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Ibsen, Lars Bo

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The Stable State in Cyclic Loading

Lars Bo Ibsen

Soil Mechanics Laboratory, Aalborg University, Denmark

ABSTRACT

The behaviour of sand during cyclic loading has been studied in the Danish Triaxial Cell (smooth pressure heads and specimens with equal height and diameter) under different anisotropic stress levels. A new characteristic state "The Cyclic Stable State" has been discovered. The cyclic stable state occurs when the positive and negative pore pressures generated during a loading cycle neutralize each other. The mean normal stress does not change when the stable state has been reached and the cyclic loading will not lead to any further hardening or softening of the soil. If cyclic tests are executed at different anisotropic stress levels the stable states describe a line in the stress space "The Cyclic Stable Line". If the anisotropic mean stress level is situated below the stable line the cyclic test will try to reach the equilibrium of the stable state. The test will generate positive pore pressures and the phenomena "Pore Pressure Buildup" or "Cyclic Liquefaction" will be observed. If the anisotropic stress level is situated above the stable line the cyclic test will generate negative pore pressures and the phenomena "Stabilization" or "Instant Stabilization" will be observed. In the paper these results will be compared with results from tests executed on specimens with double height. It will be explained why the phenomena "Stabilization" and "Instant Stabilization" seldom is observed when the test is executed on specimens with double height. The conclusion recommends that the study of pore pressure variation due to cyclic loading should only be performed with smooth pressure heads and specimens with equal height and diameter because the phenomena leading to stress hardening would otherwise not be observed.

INTRODUCTION

During the last thirty years a great number of test series with cyclic loading of sand have been performed in the laboratory. Laboratory testing gives a possibility to study soil behaviour in details under control and well known conditions. However, it is not possible to use laboratory test results for practical purposes without a high correlation between the test condition in the laboratory and the condition in-situ.

Great efforts have been made to develop test techniques where the stress history can be reconstructed by anisotropic consolidation before cyclic testing. Techniques, which ensure a high degree of saturation and homogeneously prepared specimens, have been developed very carefully so that the proper initial condition for correct pore pressure generation is present.

However, very little has been attempted to ensure homogeneously distributed stresses and strains, so that the variation of the pore pressure will develop correctly throughout the tests. The aim of this paper is to describe the importance of studying the cyclic behaviour of soil under uniformed stress and strain conditions. Studying the basic phenomena in cyclic triaxial tests under uniformed conditions new phenomena and thereby new responses of the soil due to cyclic loading have been discovered. These new phenomena are described and they give a starting point for developing a new theory which describes fatigue in sand due to cyclic loading. This new fatigue theory is also advanced and it will be explained why the new phenomena are not observed when the cyclic tests are executed on specimens with the height equal to two diameters.

HOMOGENEOUS STRESS AND STRAIN CONDITIONS

In triaxial tests – in which the loads, movements, volume- and pore pressure changes are measured on the outside of the specimen – it is obvious that homogeneous conditions must exist inside the specimen throughout the test in order to calculate the correct values of stresses, strains and void ratio. Although it has been emphasized for at least 20 years that triaxial tests must be performed on specimens with a height equal to the diameter and smooth pressure heads, Jacobsen, M. (1970), Jacobsen, M. (1981), Lade, P.V. (1982), it is generally considered that adequately uniform conditions will be achieved by using tall specimens with heights greater than or equal to two diameters and fixed or lubricated caps and bases.

Today it is still common to perform triaxial tests with rough pressure heads. If the pressure heads are rough, large porous stones with the same diameter as the specimen, stiff bodies are created at the ends, see Figure 1b. The stress state appears to be very complex with singularities along the edges. The stress state is not homogeneous because the stiff bodies

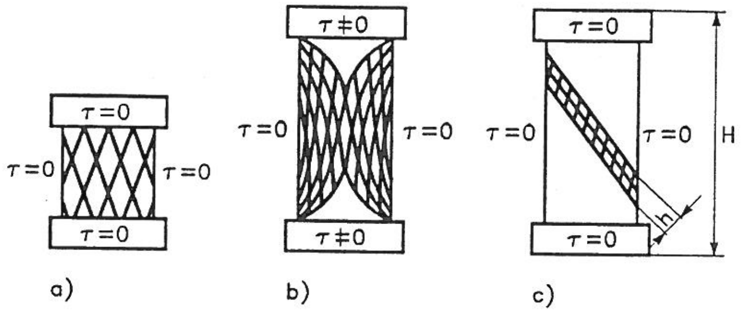

Figure 1. Outline of a triaxial test run with a) Homogeneous stress and strain conditions, zone failure occurs. b) Inhomogeneous stress condition as a result of rough pressure heads. c) Line failure can occur under nonuniform strain condition.

reinforce the specimen. If the tests are run on a specimen with the height equal to the diameter the strength of the soil will be heavily overestimated. This is generally accepted and tests run with rough pressure heads are always carried out on tall specimens to minimize the effects of the fixed ends. However, in unstable material the influence of the fixed ends will be measurable even when the test is carried out on a tall specimen, see Figure 2. Even in stable soils the stiff bodies will appear and this test type ought not to be used to study basic phenomena in soils.

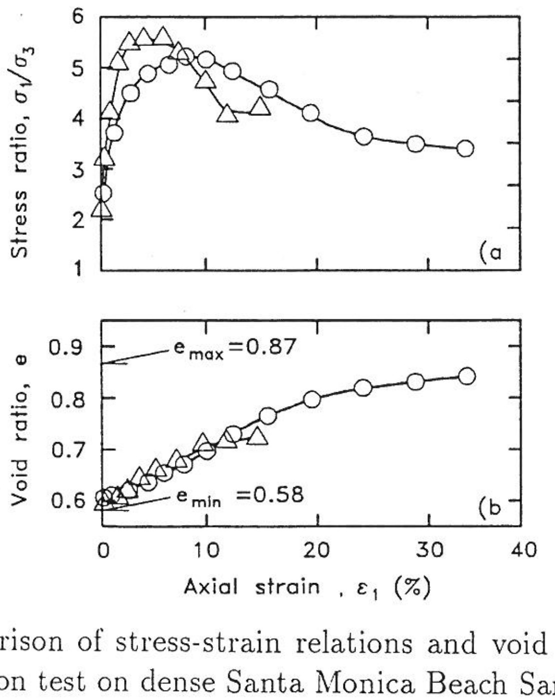

Figure 2. Comparison of stress-strain relations and void ratio changes in triaxial compression test on dense Santa Monica Beach Sand $I_D = 0.9$ with $H/D = 1.0$ marked ○ and $H/D = 2.7$ marked △, Lade, P.V. (1982).

If a homogeneous stress state is to occur, the surface load must satisfy the statical condition. The specimen is horizontally loaded by the cell pressure and the surface is a principal plane, $\tau = 0$, see Figure 1. To satisfy the statical condition the surface along the pressure heads must be principal planes too. To obtain a principal plane along a pressure head the pressure head must be smooth. The smooth pressure head consists of a glass plate with silicone grease and thin rubber membranes, Rowe and Barden (1964). The proposed system is naturally not quite frictionless but it is sufficient if the roughness is negligible. This paper is based on tests run on sand with $d_{50} = 0.4$mm and by using a sandwich of two layers of grease and rubber membranes, τ is found to be less than 0.1% of σ_1' and therefore negligible.

The relative height of the specimen does not play any role for the stress distribution inside a specimen bounded by smooth pressure heads, but severe nonuniformities in strain can develop if the height of the specimen is greater than the diameter, Lade, P.V. (1982), Jacobsen, M. (1970). After testing – especially in tests on firm soil – the normal shape of a tall specimen shows failure as a narrow ruptured zone where two practically solid bodies slide past each other, see Figure 1c and 3d. This ruptured zone is normally called line failure and it inclines 45° - $\phi/2$ to the vertical, when ϕ is the angle of internal friction.

The main shear deformations and volume changes are located in this zone of the line failure, and the soil properties measured do not refer to the whole specimen – as normally assumed – but only to the material sheared in this narrow failure zone. As a consequence the vertical deformation is dominated by the movements in this zone and the strain ought to be relative to the height h of the line failure and not to the height H of the entire specimen, see Figure 1c. Unfortunately the height of the line failure is unknown so the height of the specimen is always used. Consequently the performance curve will be too short, and the compression phase is only correct at the beginning of the test, see Figure 2a. The measured curve is seen to have very little to do with the real performance curve for the soil.

If the heterogeneous strain state is permitted to be developed the volume changes will also be concentrated at the narrow failure zone and only small volume changes occur in the sand mass outside the slip zone. Once the sand in the line failure zone has expanded to the critical void ratio no further volume change takes place. Figure 2b indicates that the maximum dilation of the line failure specimen has essentially ceased at about 12% axial strain. Whereas in the specimen with zone failure volume changes occur and continue even at 35% axial strain. As the volume change measured in a triaxial test represents the average volume change for the entire specimen, the final average void ratio for the line failure specimen is seen to be much smaller than the actual critical void ratio for the sand, at a given confining pressure.

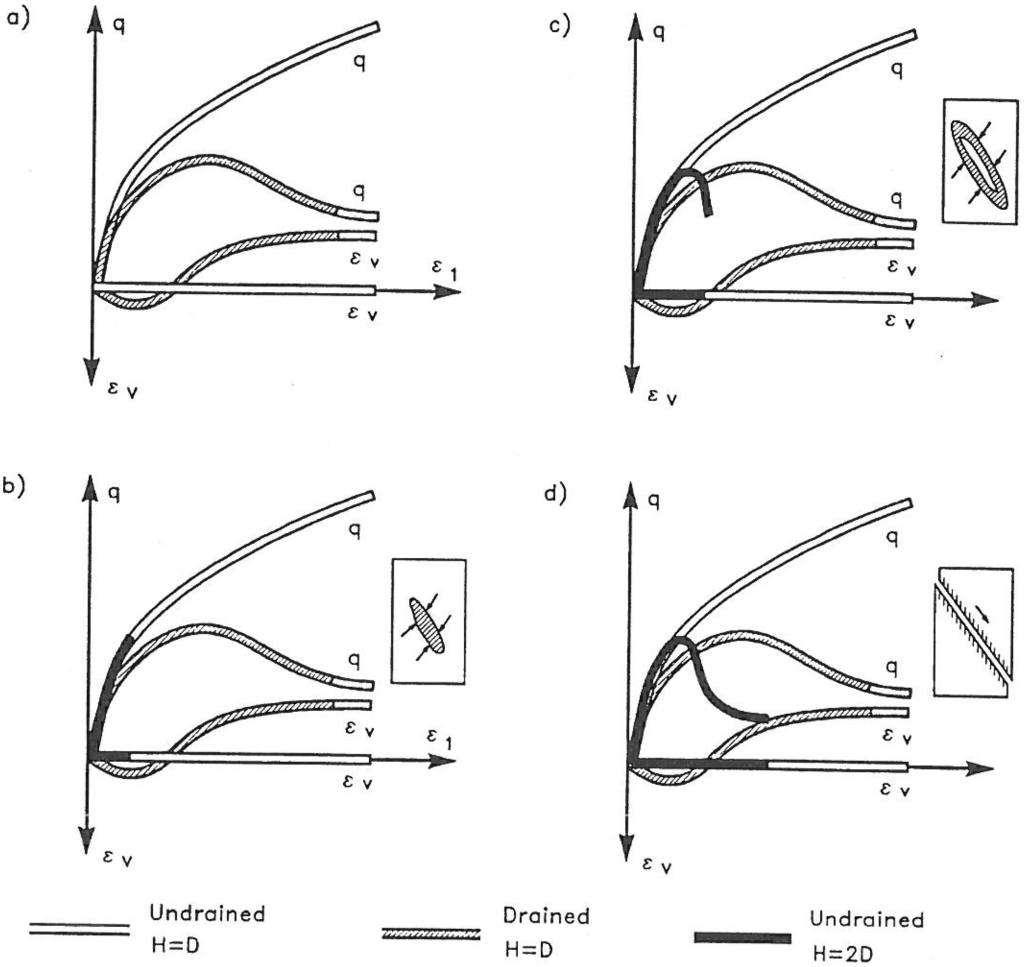

Figure 3. The consequence of an undrained test run on a specimen with double height. a) Drained and undrained test run under homogeneous strain condition. b) Development of the locally weakened zone in which the shear plane subsequently is formed. c) The drained zone dominates the performance curve. d) Two practically solid bodies sliding past each other. H = height and D = diameter of the specimen.

The double height specimen's incapability to produce uniform deformation condition throughout the test has considerable influence on the results in undrained tests. In Figure 3a test results of drained and undrained triaxial tests, run on specimens with ensured homogeneous stress and strain distribution throughout the test, are outlined. The same soil will now be investigated in an undrained triaxial test with height of the specimen twice the diameter. Initially the test result follows the undrained performance curve. When the deviator stress q increases the soil will try to expand. Under homogeneous condition this expansion is impossible since the drains are closed and the pore water will hold the grain structure together. But in the double height specimen the deformation is not homogeneous and some local zones will dilate and others will contract. Inside the sample water will flow from the zones which contract to the zones which dilate, see Figure 3b. As a consequence the test is no longer undrained even though the volume of the whole specimen is kept constant. The performance curve is dominated as the drained zone develops, see Figure 3c, and it will take form after the performance curve in Figure 2a. The critical state develops inside the drained zone as the test continues. The well known failure surface appears at the surface of the sample and two practically solid bodies slide past each other, see Figure 3d.

As a consequence of the inner draining the undrained shear strength, obtained in double height tests on Danish moraine clay, may be only 40% of the results obtained under homogeneous conditions or by vane and plate tests, Jacobsen, M. (1970). In the study of pore pressure variation due to cyclic loading it is important to realize that the locally weakened zone – in which the shear plane subsequently is formed – develops in a double height specimen, even at very small strains. The zone is very narrow and needs only a small amount of water to expand. The negative pore pressure which prevent dilatation under homogeneous condition cannot be produced as a consequence of inner draining.

It is shown that the nonuniformities in strains have a very significant effect on the strength, the stress-strain performance curve, the volume change or on the pore pressure generated and its development should not be permitted. The solution to this problem is to enforce uniform deformation condition at failure. By giving the specimen a height equal to the diameter the creation of the narrow failure zone is prevented.

The consequence of the errors described is that studies of basic phenomena in soil, drained or undrained, must be performed on specimens with a height equal to the diameter and with smooth pressure heads. This is the only way to ensure that theories do not reflect test errors but the real properties of the soil.

TEST PROGRAM

The tests performed and described in this paper are normal static CD-, CU-tests and cyclic triaxial tests. To ensure homogeneous stress and strain conditions the tests are performed on cylindrical specimens with a height of 70mm and a diameter of 70mm, and bounded by smooth pressure heads. The tests are run in an automatic, newly developed version of the Danish Triaxial Apparatus. The dynamic triaxial apparatus – in which the cyclic tests are run – is outlined in Figure 4.

The difference between the static and the dynamic device is that the dynamic triaxial apparatus is loaded by hydraulic. The measuring systems are identical and carried out by transducers. The triaxial cell is constructed by M. Jacobsen in agreement with the principles described in Jacobsen, M. (1970).

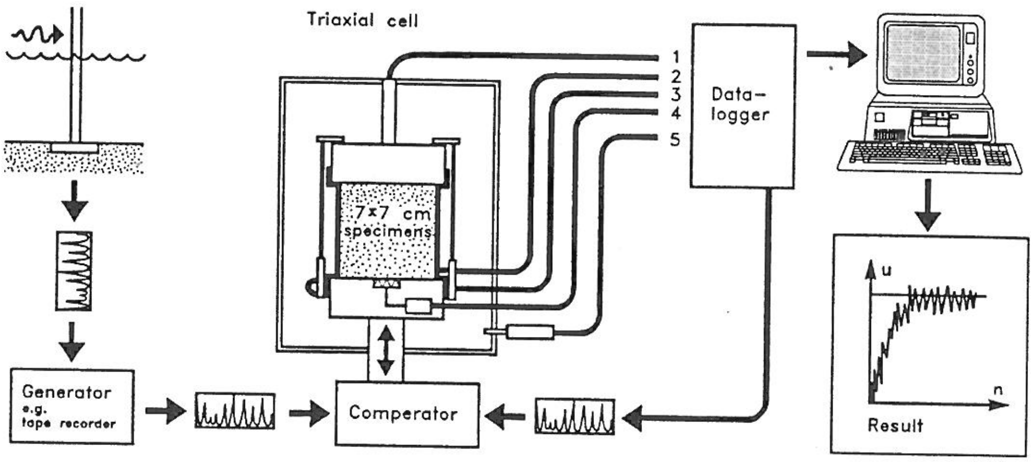

Figure 4. The dynamic triaxial apparatus is fully automatic and run by a computer. During the tests 1) vertical load 2) and 3) vertical deformation 4) pore pressure 5) cell pressure are measured and recorded.

The study described in this paper is based on tests run with a uniform sand called Lund No 0. Its index properties are: the mean diameter $d_{50} = 0.4$mm, the coefficient of uniformity $U = 1.7$, $e_{max} = 0.82$, $e_{min} = 0.55$. The test specimens were prepared by a pluvial technique and carefully saturated in vacuum. This technique ensures homogeneous and totally saturated specimens.

The static triaxial CD- and CU-tests were performed on isotropically consolidated samples with four relative densities I_D of 1.0, 0.93, 0.78 and 0.48. The cyclic triaxial tests were performed both on isotropically and anisotropically consolidated samples with a relative density $I_D = 0.78$, and the sinusoidal cyclic load was applied at a frequency of 0.1 Hz under undrained conditions.

The parameters, which describe the state of the soil under axisymmetrical stress conditions, are calculated from the measurements taken during the tests. These parameters are:

the mean normal stress $p' = 1/3(\sigma_1' + 2\sigma_3')$

the deviator stress $q' = (\sigma_1' - \sigma_3')$

the volumetric strain $\epsilon_v = \epsilon_1 + 2\epsilon_3$

the deviator strain $\epsilon_q = 2/3(\epsilon_1 - \epsilon_3)$

where σ'_1 is the vertical and σ'_3 the horizontal principal stresses. The stresses are effective and ϵ_1 and ϵ_3 are the principal strains.

THE CHARACTERISTIC STATE

Figure 5 shows the results of four drained triaxial CD-tests. The tests are performed with different levels of cell pressure σ'_3 which are held constant throughout the test. In each test failure is seen to be well defined as the state in which the deviator stress q' is maximum, see Figure 5b. In the figure the performance curve is divided by σ'_3 which causes the curve with the smallest confining pressure to be situated highest. The strain at failure is seen to be a function of σ'_3. Failure normally corresponds to a deviatoric strain ϵ_q = 5 - 10%. The values p'_f, q'_f at failure describe a curved failure envelope, which defines the stress space, limiting the stress combination p', q', see Figure 5a.

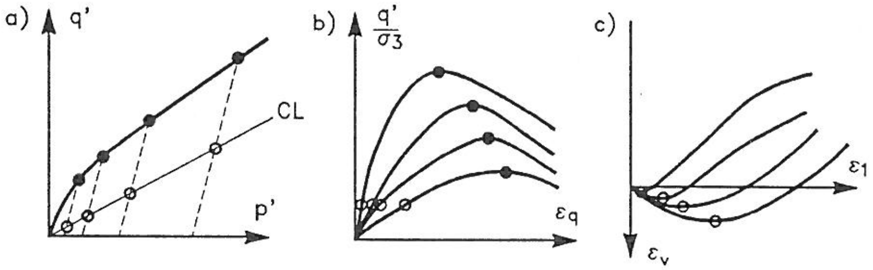

Figure 5. The figure outlines results of four drained triaxial tests. The tests illustrate the development of CD-tests carried out on dense sand performed with different levels of confining pressure, and on specimens with equal height and diameter.

Laboratory tests on several sands have shown that a characteristic threshold exists in granular materials which is defined as the stress state where the volume change goes from contraction to dilation. On the curve $\epsilon_1 - \epsilon_v$ curve, Figure 5c, the characteristic threshold is marked with open circles at the point in which the sample has minimum volume. The stress stage p'_c, q'_c where $\delta\epsilon_v = 0$ is defined and described as the CHARACTERISTIC STATE, Luong, M.P. (1980).

If the characteristic states are plotted in the stress space they describe a straight line through 0,0, see Figure 5a. The line is described as the CHARACTERISTIC LINE CL, Luong, M.P. (1980) and divides the stress space into two subspaces in which the stress combinations lead to different deformation mechanisms.

Under the characteristic line the stress combinations lead to contraction and $\delta\epsilon_v > 0$.

Above the line the stress combinations lead to dilation and $\delta\epsilon_v < 0$.

Under the characteristic line the resistance to deformation is governed by sliding friction due to microscopic interlocking depending upon surface roughness of contracting particles or interlocking friction afforded by adjacent particles. The resistance is due to pure friction and the characteristic state describes an intrinsic parameter which defines a characteristic friction angle ϕ_{cl} which determines the interlocking capacity of the grains. As ϕ_{cl} is a token of pure friction the value of ϕ_{cl} is independent of the initial sand density and the interlocking capacity is identical in triaxial compression and extension. These postulates have been verified by the four CD-test series executed and the observation is in agreement with Loung, M.P. (1980). The interlocking capacity for Lund No 0 is found to be $\phi_{cl} = 29.5°$. The stress combination situated in this subspace between $Cl+$ and $Cl-$ leads to no significant volume change as indicated in Figure 5c.

In the subspace situated between the failure envelope and the characteristic line the resistance to deformation is governed by interlocking disrupture where the individual particles are plucked from their interlocking seats and made to slide over the adjacent particles leading to large dilative volume changes. The resistance to the deformation and thereby the size of the subspace is strongly depending on the initial sand density as it requires more energy for the grains in a dense sand to move the adjacent particles while sliding over each other than for the grains in a loose sand. This subspace does not exist in a very loose sand where no dilation occurs and in this case the failure envelope will be identical to the characteristic line.

The characteristic state is an intrinsic parameter which is independent of the type of loading. In Figure 6 cycling sequences are carried out at different deviator stress levels under drained condition and at constant confinement $\sigma'_3 = 200$ kPa. Each cycling sequence consists of 20 cycles with an amplitude of 100 kPa. The figure shows very clearly that contracting behaviour of the soil is obtained each time the mean deviator stress level is lower than the characteristic level. The dilating behaviour of the soil during load cycling is evident each time the mean deviator stress level becomes higher than the characteristic threshold q'_c. Loung (1980) has described that the contraction decreases when the characteristic threshold is approached and dilatation increases when deviator stresses increase further. The contracting effect is more pronounced for extension loading having $q' < 0$. These conclusions are verified by the tests carried out on Lund No 0.

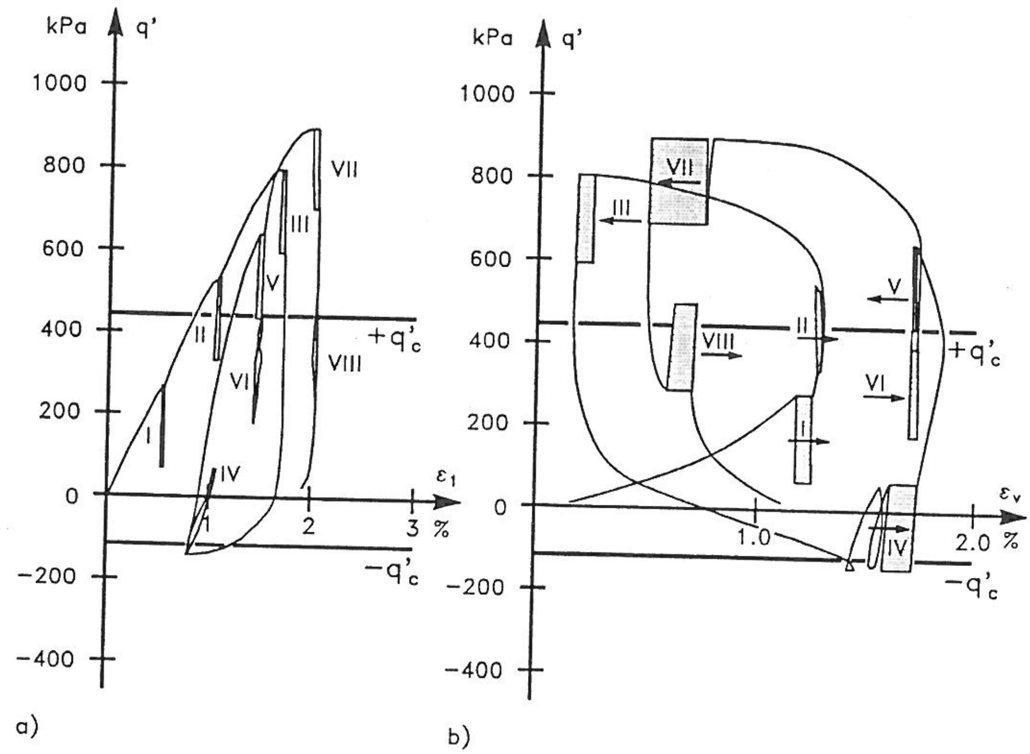

Figure 6. Cyclic loading under constant confinement condition $\sigma'_3 = 200$ kPa performed on Fountainbleau sand $I_D = 0.64$, Loung, M.P. (1980).

THE CHARACTERISTIC STATE UNDER UNDRAINED CONDITION

Under undrained condition the characteristic state is also an intrinsic parameter which controls the effective stress path.

During a conventional undrained triaxial test the pore pressure increases at first in order to prevent the sand from contraction and $\delta u > 0$. When the deviator stress approaches the characteristic line $\delta u \rightarrow 0$. In Figure 7, where results of four undrained triaxial $CU_{u=0}$-tests are shown it appears that the stress stage, where $\delta u = 0$, is identical with the interlocking capacity in the sand, see Figure 7.a. If q' increases further the effective stress path is situated in the subspace which is characterized by dilation. In order to prevent interlocking disrupture the pore pressure generation becomes $\delta u < 0$, see Figure 7.a.

The figure shows that the effective stress path approaches asymptotically to a common stress path. This common stress path is normally considered to be the undrained failure envelope. In Figure 7.b it is shown that the common stress path does not represent any failure state in the sand and tests, covering a σ'_3-interval from 5 kPa to 2000 kPa, do not unveil any

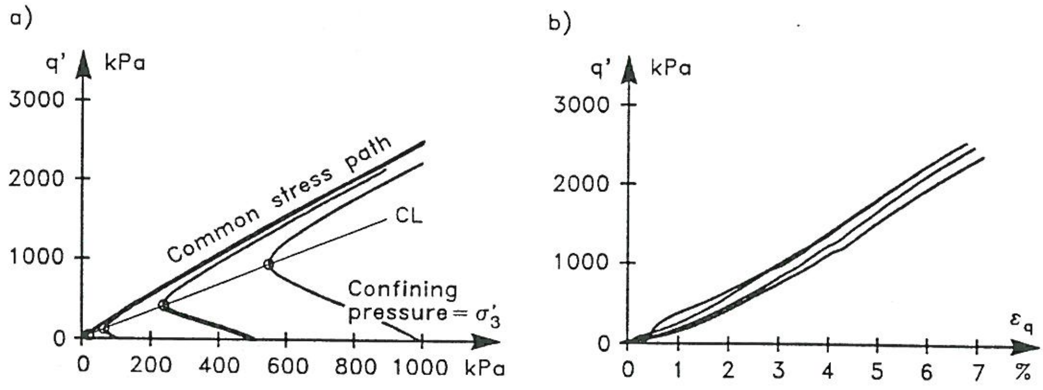

Figure 7. Results of 4 $CU_{u=0}$-tests performed on Lund No. 0 with I_D = 0.78. The tests are run on specimens with equal height and diameter.

maximum at the performance curves. This leads to the conclusion that stress defined failure in sand does not exist under undrained condition. On the other hand the common stress path is a result of the strain state in the material and failure must be defined in regard to a deformation criteria. This conclusion is not in agreement with the state of the art. The state of the art is build on undrained tests run on specimens with double height and thereby it reflects the errors described earlier.

Figure 8 shows the results of CU-tests run on a specimen with double height submitted to a) static loading and b) cyclic loading. Similar to the result in Figure 7 the pore pressure in Figure 8.a increases in the beginning, but as soon as the deviator stress approaches the characteristic line CL, the locally weakened zone starts to develop. The inner draining is now possible and the pore pressure, which prevent the interlocking disrupture under homogeneous condition, cannot be produced. As a consequence the effective stress path cannot be situated in the subspace, which is characterized by dilation as seen with the tests under homogeneous condition in Figure 7.a. In figure 8.a the test results show that the effective stress path follows the characteristic line CL when the characteristic threshold is reached, which has lead to the opinion that the undrained failure envelope describes a straight line through 0,0.

In the case of undrained cyclic loading the stress variation can only be situated in the subspace which is characterized by contraction and the stress variation will therefore always lead to pore pressure buildup. If the cyclic loading is reversal and the number of cycles is big enough the effective stress variation will approach the characteristic line CL. Similar to the static loading the locally weakened zone starts to develop and the stress variations are seen to follow the characteristic line in both compression and extension,

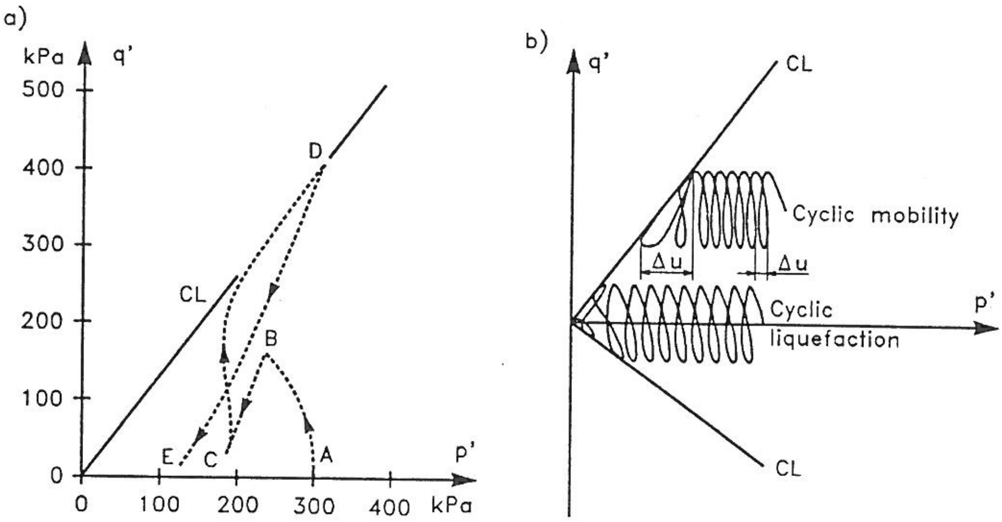

Figure 8. Results of *CU*-tests run on a specimen with double height submitted to a) static loading and b) cyclic loading. Luong, M. (1980)

see Figure 8.b. This characteristic phenomenon is described by many authors and is called Cyclic Liquefaction (initiated by Seed and Lee, 1966). If the cyclic loading is not reversal Cyclic Mobility can occur as described by Casagrande (1971). In both phenomena the shear plane is developed progressively as the cyclic stress variation continues to follow the characteristic line in each cycle. Large residual deformations are observed and the phenomena are normally considered a token of failure due to cyclic loading. As the effective stress variation follows the same effective stress path, both under cyclic and static loading, the "undrained failure envelope" – which is seen to be identical with the characteristic line – is widely accepted to control the development of failure due to cyclic loading, see for instance Gousmann et al (1988).

In the case of undrained cyclic loading on specimens with ensured homogeneous conditions the effective stress is situated in both subspaces and the interlocking capacity of the sand is constant throughout the test until failure occurs, see Figure 9. The loading part of the cycle is seen to pass by the interlocking capacity at the measure point 1.6, see Figure 9.a. In Figure 9.b this point is seen to define the stress state where δu goes from $\delta u > 0$ to $\delta u < 0$ and the stress state is seen to be identical in point 9.6. The characteristic state is seen not to influence the variation of the pore pressure at the unloading part of the cycle.

The drained failure envelope and the common stress path are shown in the figure. During cyclic loading the effective stress path is seen to cross

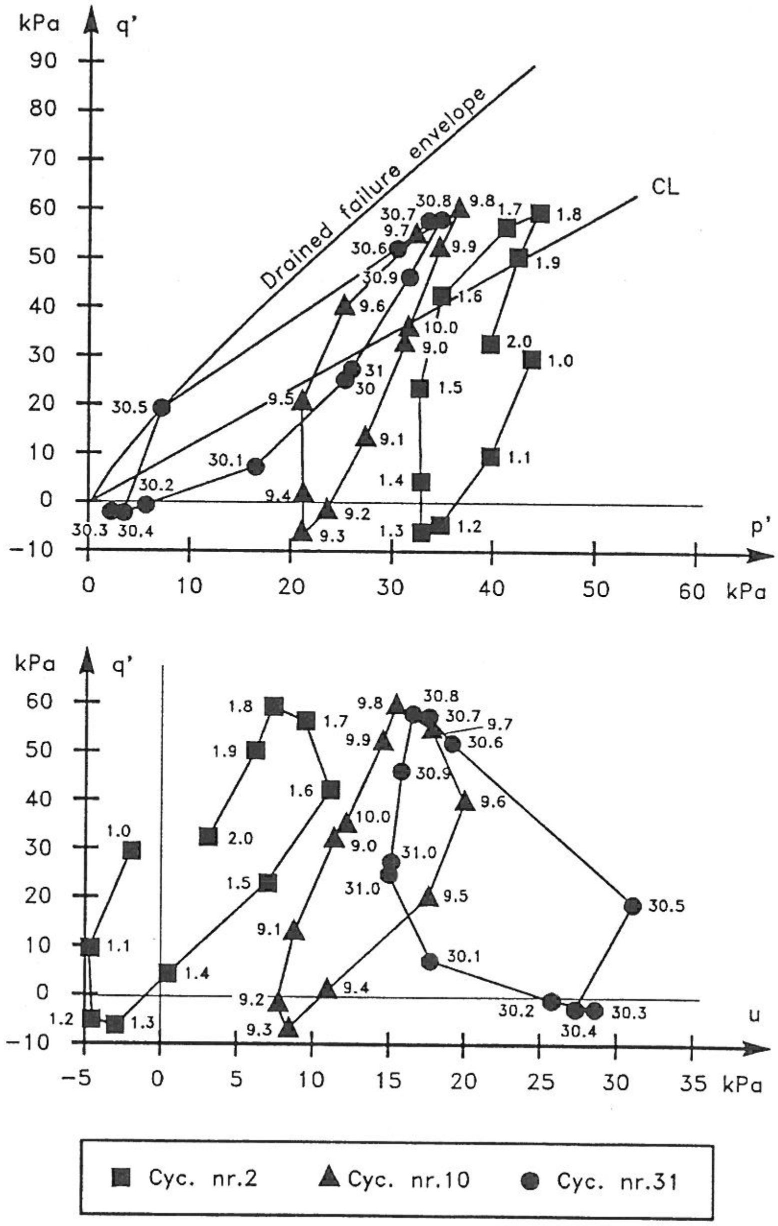

Figure 9. Selected cycles from a cyclic test on Lund No 0 are shown in the figure. The test is carried out on a specimen with equal height and diameter. The result is characterized as Cyclic Liquefaction.

the common stress path (undrained failure) and failure occurs when the effective stress part reaches the drained failure envelope. This observation confirms the postulate – that undrained failure does not occur in sand – and leads to the conclusion that failure due to cyclic loading is controlled by the drained failure envelope.

THE CYCLIC STABLE STATE

Similar to the drained cyclic test in Figure 6 – and with the experience from the static test in mind – the characteristic state could then be expected to control the stress hardening/softening in the soil. However, the changes of the effective stress path due to cyclic loading are composed of the pore pressure produced during the loading part of the cycle and the changes in the pore pressure during the unloading part of the cycle. In Figure 9 the characteristic state is seen only to control the changes in the pore pressure during the loading part of the cycle. Therefore, the characteristic state is not able to control the stress hardening/softening of the soil during a complete cycle.

It is assumed that a stress state exists, where the positive and negative pore pressure generated during a loading cycle neutralize each other. The stress state is defined as $\sum \delta u = 0$ during a cycle. The stress state is described as the Cyclic Stable State. Each time the mean deviator stress level is lower that the cyclic stable state positive pore pressure will be generated and negative pore pressure is generated each time the mean deviator stress level becomes higher. As a consequence the cyclic loading leads the mean normal stress towards the cyclic stable state in each test, see Figure 11, and when the cyclic stable state has been reached the mean normal stress does not change and the cyclic loading will not lead to further hardening or softening of the soil, see Figure 10. In Figure 10 the pore pressure u is seen to develop very rapidly during the first 45 cycles. After 160 cycles the cyclic stable state is seen to be fully developed and no further pore pressure develops even though the effective stress variation is situated very close to the drained failure envelope. The specimen can be exposed to several thousands of cycles without any further softening of the soil. In the figure the effective stress variation is seen to cross the common stress path without developing failure.

THE CYCLIC STABLE LINE

If cyclic tests are executed at different anisotropic stress levels the stable states describe a curved line in the stress space, see Figure 11. The curved line is described as the Cyclic Stable Line (CSL) and is seen to be situated below the Characteristic Line (CL) in Figure 12. The cyclic stable state is similar to the characteristic state (an intrinsic parameter) and it is found to be independent of the amplitude of the cyclic load. The cyclic stable line divides the stress space into two subspaces characterized by the response due to the cyclic loading. Similar to the characteristic line the cyclic stable line is found to be characterized by a constant stress path independent of

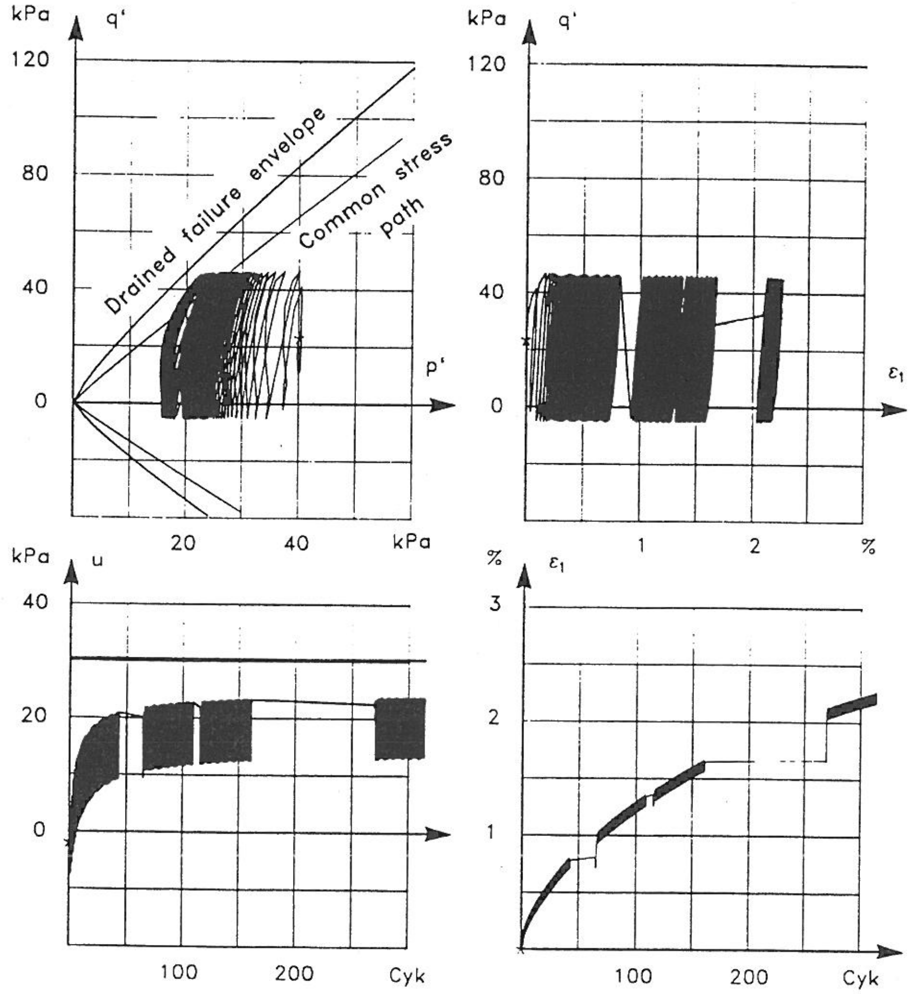

Figure 10. Result of a cyclic triaxial test classified as Pore Pressure Buildup. The cyclic loading is seen to generate positive pore pressure which leads the mean normal stress towards the cyclic stable state. The cyclic stable state is reached after 160 cycles. The test is performed on Lund No 0 with $I_D = 0.78$ and equal height and diameter.

the relative density of the material. For the tests in Figure 11, which are performed on Lund No 0 with $I_D = 0.78$, the cyclic stable line is found to be described as $q'_{SCL} = 1/2q'_f$.

THE CYCLIC FATIGUE THEORY FOR SAND

The drained stress state p'_s, q'_s – present before the cyclic loading is added to the test – controls the variation of the effective mean stress. If the anisotropic mean stress level is situated below the stable line CSL, the increase in pore pressure during the loading part of one cycle is greater than the reduction during the unloading part. The cyclic loading will then

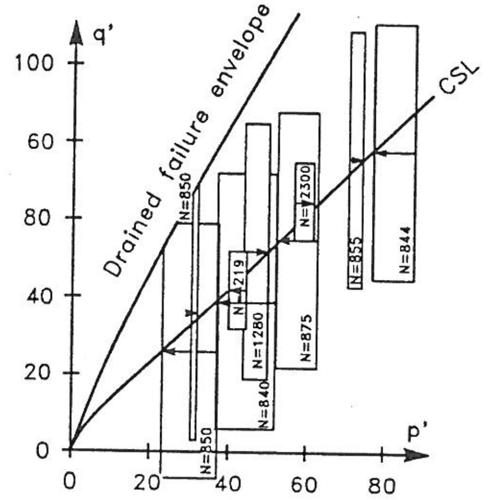

Figure 11. The effective stress path of 9 cyclic tests is shown. The test is performed on Lund No 0 with $I_D = 0.78$ and on specimens with equal height and diameter. N is the number of cycles added to the test and the arrow describes the changes in the effective mean stress.

lead to pore pressure buildup and the effective normal stress will decrease, see Figure 12.a and b.

Failure can take place, if the minimum stress level q'_{min} after a number of cycles, exceeds the characteristic line in extension CL^-. The pore pressure generation δu will then go from $\delta u > 0$ to $\delta u < 0$ twice during each cycle and the equilibrium of the stable state cannot be created. After the minimum stress level q'_{min} has exceeded the characteristic line in extension CL^-, the drained failure envelope will be reached during the subsequent cycle. Cyclic Liquefaction as defined by Casagrande (1971) will be observed if the maximum stress level q'_{max} during the subsequent cycle reaches the drained failure envelope in compression. Necking, which is a similar phenomenon to Cyclic Liquefaction and defined by Casagrande (1971), will be observed if q'_{min} reaches the drained failure envelope in extension. This failure mechanism is outlined in Figure 12.a. If the cyclic stable state is developed during the cyclic loading no failure will occur and the phenomenon Pore Pressure Buildup will be observed, see Figure 12.b. According to the figure no further changes in the mean normal stress will be observed while adding further cycles, see Figure 10.

If the anisotropic mean stress level is situated above the stable line CSL, the cyclic loading will generate negative pore pressure. Similar to the phenomenon Pore Pressure Buildup the cyclic stable state can be developed during the negative pore pressure generation and the phenomenon Stabilization will be observed, see Figure 12.c. If the amplitude of the cyclic load is so large that the stress variation during the first half cycle follows the common stress path, the mean stress level changes from being situated above the cyclic stable line to be situated below as outlined in Figure 12.d. After the first half cycle, where considerably negative pore pressure is generated, the cyclic loading will generate positive pore pressure due to the fact that the mean normal stress state is situated below the stable line. The phenomenon which is described as Instant Stabilization will be observed. Similar to Pore Pressure Buildup failure can take place, if the minimum stress level q'_{min} after a number of cycles exceeds the characteristic line in extension CL^-, otherwise the Stable State will develop.

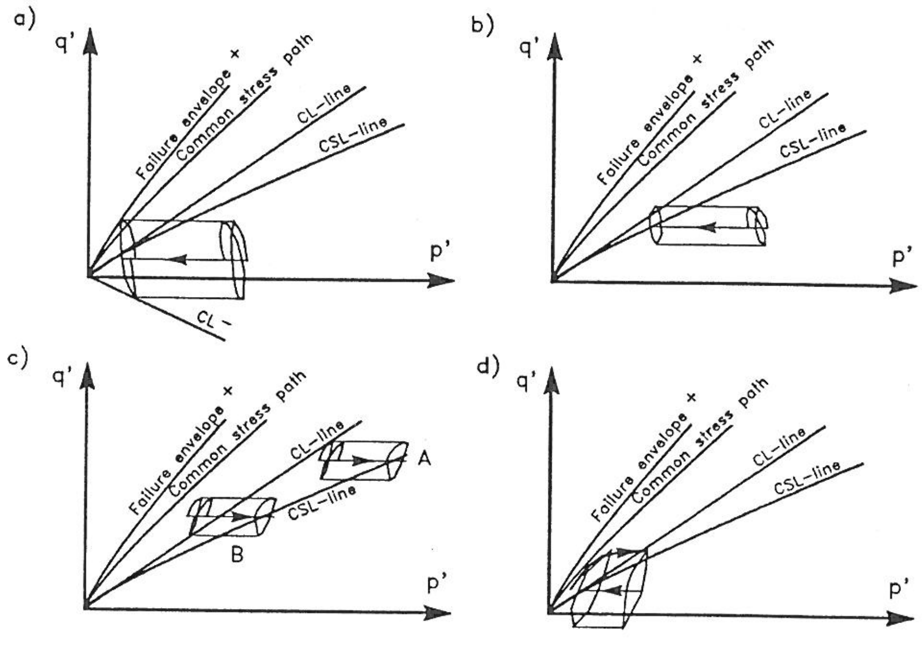

Figure 12. In the figure the phenomena a) Cyclic Liquefaction, b) Pore Pressure Buildup, c) Stabilization and d) Instant Stabilization are outlined. These phenomena represent the response due to cyclic loading, when the tests are performed on specimens with equal height and diameter.

CONCLUSION

By performing cyclic triaxial tests on specimens, which ensure homogeneous stress and strain distribution throughout the test, the existence of a cyclic stable state is found. The cyclic stable state is shown to have considerable influence on the development of the effective stress variations during cyclic loading. The cyclic stable state represents an ideal stress state, where the positive and negative pore pressures generated during a loading cycle neutralize each other, and the mean normal stress is shown not to change, when the stable state is reached, and further loading will not lead to any stress hardening or softening of the soil. The cyclic stable state is an intrinsic parameter, which is found to be independent of the amplitude of the cyclic load.

The cyclic stable state is shown to divide the stress space into two subspaces, where the cyclic loading will generate positive or negative pore pressures, respectively. Especially the new phenomena Stabilization and Instant Stabilization observed in connection with development of negative pore pressure are of great interest.

The cyclic stable state and the new phenomena Stabilization and Instant Stabilization have not been discovered earlier due to the traditions of performing triaxial tests on specimens with double height. Specimens with double height develop nonuniformities in strains, which result in inner draining and the correct generation in the pore pressure cannot be produced. The consequence of the errors described in this paper is that studies of basic phenomena in soil, drained or undrained, must be performed on specimens with a height equal to the diameter and with smooth pressure heads. This is the only way to ensure that theories do not reflect test errors but the real properties of the soil. In the study of pore pressure variation due to cyclic loading the phenomena leading to stress hardening are shown not to be observed, when the test is performed on a double height specimen.

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