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Experimental analysis of instabilities in very loose sands

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Abstract:

In this paper, the influence of strain and stress loading paths on the behavior of very loose sand is analyzed with particular attention to potential instabilities. A particular stress path, i.e. a quasi constant shear undrained path, is presented and comparisons are made with data from literature. Experimental results show that diffuse mode failure occurs before the Mohr-Coulomb failure surface is reached. It is shown for these undrained tests that excess pore pressure results from collapse and it is not a trigger parameter. Moreover, stress ratios at collapse and corresponding mobilized angle of friction are very close for classical consolidated undrained tests, constant shear drained tests and quasi constant shear undrained tests. The onset of collapse is thus independent of the loading path under drained and undrained conditions but depend on the direction of the stress increment.

I. INTRODUCTION

Several studies have shown that collapse can occur before reaching the plastic limit criterion, such the Mohr-Coulomb criterion ([1], [2], [6], [9], [15], [16] and [17]). This phenomenon is related to the bifurcation of the response of the material and to the loss of stability. At the bifurcation point, the sample should behave of in several ways. If considering the response curve, the sample can follow the fundamental branch and thus remain stable. Depending on the wavelength of the eigenmode comparatively to the characteristic length of the sample or the structure, two principal cases should occur. If the eigenvector is (very) small, localization of the strains occurs and shear band will appear. If the wavelength is close to or greater than the characteristic length, a diffuse mode of failure will then take place. The first type of behaviour have been thoroughly studied and simulated by using of the vanishing determinant of the acoustic tensor. The second type of response of the material after bifurcation will experimentally explored in this paper. Diffuse mode of failure manifests itself by a collapse of the sample strictly inside the Mohr-Coulomb plastic limit criterion and a sudden increase of strains. In this case, no more localization of strains appears and conventional failure analysis cannot explain them. Recent study ([1]) has shown that results obtained for clayey sands are very close to those obtained for very loose sands. Conclusions made for the very loose tested sand can thus be extrapolated to clayey sands containing a percentage of fine content up to 10 or 15 percent by weight. This work proposes a new test which follows a particular path: quasi constant shear stress undrained test (C.S.U.) in order to check if excess pore pressure is the consequence of collapse. In other words, is excess pore pressure a trigger parameter under undrained conditions? It is analytically established that collapse occurs inside a domain which is include in the Mohr – Coulomb limit criterion (a necessary condition) and that collapse depend on the direction of the increment of the loading [4]. A definition of an unstable domain is also proposed for classical undrained compression triaxial tests (ICU), for constant shear drained tests (CSD) and for quasi constant shears undrained tests (CSU). All tests have the same stress increment direction at collapse and are isotropically consolidated.

II. EXPERIMENTAL SETUP

This section presents the experimental apparatus developed to impose both axial loading (stress and strain controlled) and constant shear stress on the soil specimen under drained and undrained conditions. Unlike in previous experimental studies on dry sand [16] and on drained saturated sands ([6] and [9]) a particular loading path (detailed in section 4) is applied under undrained conditions. First, the components of the entire experimental setup will be presented, then the testing procedure and the characteristics of the sand used will described.

A. Testing apparatus

Axial displacement is imposed by a Whykeham Farrance ® Tritech (100 KN) displacement-controlled apparatus. The displacement rate is chosen by selecting a velocity in the range of 0.00001 to 5.99999 mm/mn. However, the loading can be completely applied via a piston type cylinder which can be assembled on the apparatus. With this arrangement mixed loading (i. e. displacement-controlled then load controlled) can be applied. Two digital pressure / volume controllers (DPVC) are used to perform isotropic consolidated drained tests. Back pressure is measured with a pore pressure transducer. During testing, the measured value is compared to the applied pressure. Axial displacement is measured by an external LVDT and changes in volume are measured by the piston displacement into and out of the DPVC. The forces are measured by an internal submersible load cell with the capacity of 1 or 5 KN depending of the initial density and the effective mean pressure. All tests variables are stored in a microcomputer.

B. *Sample preparation*

- Specimen preparation: The moist tamping (MT) method is used to prepare the specimen. The sand has a moisture content of about 3-4% in order to reach near maximal void ratio before and after isotropic consolidation of about the maximum void ratio $e_{max} \approx 1$ (TABLE 1). In order to obtained homogeneous samples, sand was carefully placed in five layers in the mould [1]. The membrane is sealed at the base of the cell and at a top cap. Saturated porous stones are placed in enlarged end plates to minimize friction and to allow homogeneous deformations at large strains. A small vacuum (10KPa) is applied to permit the removal of the split mould. The cell is then put into the testing apparatus and filled with water. A small confining pressure (20KPa) is then applied and reached a value of 30KPa when the vacuum is nil.

- Saturation process: A de-aerated water percolation is carried out to ensure sample saturation. Both cell pressure and back pressure are increased simultaneously in steps of 30KPa. During this process the B-Skempton coefficient is checked and samples are assumed to be saturated if $B \geq 96\%$.

- Void ratios: By drying the sample, the final void ratio is calculated. This void ratio is the same as the void ratio after consolidation e_c (as the test is undrained). The void ratio before consolidation (e_o) is obtained from the measured volume variation during isotropic consolidation and by assuming that grains and water are incompressible. Comparison of the initial void ratio e_i (accounting for the membrane thickness) and e_o shows that e_i is slightly higher than e_o due to strain that occur during the saturation phase and also during consolidation (see table 1).

TABLE 1 : CHARACTERISTICS OF THE UNDRAINED TESTS ON HOSTUN SAND S28

Test #	p'_0 (KPa)	e_i	e_o	e_c	q/p'_0
6	100	1.159	1.086	0.974	-
42	300	1.140	1.060	1.025	-
7	750	1.196	1.083	1.000	-
31	100	1.180	1.136	1.085	0.285
26	300	1.147	1.070	0.996	0.218
41	300	1.132	1.101	1.026	0.305
27	750	1.154	1.136	1.005	0.210

C. *Tested material*

The sand used for this study is Hostun S28 sand. The quartzic grains are sub-angular and the grain size distribution is uniform (Figure 1). The principal characteristics are summarized in TABLE 2.

TABLE 2 : BASIC PROPERTIES OF HOSTUN S28 SAND

Type	Mean Size (mm)	Cu	Specific gravity	Max void ratio	Min void ratio	Frictional angle (°)
Quartzic	0.3 - 0.35	2	2.65	1	0.656	32

D. *Testing program*

For a given void ratio, three effective mean pressures are used: p'_o = 100, 300 and 750KPa. The first series concern classical undrained triaxial tests (section III), the second series are constant shear drained tests (section IV) and the third series are quasi constant shear undrained tests (section V).

III. CLASSICAL LAOD AND DISPLACEMENT CONTROLLED UNDRAINED TESTS

Results obtained for load-controlled tests are compared with those obtained by Matiotti [13] and by Doanh et al. [7] for displacement-controlled tests. Results obtained for displacement-controlled tests are also compared to those obtained on same sand with a very close density by Desrues et al. [5]. The experimental device used in their study is a displacement-controlled apparatus which was modified to permit application of a constant force corresponding to peak shear stress. The constant force results from a spring attached to the displacement controlled device and the force is therefore proportional to the spring stiffness. As shown in Figure 2, it is noted at the peak of stress that $\eta = q/p' = 0.6$ which corresponds to a mobilized angle of friction of approximately 16°.

Good agreement is shown in this figure and these results validate both the experimental device and also the sample preparation (TABLE 1).

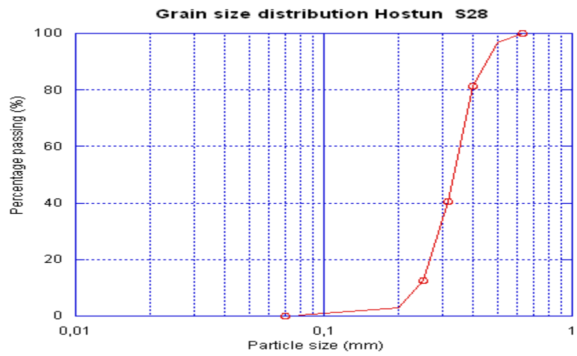

Figure 1 Hostun fine sand grain size distribution.

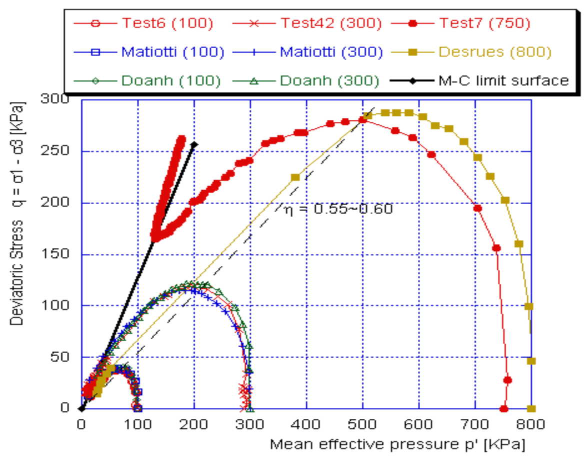

Figure 2 : Comparison of our results with those of Matiotti and al. ([13]), Doanh and al. ([7]) and of Desrues and al. [5] for p'_o = 100, 300, 750 KPa.

IV. CONSTANT SHEAR DRAINED TEST

This test was originally developed by Sasitharan et al. [15] on saturated loose Ottawa sand sheared under drained conditions with dead load. This special stress path was performed to simulate the loading of soil within a slope or an embankment subjected to a low increase in pore pressure. Di Prisco and Imposimato [6] and Gajo [9] have performed similar tests on Hostun's sand for approximately the same void ratio as in our experiments. The constant shear stress is also applied using dead load. Our main results are presented in Figure 3 either by increasing the pore pressure (tests # 58 and 61) or by decreasing total stresses (tests # 51 and 60).

V. QUASI CONSTANT SHEAR UNDRAINED TEST

An original test is presented in this section. The practical applications of this test correspond to the construction of a structure under undrained conditions on loose sand when boundary conditions prevent the drainage (retaining walls without drainage for example). It should be noticed that these tests also correspond to those obtained for denser sands containing 10 or 15 % (by mass) of fines (clayey sand for example). Practical application examples for such studies include the analysis of landslides and deep foundations.

A. *Quasi Constant Shear Undrained test*

In these tests, isotropically consolidated samples (point A in Figure 4) are submitted to either displacement- or stress-controlled loading (Point B). It important to emphasize that it is difficult to maintain the shear stress constant during these tests. Indeed, a similar reduction in shear stress is noted in tests conducted by Lade [11] on closure of the drainage valve which corresponds to the undrained phase of the classical drained then undrained test and by Chu et al. [2] for constant shear drained (CSD) tests**.** Undrained conditions are maintained and the sample is then sheared along a quasi constant deviator stress path by decreasing the total axial stress (σ_1) and the total radial stresses (σ_3) from point C to point D for which an increase of the axial deformation is noticed (Figure 7). The deviatoric stress is then maintained quasi constant up to the point E. Nevertheless, at this point, it is no longer possible to maintain the constant shear stress as on CE. Indeed, it decreases continuously; the test becomes non-controllable as defined by Nova [14].

B. *Experimental results*

Figure 4 shows results of a constant deviatoric undrained test on an isotropic consolidated sample ($p'_0 = (\sigma'_1 + 2\ \sigma'_3)/3$ = 750 KPa) with a void ratio before isotropic consolidation of e_0 = 1.136. The deviatoric stress - effective mean pressure response is shown in this figure. After undrained loading, the stress deviator ($q = \sigma_1 - \sigma_3$) was maintained approximately constant up to 157.5 KPa which corresponds to an anisotropic stress state q/p' = 0.210. To study the influence of the effective mean normal pressure, results obtained for p'_0 = 100, 300 and 750 KPa are presented in Figure 5. Figure 6 presents results obtained for two tests carried out with the same effective mean normal pressure of 300 KPa and for two values of constant deviatoric stress. Figure 7 shows the deviatoric stress and the excess pore pressure, the axial deformation as well as the second order work versus time which is defined in section VI.C.

VI. RESULTS AND DISCUSSION

A. *Onset of collapse*

Experimental collapse points obtained from C.U., C.S.D. and C.S.U. tests are plotted in Figure 8. Herein, collapse means that the sample is no more controllable [14]. The loading program, with the correct control parameters, can thus not be maintained. One can see that the stress ratios and the corresponding mobilized angles of friction at collapse are very close (Tab. 3). It is relevant to note that collapse occurs strictly inside a domain which is included in the Mohr-Coulomb limit surface. Taking into account experimental uncertainties, the unstable line seems to be unique for all the considered loading paths. This result is of course not generally valid and could be induced here by the fact that, at collapse, stress increments are parallel as depicted in Figure 9.

B. *Diffuse mode of failure*

The deformation of all the samples seems to be homogeneous even for large strains (after collapse): no strain localization neither by a single shear band nor by shear bands pattern were noted. One can observe in Figure 7 that pore pressure monotonically decreases after the application of constant shear stress (between the points C and D). Moreover, the analysis of pore pressure variation at the time of collapse highlights two points:

- Collapse occurs while the pore pressure decreases meaning that the increase in pore pressure is related to collapse (see Figure 7). Therefore, pore pressure is not a trigger parameter for collapse.

- The increase in pore pressure during collapse is more than likely the result of a diffuse mode of deformation rather than a localized one since, upon localized deformation, no notable change in pore pressure should be observed.

C. *Characteristic of the constant shear undrained test*

The sign of the second order work, based on the local condition of stability defined by Hill [10], can be used to study the stability of the tested material. In axisymmetric conditions second order work is defined ([3], [4], and [17]):

$$d^2 W = d\sigma_{ij}\, d\varepsilon_{ij} = d\sigma_1 d\varepsilon_1 + 2\, d\sigma_3 d\varepsilon_3 \qquad (1)$$

which can be re-arranged to give:

$$d^2 W = dq\, d\varepsilon_v + d\sigma_3 d\varepsilon_v \qquad (2)$$

with $dq = d\sigma_1 - d\sigma_3$ and $d\varepsilon_v = d\varepsilon_1 + 2\, d\varepsilon_3$.

Along the C.S.U (isochoric condition and q = constant) $d\varepsilon_v$=0 and dq = 0 which implies that $d^2W = 0$.

It is not possible to apply this path as showing in Table 4. As no volume variation is imposed, it is not possible to impose also no variation of shear stress. For this reason, a very small variation of the shear stress

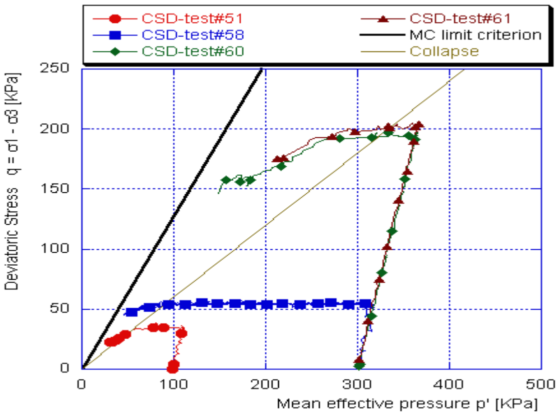

Figure 3. Constant shear drained tests on Hostun's sand

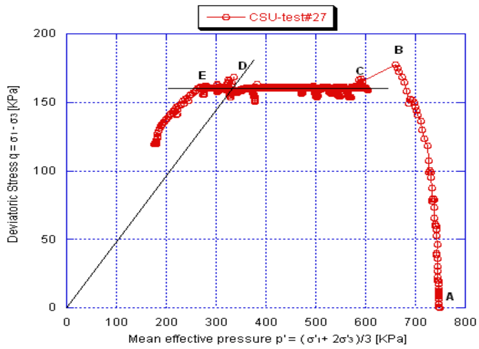

Figure 4 Quasi constant shear undrained test

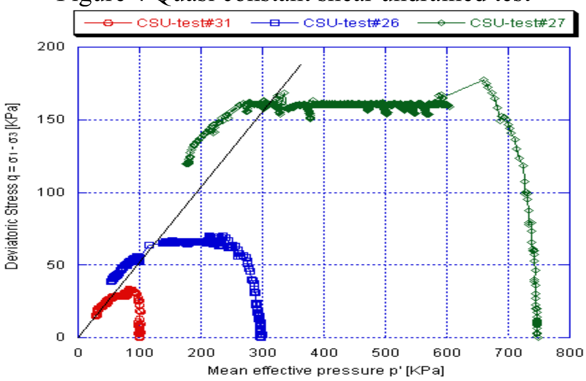

Figure 5 Quasi constant shear undrained test p'_o = 100, 300, 750 KPa

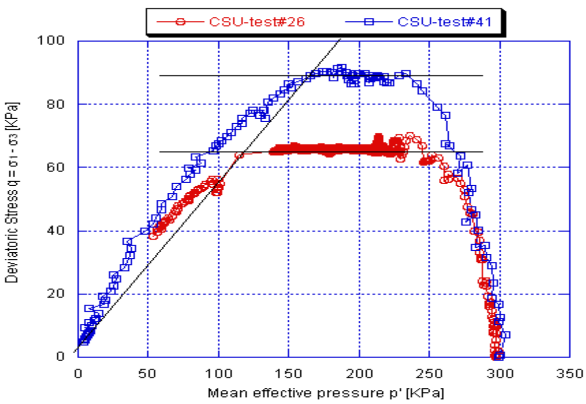

Figure 6 Quasi constant shear undrained test p'_o = 300 KPa, q_{const} = 65.26 and 91.55 KPa.

TABLE 3 : MOBILIZED ANGLE OF FRICTION AND STRESS RATIO AT COLLAPSE DURING C.U. ,C.S.D. AND C. S. U. TRIAXIAL TESTS ON HOSTUN SAND

Type of test	Test #	p'_0 (KPa)	e_o	ϕ_{mob} (°)	$(q/p')_{coll}$
I.C.U.	6	100	1.086	15.26	0.58
	42	300	1.060	15.67	0.59
	7	750	1.083	14.89	0.56
C.S.D.	51	100	1.192	15.06	0.57
	58	300	1.068	14.49	0.55
	60	300	1.161	16.53	0.62
C.S.U.	31	100	1.136	13.91	0.52
	26	300	1.070	14.72	0.56
	41	300	1.101	13.64	0.51
	27	750	1.136	15.36	0.58

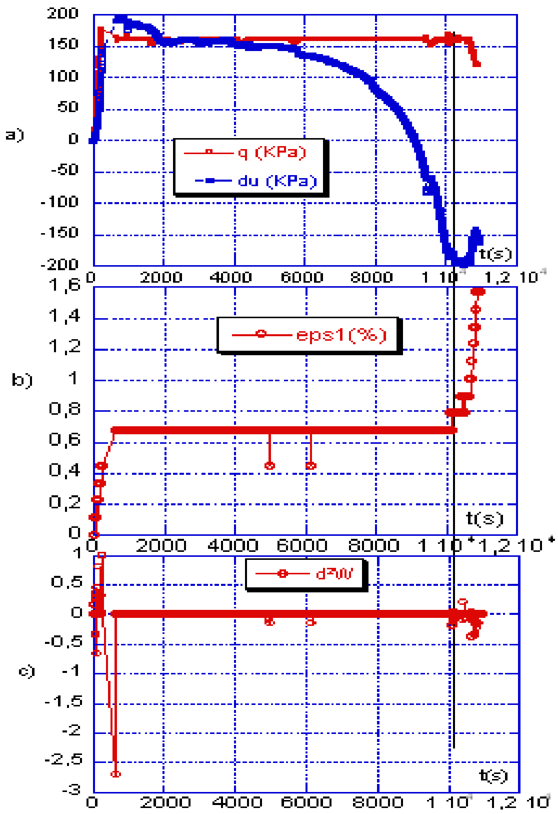

Figure 7 Constant shear undrained test (p'_o = 750 KPa): deviatoric and excess pore pressure, axial displacement and second order work versus time (test 27).

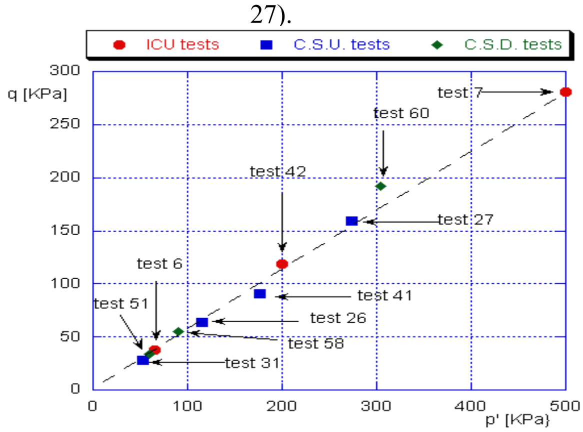

Figure 8 Comparison between stress ratios obtained from CU, CSD and CSU tests

$$\begin{cases} d\varepsilon_v = 0 \\ dq = \eta \end{cases} \tag{3}$$

Table 4: Available equations for load-controlled tests under undrained conditions.

Conditions	Stresses	Strains
Axisymmetry	$d\sigma_2 = d\sigma_3$	$d\varepsilon_2 = d\varepsilon_3$
Isochoric	/	$d\varepsilon_v = 0$
p′ and q decrease	$d\sigma_1 < 0$	/
	$d\sigma_3 < 0$	/
Constitutive relation	$\sigma = C^{ep}\,\varepsilon$	
q constant	impossible	/

$dq = \eta$ is needed. Figure 7 illustrates that, the second order work is negative when collapse occurs.

$$\begin{bmatrix} dq \\ d\varepsilon_v \end{bmatrix} = \begin{bmatrix} A & C \\ D & B \end{bmatrix} \begin{bmatrix} d\varepsilon_1 \\ d\sigma_3 \end{bmatrix} \tag{4}$$

$$\begin{cases} Ad\,\varepsilon_1 + Cd\,\sigma_3 = \eta \\ Dd\,\varepsilon_1 + Bd\,\sigma_3 = 0 \end{cases} \tag{5}$$

$$d\sigma_3 = \frac{D}{CD - AB}\eta \quad \left(d\sigma_3 < 0 \text{ if } \eta > 0\right) \tag{6}$$

$$d\sigma_1 = \left(1 + \frac{D}{CD - AB}\right)\eta \quad \left(d\sigma_1 < 0 \text{ if } \eta > 0\right) \tag{7}$$

$$d\varepsilon_1 = \frac{B}{AB - CD}\eta \quad \left(d\varepsilon_1 > 0 \text{ if } \eta > 0\right) \tag{8}$$

$$d\varepsilon_3 = \frac{B}{2(CD - AB)}\eta \quad \left(d\varepsilon_3 < 0 \text{ if } \eta > 0\right) \tag{9}$$

The parameter η takes the successively the value:

$$\eta = \pm k \tag{10}$$

where k is imposed and depends on the test (0.5 or 1 KPa). The second order work is then given by:

$$d^2W = \eta d\varepsilon_1 = \frac{B}{AB - CD}\eta^2 \tag{11}$$

At the bifurcation point, which corresponds to the peak of the shear stress in the classical undrained test, we have:

$$\begin{cases} \eta = 0 \\ AB - CD = 0 \\ d^2W = 0 \end{cases} \tag{12}$$

Amplitude of the second order work goes to infinity in the vicinity of the bifurcation point. However, it was experimentally noted that the second order work value is finite as depicted in Figure 7.

Sometimes, sample collapse can occur before reaching the unstable domain. This behavior can be explained by an imperfection of the experimental device or by a disturbance in the loading. This observation is important for the study of slope instability because such disturbances can occur that lead to failure on slopes smaller than 10° and can be shown as trigger factors.

VII. CONCLUSIONS

In this work, classical consolidated undrained (C.U.) tests, constant shear stress drained tests (C.S.D.) and quasi constant shear stress undrained tests (C.S.U.) were applied to very loose sand. Results obtained are presented and discussed. Analysis of pore pressure evolution highlights a diffuse mode of deformation during collapse. Moreover, the break in the pore pressure evolution slope makes it possible, during the test, to locate the instant when collapse occurs. The increase in the pore pressure is thus the consequence of collapse and not a triggering factor.

Collapse of samples occurs for closed values of stress ratios and mobilized angles of friction. It can thus be stated that the onset of collapse is not dependant of the loading conditions under drained and undrained cases but depend on the direction of the stress increment.

Second order work calculated from the experimental results becomes negative at the time of collapse and can be a valuable tool for identifying collapse.

One should notice that this collapse always occurs strictly inside the Mohr-Coulomb limit surface of plasticity and that explain certain ruptures which can not be explained by the classical theory of plasticity.

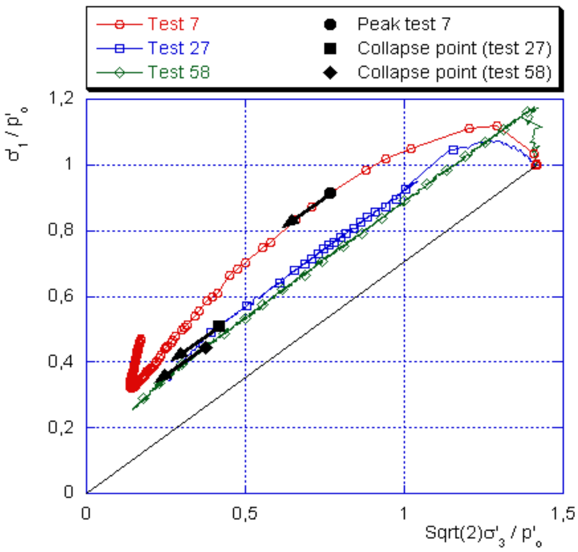

Figure 9. Orientation of stress increments for ICU, CSD and CSU tests.

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