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Excess Pore Water Pressure Change Necessary to Cause Flow Liquefaction Failure of Sands Loaded in Undrained Condition

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Synopsis

A theoretical model predicting the pore pressure change necessary for liquefaction failure of saturated soil masses in undrained condition is assessed. It is shown that a threshold pore pressure, u_c, 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. 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.

Keywords: landslides, sliding-surface liquefaction, peak strength, phase transformation, pore pressure

1. 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 catalogs 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.

2. 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 critical 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.

2.1 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.

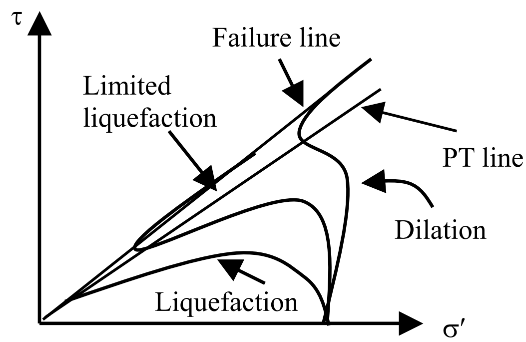


Fig.1 A sketch of the three basic behaviors of granular materials (after Castro 1969)

2.2 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 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.

3. The solution: hypothesis

Normally consolidated soils (Figure 2a, b) at same confining stresses will follow stress paths SX and SY 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 SZ 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 behaviors 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 behavior. To enhance comprehension, the pore pressure at which this critical is observed will be called a critical 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.

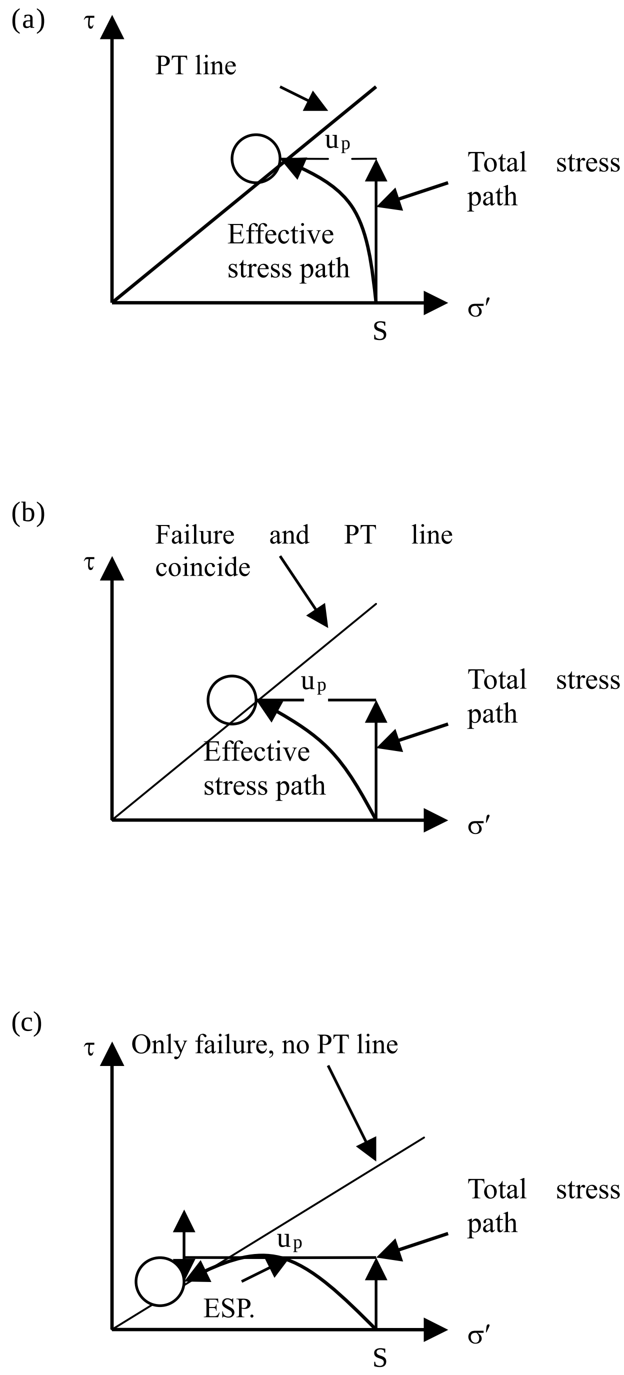


Fig 2 Schematic diagrams illustrating the concepts of least dilation and critical pore pressure **(a)** dilation **(b)** critical **(c)** liquefaction

3.1 Experimental verification

3.1.1 When failure and PT 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.

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

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 behavior. Theoretically, it may be easy to see that any stress path below this critical will liquefy while any above will dilate. 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. The same behavior 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.

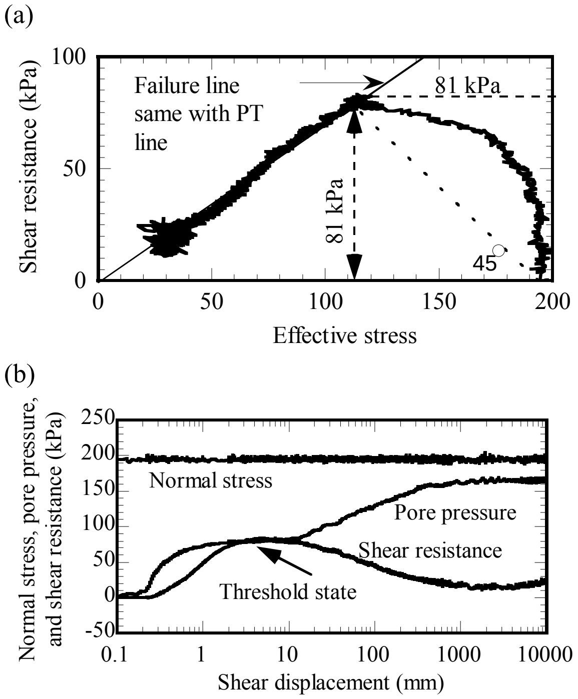


Fig. 4 (a) Stress path showing a threshold condition (b) Pore pressure and shear resistance behavior in a silica sand specimen

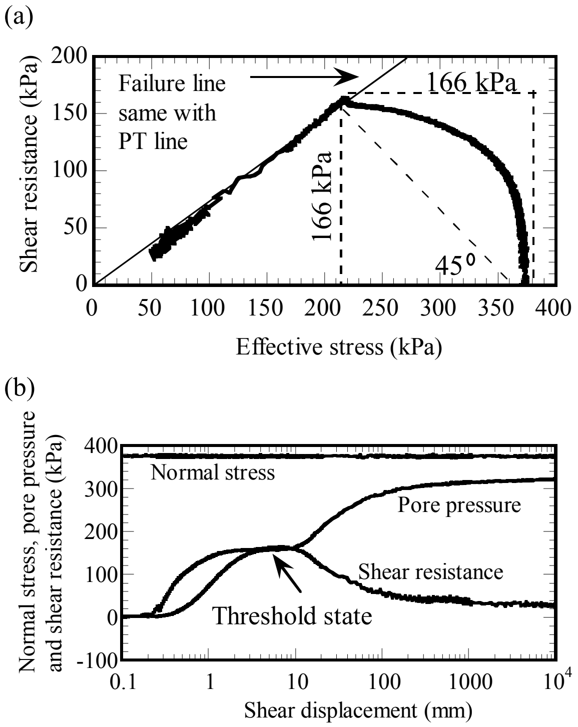


Fig. 5 (a) Stress path (b) Pore pressure and shear resistance behavior in a Takarazuka specimen

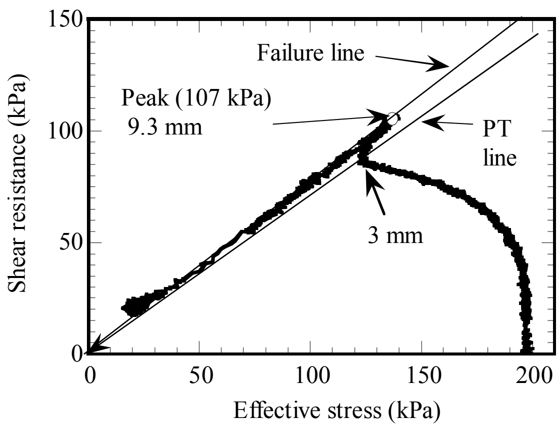
3.1.2 When failure and phase transformation lines do not coincide

Two cases, among many others, that typify situations where pore pressure at the phase transformation stage is not equal to the corresponding shear resistance are illustrated in Figs. 6 and 7. The consequence of this situation is that the specimens dilated and ensured that the phase transformation line remained different from the failure line. Fig. 6b 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 (as in Fig. 7). 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. 6a and b. As density increases, the difference increases too. 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 confining stress. Specimens denser than that for which a critical condition was defined would dilate, while those looser than the critical would collapse. At any confining 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 threshold. The 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. 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 colleagues elsewhere.

(a)



(b)

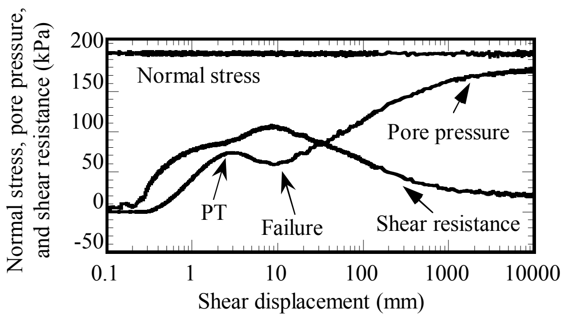


Fig. 6: behavior of a silica sand specimen with Dr = 44 % (a) stress path (b) stress versus displacement

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.

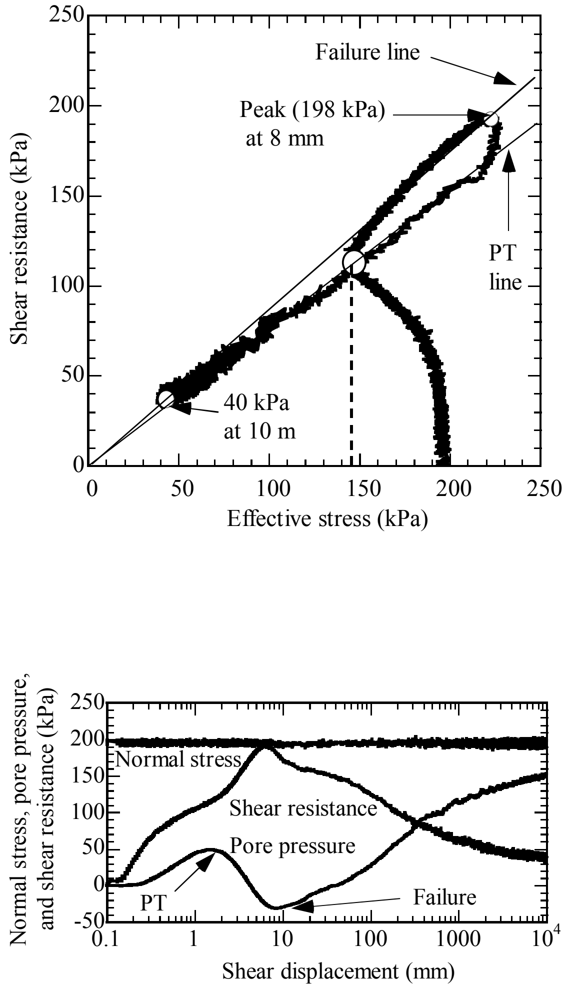


Fig. 7: (a) stress path (b) stress versus displacement of silica sand specimen with Dr = 50 % illustrating inequality between pore pressure and shear resistance at failure

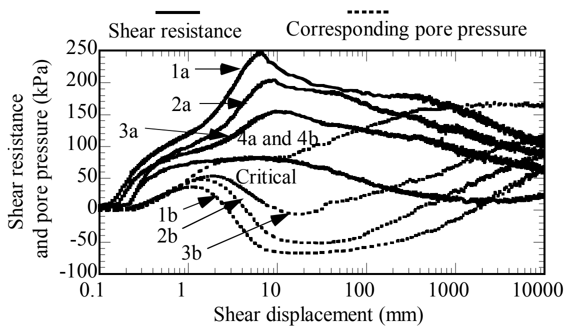


Fig. 8 Shear resistances and the corresponding pore pressures of some sands as density changes (specimens at 196kPa confining stress)

4. 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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非排水せん断による液状化の原因となる過剰間隙水の変化

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要旨

本研究では、非排水条件における飽和土塊の液状化崩壊に必要な間隙水圧の変化予測モデルを評価した。モール・クーロン破壊基準おける土のせん断抵抗が、これと等しくなる時に発揮されるしきい間隙水圧が液状化崩壊した試料で示された。崩壊性地すべり地の滑落崖の試料を用いて様々な条件下で試験し、崩壊後に示される数秒間のせん断抵抗が土の密度を発達させた。しきい間隙水圧によりせん断開始直後に液状化した試料は、崩壊以前にもダイレタンシー傾向の強度が得られた。

キーワード:地すべり, すべり面液状化, ピーク強度, 変相点, 間隙水圧