths reaching a failure line from the same confining stress.

It may be interesting to note that with the behavior of a critical stress path (the stress path of a sample at a critical density) now established, interpreting or predicting, with a fair measure of reliability and accuracy, other stress paths on the same confining stress as the critical would be less difficult. For instance, if the critical pore pressure of a specimen confined at 196 kPa is found, from calculation, to be 81 kPa, then the stress path of that specimen can be easily sketched in a stress-strain space. Consequently, it will be easy to see, at least in theory, that any stress path going over the critical may dilate, and any going below will collapse. This method is reliable because no two stress paths emanating from the same confining stress can reach the failure line at equal pore pressure values. In this light therefore, no other stress path can have the same value of pore pressure as the critical for indeed they cannot be two critical stress paths under one confining stress. The concept of critical pore pressure may derive its relevance from the fact that there exists an invaluable relationship between pore water pressure at failure and shear displacement. Although this is a well-known fact that does not need further commentary, it may be important to confirm that indeed the critical pore pressure as used in this paper was found to mark the boundary between small and large displacements. All specimens in which the critical pore pressure was exceeded were all found to suffer very large displacements with stunning rapidity.

CONCLUSIONS

1. Test results have shown that there is a critical or limit value of pore pressure, above which the sandy samples suffered sudden collapse and liquefaction, and below which they dilated and gained some measure of stability.

2 Any specimen whose pore pressure at the phase transformation point equals the corresponding shear resistance will dilate the least among other specimens held under the same confining stress.

3. Once pore pressure becomes equal with the corresponding shear resistance at the PT point, they remain the same until failure and ensure that the specimen dilates the least at a given confining stress.

4. The new concepts highlight the fact that there exist a fundamental relationship between changes in effective stress at failure and the shear displacement of the materials. The displacement of a material will remain at a 'safe', small value until the critical is exceeded. Once exceeded, the material suffers very large displacement, very rapidly.

5. Considering that all changes in shear resistance are entirely due to changes in effective stress, the changes in stress (shear resistance and effective stress) that give rise to the ratio of pore pressure at and shear resistance at failure being unity should be considered a crucial boundary distinguishing two very important soil behaviors – liquefaction and dilation.

6. One of the implications of these results is that some slopes are still sitting safe probably because a certain threshold pore pressure has yet to be exceeded. If such slopes must keep sitting safe, in situ pore pressure measurements followed with adequate drainage regime should be a grave necessity.

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Corresponding author: Mr Ogbonnaya Igwe, Research centre on Landslides, Kyoto University, Gokasho, Uji, Kyoto, 611-0011, Japan. Tel: +81 774 38 4107. Email: igwe@landslide.dpri.kyoto-u.ac.jp.

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