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Liquefaction Potential of Granular Materials using Differently Graded Sandy Soils

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Synopsis

A series of undrained ring shear tests was performed on four categories of sand samples with varying grain size distributions to scrutinize their potential for liquefaction. Results of the tests indicate that well graded and gap graded specimens showed the greatest and the least resistance to static liquefaction against monotonic loading respectively. However, a series of tests conducted on the samples confined under a normal stress of about 200 kPa revealed that well graded specimens had lower values of residual shear resistance than the intermediately and narrowly graded specimens. Notable was the observation that there was a critical or limit value of pore pressure, above which all the samples, regardless of grade, suffered sudden collapse and liquefaction, and below which they dilated and gained some measure of strength. Specimens with a proposed dilation potential index, $r_f = \Delta\tau_p/\Delta\sigma'_p > 1$ experienced significant dilation, while those with critical dilation potential index, $r_f = 1$ dilated the least and demarcated the dilative from the contractive specimens.

Keywords: dilation, liquefaction, peak strength, phase transformation, pore pressure

1. Introduction

Landslides claim so many lives, decimate and disfigure the environment, and account for inestimable amount of property damage in virtually every part of the world. Their frequency and sheer ferocity have not only resulted in some acute socio-economic difficulties but have also brought mankind closer to environmental crisis. Liquefaction of saturated soils 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 in landslide investigation and mitigation

Considerable amount of research has been conducted on this subject, ever since the 1964 Alaska, U.S.A., and Niigata, Japan, massive earthquake-induced soils liquefaction, 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. Worthy of salute are sublime and prestigious publications by Castro (1969), Casagrande (1976), Poulos (1981), Poulos et al. (1985), Eckersley (1990), Marui (1996), Sassa et al. (1997a), and Sassa (1998a) which have formed excellent reference materials on the mechanism of landslides triggered by static loading. Equally notable are great works by Seed (1966), Seed and Lee (1966), Seed and Idris (1971), Castro and Poulos (1977), Ishihara et al. (1993), McRoberts and Sladen, (1992) and Sassa et al. (1996a and b) which have shed some profitable light on landslides induced by dynamic loading.

Although liquefaction phenomenon has been the subject of a barrage of investigations and publications for decades now, not much is known about the influence of particle size distribution on the mechanical behavior of granular materials, despite the universal knowledge that whether a material slides or flows is closely tied to its nature. Greater emphasis, it seems, has been placed on the influence of such other factors as void ratio, confining stress, and rate of loading than on particle characteristics. Yet, it is the thorough knowledge of particle characteristics, especially, particle size distribution, which may hold key answers to some of the hard puzzles surrounding the liquefaction of saturated cohesionless soils.

No researcher who investigates the subject of liquefaction can evade the ever- present questions – why really do some soils collapse and liquefy whereas others dilate and gain some measure of stability; and what are the primary factors triggering liquefaction and flow failures, especially, in loose cohesionless soils? 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. Attempts at solving the puzzle surrounding this subject had led Lindenberg and Koning (1981) to carry out wet and dry critical density tests using triaxial apparatus. Although they were able to observe liquefaction in a sample whose density was below a certain limit, and hardening in others with densities above the limit, they had to ultimately rely on mathematical calculations to arrive at 'a safe' critical density for the samples tested.

On the basis of experimental results, two new concepts – the concepts of least dilation, and critical pore pressure – are presented in this paper to interpret the undrained shear behavior of granular soils at critical density, and to use the characteristics of the soils so interpreted to define the boundary between contraction in loose, and dilation in dense soils held under same effective normal stress. The present study demonstrates that a critical state develops, at small strains, in sands at critical density, and that, this critical state unambiguously separates the contractive from the dilative soil specimens. It is shown that whereas specimens whose excess pore pressures at the phase transformation line (PT line) are equal to their corresponding shear resistances undergo the least dilation possible at any given confining stress, specimens whose excess pore pressures at the PT line are less than their corresponding shear resistances experience significant dilation. It is further shown that loose specimens do not pass through the so-called phase transformation because they rapidly generate enormous excess pore pressures from the outset of tests.

The new concepts have been tested on artificially-constituted silica sands, and on natural Osaka soils obtained from a landslide site.

2. Sample characteristics and test program

Four kinds of samples were constituted to study the influence of particle size distribution on the mechanical behavior of granular materials. These are, well-graded sample, WG, with uniformity coefficient, Uc = 9.0, made by combining five sizes of silica sand in equal proportion. Intermediately graded sample, ING, with Uc = 4.5, made by combining three finer sizes of silica sands in the ratio of 2:1:1. Narrowly graded sample, NAG, with Uc = 3.3, made by combining two finer sizes of silica sand in equal proportion. Gap graded sample, GAG, with Uc of 17.5, made by combining two silica sand sizes, the smallest and the largest sizes, in equal proportion. The grain size distribution curves of the samples are shown in Fig.1. A series of undrained, torque-controlled tests was conducted on fully saturated, normally consolidated specimens. Shearing was performed by incrementally loading shear stress at the rate of 0.098kPa/sec.

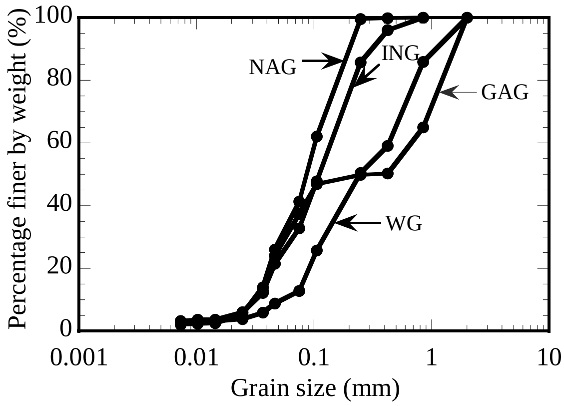


Fig. 1 Grain size distribution curves of the samples

3. Undrained shear tests

Results of the tests clearly indicate that, for specimens confined under a normal stress range of 100 – 374 kPa, well-graded specimens have higher values of peak resistance than the rest of the specimens, Figs. 2 and 4. However, contrary to some works in the literature, test results show that for specimens confined within a normal stress range of 196 – 202 kPa, well graded specimens have significantly lower values of residual shear resistance than the intermediately or narrowly graded specimens, Fig. 5. Worthy of attention are the sensitivity and responses of the materials to variations in relative density.

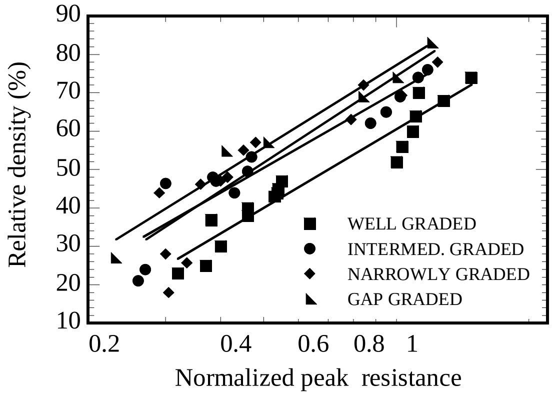


Fig. 2 Normalized peak strength versus relative density

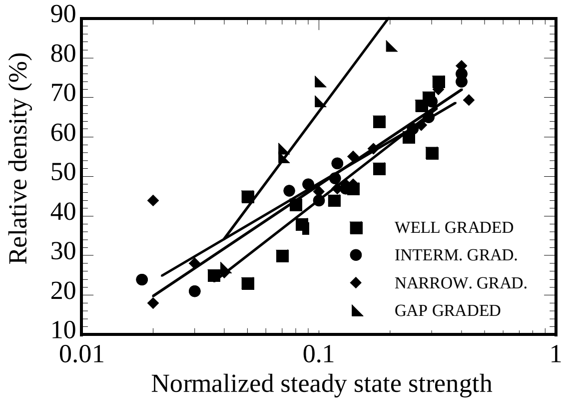


Fig. 3 Normalized steady state strength versus relative density

It may be observed from Figs 2, 3, 4, and 5 that the values of peak and residual resistances of all the specimens are very close at lower densities. As relative density becomes greater, these strength values begin to differ considerably. Even more significant is the fact that well graded specimens have considerably higher peak strengths than the rest, at higher densities. But, at the same the time, results of the tests conducted at a normal stress range of 196 – 202 kPa show they also have considerably lower residual resistances than the intermediately and narrowly graded specimens. Particularly interesting is the revelation that the gradation of a given mass of soil might be as important as the grain sizes it is composed of; and that the one cannot be discussed in partial or total isolation of the other. It may be noticed from the figures above that the behavior of narrowly graded specimens, which are composed of only two sizes of silica sand particles, while being considerably different from the behavior of gap graded specimens, which are also composed of two sizes of silica sand particles, closely resembles that of the intermediately graded specimens which are composed of three sizes of silica sand particles.

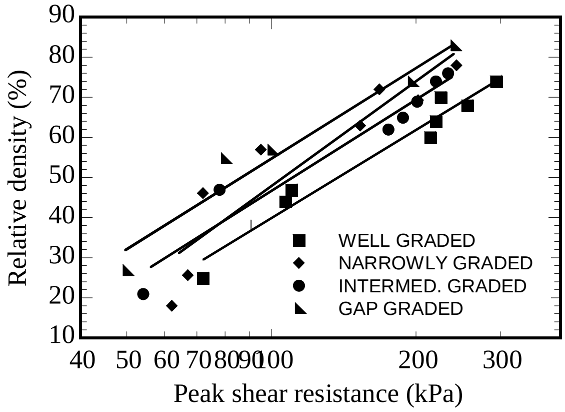


Fig. 4 Peak strength of specimens confined within a normal stress range of 196 – 202 kPa

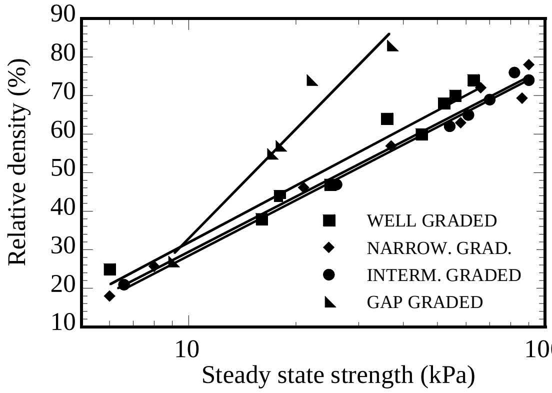


Fig. 5 Steady state strength of specimens confined within a normal stress range of 196 – 202 kPa

These differences in behavior not only suggest that the mechanisms of deformation in these samples are different, but that the separate effects of grain size, and grain size distribution on the shear behavior of any material composed of more than one grain size may be difficult to isolate. It is also clear, from the results, that although well graded specimens, on the basis of their peak strength, offer greater resistance to static liquefaction than the rest of the specimens, the values of their residual resistance should be a source of concern from the viewpoint of public safety because of their potential for large travel distances. Stronger effective interlocks existing between the many grain sizes in the well graded specimens might be responsible for their higher peak strengths, while grain crushing, and particle breakage, which tend to be more pronounced in larger grains than on smaller particles, may be responsible for their relatively low residual resistance. It is also possible that the interlocks existing between the particles of gap-graded specimens are much weaker than those existing between the particles that make up the narrowly graded specimens. This, and other factors including particle breakage and crushing, may also be responsible for the behavior of gap graded specimens which may be worthy of more investigations.

3.1 Liquefaction in loose specimens

The behaviors of the specimens in loose state deserve some attention. Fig. 6a shows the stress paths of loosely deposited specimens of WG, Dr = 25%; ING, Dr = 21%; NAG, Dr = 18%; and GAG, Dr = 27%. Fig. 6b shows the relationship between excess pore pressure, and shear displacement. It is clear from the Figures that the specimens suffered liquefaction as recognized by several other researchers including Castro (1969), Casagrande (1976), Poulos (1981), Poulos et al. (1985), largely because of the rapidity and magnitude of excess pore pressure they generated as loading progressed. Excess pore pressure rose suddenly, reaching nearly the same value with the normal stress, in a behavior that closely mirrors sand boils. This kind of behavior is known to be responsible for the occurrence of flow slides which are often catastrophic in nature.

Although these loose specimens are closely related by their flow-liquefaction behavior, some important differences between them remain; and can be traced to their differences in grain size distribution. It may be noticed, for instance, that while the pore pressure evolution of WG, ING, and NAG specimens follow a similar trend, that of the GAG follow a strikingly different trend, perhaps in obedience to their grain size distribution curves which show gap graded sample is in a totally different class. All but gap graded sample are made of particles that smoothly and gradually grades into different sizes. The gap in size between the particles comprising gap graded specimens may account for this remarkable trend of excess pore pressure generation.

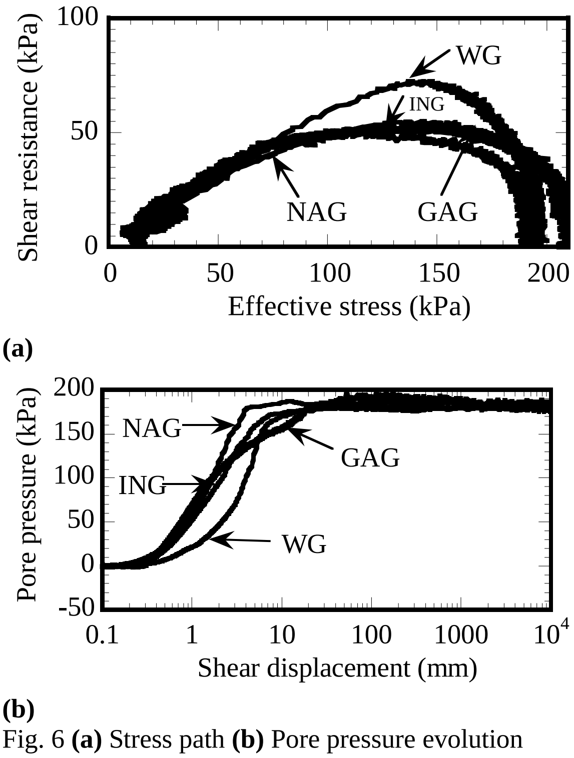


Fig. 6 **(a)** Stress path **(b)** Pore pressure evolution

4. Conditions necessary for liquefaction

The conditions necessary for liquefaction and the mechanism underlying the liquefaction of saturated soils subjected to dynamic or static loading have been widely discussed by a great deal of researchers, including Casagrande (1936, 1976), Terzaghi and Peck (1948), Seed (1966, 1979), Castro (1969, 1975), Seed and Idris (1970), Whiteman (1971), Ambraseys (1973), Gilbert (1976), Poulos (1981), Kutter (1982), Poulos et al. (1985), Eckersley (1985), Gilbert and Marcuson (1988), Ishihara et al. (1990), Ishihara (1993), Marui (1996), Sassa et al. (1996 a, b), Sassa et al. (1997a), and Sassa (1998a).

It has been well established that the void ratio or relative density of the soil, the confining stress on the soil, the intensity and duration of ground shaking, are important factors (but not the only factors) determining the liquefaction susceptibility of a saturated soil. While highlighting this importance in their distinguished publication, Kramer and Seed (1988) demonstrated the dramatic effect of relative density, confining stress, and initial shear stress on the static liquefaction resistance of some granular materials. Painstaking field investigations and laboratory tests, by numerous other researchers, have also provided results that indicate liquefaction is, primarily, associated with loosely deposited, poorly graded sands and silts.

In spite of widespread publications on the subject of liquefaction, the exact mechanisms controlling contraction in loose soils and dilation in dense ones remain a mystery. Many have postulated that the breakdown of a metastable structure in loose soils in association with excessive volume changes, during loading, ultimately lead to collapse and liquefaction. While the validity or the lack of it, of this and other postulates is far beyond the scope of this paper, a new perspective on the possible reasons for liquefaction and dilation is presented.

5. The Concepts of least dilation and critical pore pressure: when a failure line coincides with the phase transformation line

5.1 Background

Three basic behaviors of granular materials are very commonly referred to in geotechnical discourse: dilation, limited or partial liquefaction, and liquefaction, Fig. 7. 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 below are a very useful means of characterizing granular soils. The phase transformation line (PT line) as recognized by Ishihara 1993, is a line running through points where contractive behaviors terminate and dilative behaviors begin, in specimens that first contract, and then dilate. 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.

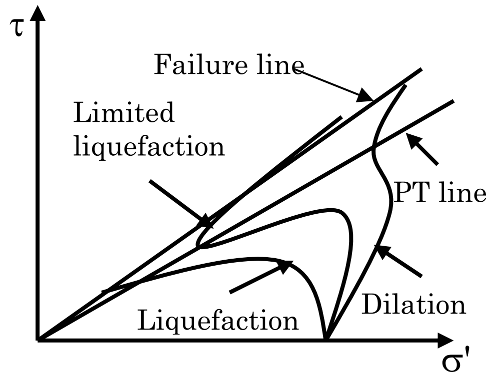


Fig. 7 A sketch of the three basic behaviors of granular materials (after Castro 1969)

5.2 Theoretical conception

Normally consolidated soils (Figure 8a, 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 the dilation potential index, rf, ($r_f = \Delta\tau_p/\Delta\sigma'_p$) are > and = 1 respectively. The conditions prevailing at 8b 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. 8c, the specimen will not go through the phase transformation stage because its r_f would be less than one. The specimen will, instead, collapse and liquefy. The present theory underlines the fact that the magnitude of excess pore pressure form the outset of 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 $\Delta\tau_p/\Delta\sigma'_p$ at the phase transformation line. If this ratio is unity, pore pressure and shear resistance will 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. This condition will define a critical situation. All other stress paths above this critical will dilate, while other stress paths below it will show contractive behavior. The pore pressure at which his critical is observed is a critical pore pressure. If the ratio is greater than one at the phase transformation line, the material will dilate significantly and its PT line will be different from its failure line.

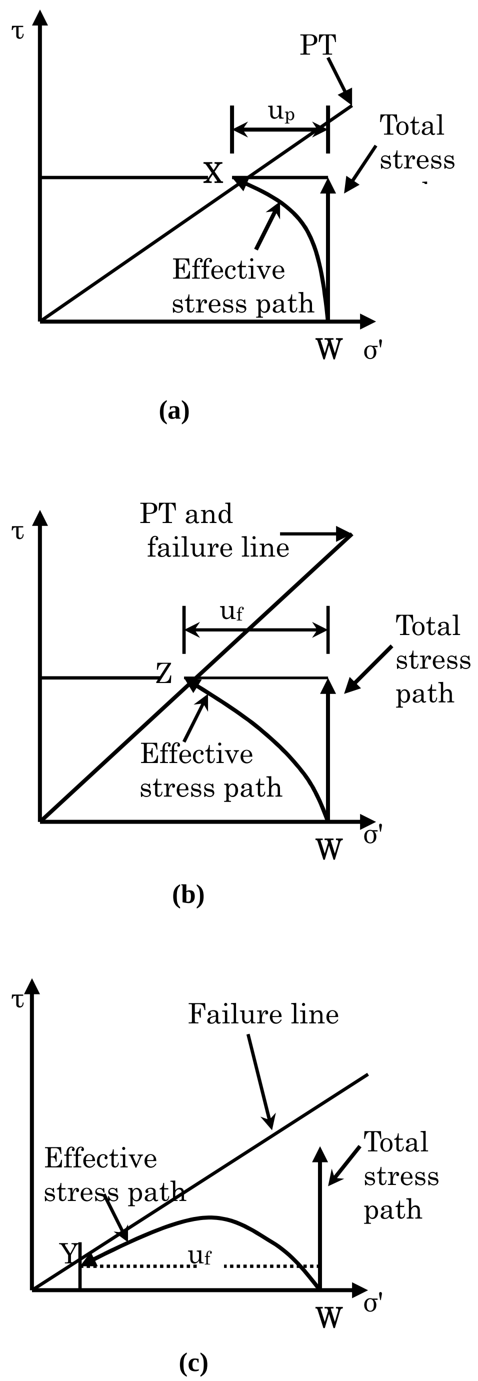
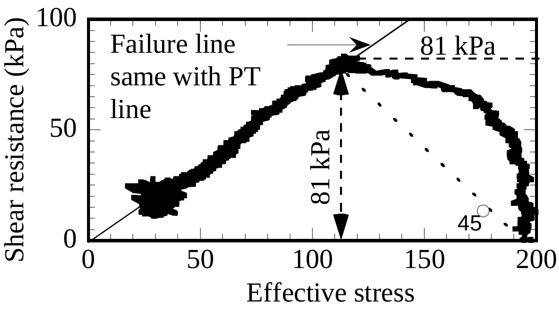


Fig. 8 Schematic diagrams illustrating the concepts of least dilation and critical pore pressure **(a)** dilation **(b)** critical **(c)** liquefaction

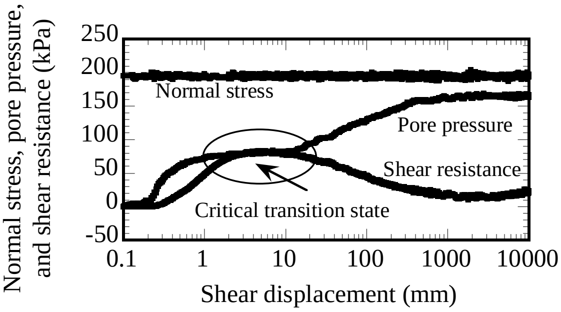
5.3 Experimental verification

5.3.1 When pore pressure and shear resistance at PT are equal

Artificially constituted silica sands and natural samples (Osaka samples) taken from a landslide site in Japan were used to verify the concepts. Figures 9a 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 phase transformation point and failure because the specimen experienced the least dilation possible at the given confining stress. Pore pressure and shear resistance became equal at the phase transformation point, and because of this, they remained the same until failure. 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 9b 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 behavior was found to be true in Osaka specimens confined at 372 kPa and consolidated to a void ratio of 0.77, Figures 10a and b.

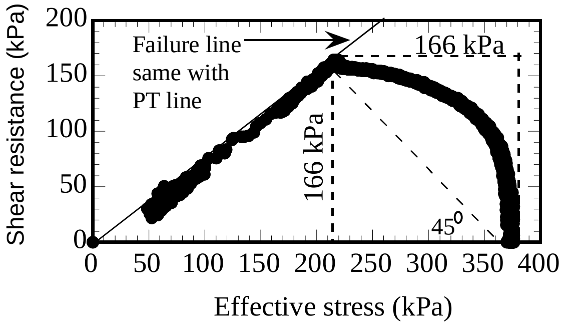


(a)

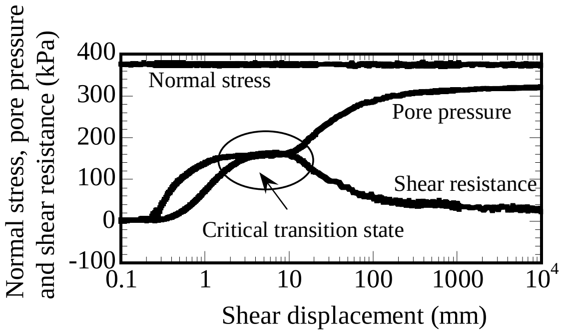


(b)

Fig. 9 (a) Stress path defining a critical condition (b) Pore pressure and shear resistance behavior.



(a)



(b)

Fig. 10 **(a)** Stress path defining a critical condition **(b)** Pore pressure and shear resistance behavior as loading progressed.

The validity of the concepts under consideration were also tested at other normal stresses and found to be true in all cases. 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. Osaka specimens are shown in Fig. 11 while Silica sand specimens are displayed in Fig. 12. The specimens, experienced least dilation possible, at the given normal stresses, whenever pore pressure equaled 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.

Since the undrained stress path followed by a specimen, at a given confining stress, to failure is, to a large extent, a function of void ratio and excess pore pressure, it is reasonable to state that at any given effective confining stress, there is only one stress path which pore pressure at failure will be equal to its corresponding shear resistance. The parameters of such a stress path have been used to define the boundary between dilating specimens and liquefying ones.

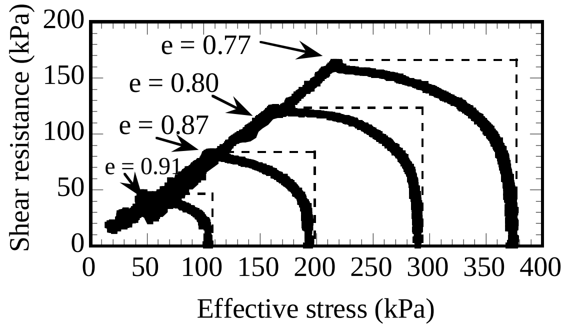


Fig. 11 Critical stress paths of Osaka sand specimens

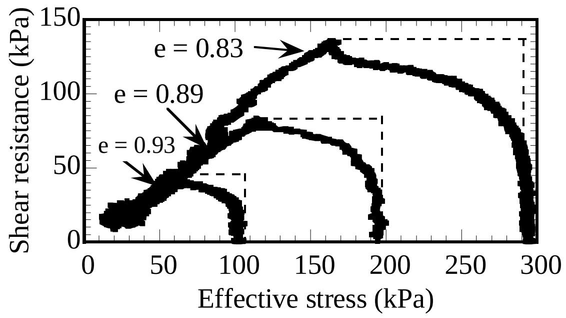
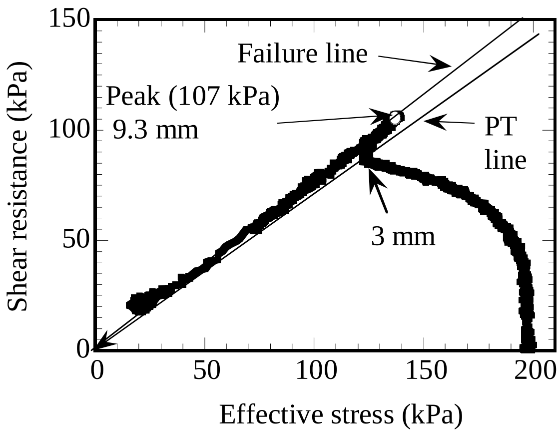


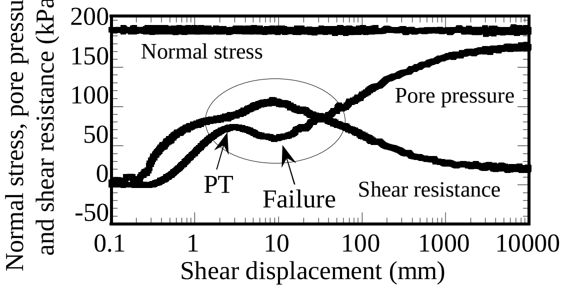
Fig. 12 Critical stress paths of Silica sand specimens

5.3.2 When pore pressure and shear resistance at PT are not equal

One case, among many cases, that typify situations where pore pressure at the phase transformation stage is not equal to the corresponding shear resistance is illustrated in Figs 13a 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 13b 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. 13a 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 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. This situation establishes and unambiguously defines a critical 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 Figure 14. 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 the present paper that this is one of the conditions, if not the only condition, that can lead to 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 at a given effective normal stress.



(a)



(b)

Fig. 13 **(a)** Typical stress path showing significant dilation because pore pressure and shear resistance at PT are not equal **(b)** Stress versus shear displacement

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.

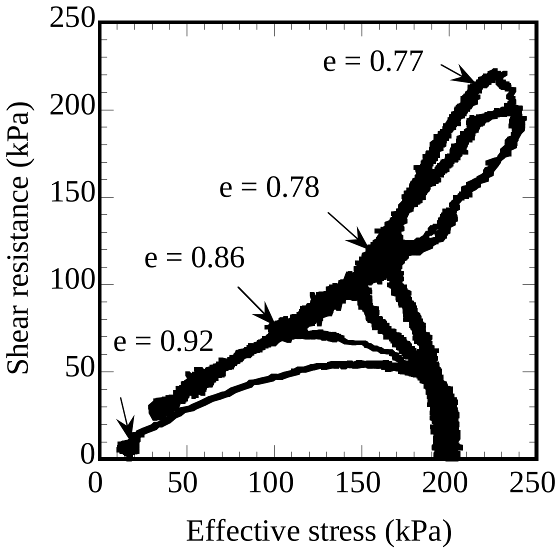


Fig. 14 Stress paths illustrating the influence of density on the behavior of silica sand materials

On the strength of laboratory evidences, it is possible to state that whenever the PT line coincides with the failure line of a granular material 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 colleagues elsewhere.

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: $\mathbf{u_c = \sigma\tan\phi/1+\tan\phi}$.

Where $\mathbf{u_c}$ 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.

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.

This may probably be the first documented attempt at inserting into the Mohr Coulomb equation equal values of shear resistance and pore pressure at failure for granular materials; and using the resultant equation to predict the behavior of a critical stress path. It may be interesting to note that with the behavior of a critical stress path 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 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.

6. Conclusions

Results of the tests indicated that well graded and gap graded specimens offered the greatest and the least resistance to static liquefaction respectively

2. The residual shear resistance of well-graded specimens should be a source of concern from the viewpoint of public safety because of their potential for large travel distances

3. Whether or not a specimen liquefies seems to depend more on relative density rather than on gradation.

4. Results show that there is a critical or limit value of

pore pressure, above which all the samples, regardless of grading, suffered sudden collapse and liquefaction, and below which they dilated and gained some measure of stability.

5 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.

6. Once pore pressure becomes equal with the corresponding shear resistance at the PT point, they will remain the same until failure and ensure that the specimen dilates the least.

7. The new concept highlights 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

8. 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 a stress ratio which value is unity should be considered a crucial boundary distinguishing two very important soil behaviors – liquefaction and dilation

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粒度の異なる砂質土を用いた粒状体材料の液状化ポテンシャル

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要旨

本研究では、砂試料の液状化ポテンシャルを調べるために粒径分布の異なる４種類の砂試料で非排水リングせん断試験を行った。単調載荷試験の結果では、静的な液状化に至るまでに、粒度が良い試料で最も大きな抵抗が、粒度が悪い試料で最も小さな抵抗がそれぞれ発揮された。しかし、垂直応力約200ｋPaでの試験では、残留強度は粒度が悪い試料よりも粒度が良い試料の方が小さくなることが明らかになった。またある限界の間隙水圧値があり，これより大きな間隙水圧が発生すると粒度にかかわらず全ての試料が突然コラプス的挙動を示し液状化した。またこの値よりも小さな間隙水圧下では、試料は膨張し強度が上がった。今回新たに提案した膨張ポテンシャル指数$r_f = \Delta\tau_P/\Delta\sigma_P$に対して$r_f > 1$である試料は著しく膨張し、一方で限界膨張ポテンシャル指数$r_f = 1$を示す試料は最も膨張率が小さく、その後の収縮挙動と膨張挙動の境界を示すことがわかった。

キーワード：膨張，液状化，ピーク強度，変相点，間隙水圧