# Factors affecting sand behavior in constant deviatoric stress loading

Reza Imam, Arash Azizi and Rashid Zandian
*Amirkabir University of Technology, Tehran, Iran*

ABSTRACT
A rise in pore water pressure within a soil mass leads to a decrease in the mean effective normal stress, while the vertical load due to soil weight remains unchanged. Such loading may be applied in the triaxial apparatus using a test referred to as the Constant Deviatoric Stress (CDS) test. Under such loading, loose dry sand contract substantially, and saturated sand experience high pore pressure and loss of strength if not allowed to drain. This study examines factors that affect the volume contractions of loose sands in CDS loading. Results showed that void ratio, confining pressure, applied deviatoric stress, anisotropic consolidation, method of sample preparation, sand type and angularity affect these volume contractions.

RÉSUMÉ
Une élévation de pression d'eau interstitielle dans une masse de sol mène à une diminution de l'effort normal efficace moyen, alors que la charge verticale due au poids de sol demeure sans changement. Un tel chargement peut être appliqué utilisant un essai de la fatigue Deviatoric constant (CDS). Cette étude examine le comportement du sable lâche sous le chargement de CDS.

## 1 INTRODUCTION

### 1.1 The constant deviatoric stress loading

Many cases of flow-type slope failures have been attributed to rises in pore water pressure (see e.g. Morgenstern 1994; Eckersley 1990; Lade 1993, and Anderson and Sitar 1995). These failures have been observed to occur following pore water pressure increases due to the infiltration of rainwater, snowmelt, cyclic loading, etc. and are often of the types referred to as flow slides or debris flows.

A rise in pore water pressure within the soil mass leads to a reduction in the soil mean normal stress or confining stress. This may occur while the vertical loads due to soil weight remain unchanged, or increase as a result of soil wetting. Such loading condition may be represented in the triaxial apparatus by a test in which a constant load is applied to the top of the soil sample, while the confining stresses are gradually reduced. Since in such tests the deviatoric stress applied to the soil sample remains approximately constant, the test may be referred to as a "Constant Deviatoric Stress" or a CDS test.

Previous studies have shown that cohesive soils, and medium to dense granular soils generally dilate when subjected to CDS loading, while very loose and loose sands initially experience small or no volume change, and then start to contract substantially as failure is approached (see e.g. Sasitharan et al., 1994; Skopek 1994; and Anderson and Riemer 1995). If the sand is saturated and drainage can not occur fast enough, such contractions can lead to the generation of substantial pore water pressures and loss of strength. The same can happen in the field under poor drainage conditions, and the loss of soil strength may lead to sudden, catastrophic failure of slopes.

Conventional limit equilibrium methods of slope stability analysis can not predict such failures since the volume contractions and loss of strength usually occur well before the mobilization of failure shear strength. It is important, therefore, that the conditions leading to the onset of volume contractions are determined, and the factors that affect the amount of contractions are examined in order that the potential for such failures can be avaluated.

In this study a number of CDS tests on loose and very loose samples of sand are carried out in order to examine the factors that affect the aforementioned volume contractions. The relationship between the contractive behavior of dry samples and the collapse of saturated samples in CDS loading is also examined by conducting CDS tests on saturated samples of the same sand.

### 1.2 Behavior of loose sand in triaxial and CDS loading

Figure 1 illustrates typical test results for samples of very loose liquefiable sand subjected to drained and undrained triaxial compression and CDS loading.

Under drained triaxial compression loading (A-E), the sample shear strength increases monotonically until it reaches the critical state strength, represented by the critical state line (CSL), at E. When drainage is prevented, the sample shear strength initially increases to a peak at P, and then starts decreasing (strain softening) until it reaches the critical state strength at F, where it continues to experience shear strain under constant shear stress (q), mean effective normal stress (p´), and void ratio (e) as described by Casagrande (1936).

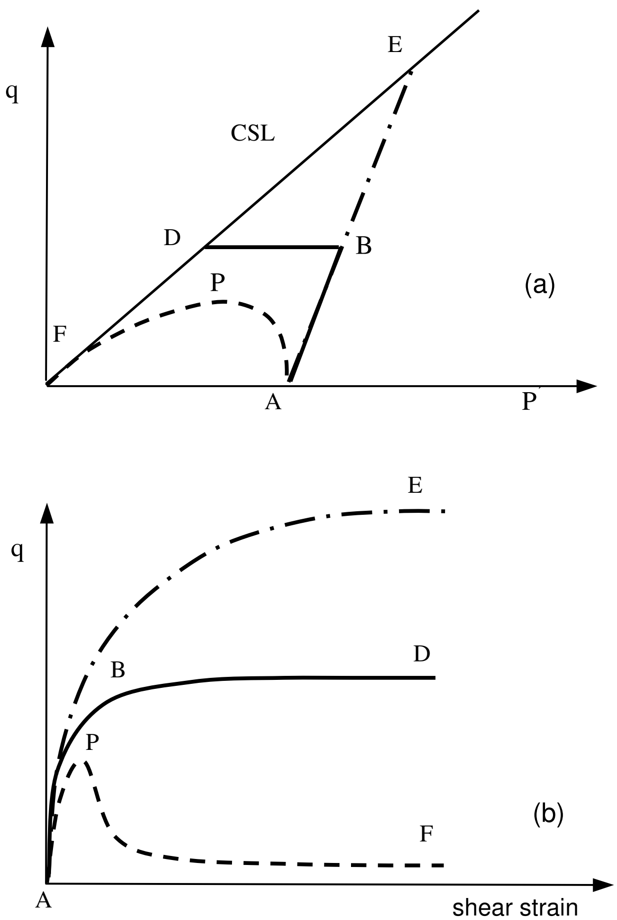


Figure 1. Typical results of tests on very loose sand
(a) Stress paths (b) stress-strain relationships (CSL: Critical State Line)

Stress path ABD represents CDS loading in which the sample is first subjected to drained triaxial loading (A-B) until the deviatoric stress reaches a certain value at B. The deviatoric stress is then kept constant while the mean effective normal stress is reduced. This reduction is continued until the soil state reaches the crtical state condition in D, where it continues experiencing shear strain at a constant deviatoric stress, mean effective normal stress, and void ratio. It is noted that in order to apply CDS loading, drainage of the sample should be allowed, since preventing drainage will casue the soil sample to follow a stress path with a decreasing shear (deviatoric) strength (such as P-F) and, therefore, a constant deviatoric stress cannot be maintained.

## 2 DESCRIPTION OF THE TESTS

### 2.1 The sands tested

Most tests were carried out on a local uniform, subangular to angular, quartzic sand called the Firoozkooh No. 161 sand. Two other sands, namely the Ottawa sand and another local sand named Chamkhaleh were also tested for comparison. Properties of the sands tested, as reported by Azizi (2009), are provided in Table 1.

All samples were prepared using the moist tamping procedure, in which 71 mm diameter, 160 mm high samples were prepared by pouring 8 layers of sand with 2.5 percent moisture. Each layer was lightly tamped using a metal tamper with a diameter equal to that of the mould. The initial layers were tamped once and more gently, and the last layers were tamped twice and using a somewhat higher energy such that the sample will have as uniform a density as possible throughout its height.

Table 1. Properties of the sands tested

| property | Ottawa | Firoozkooh No. 161 | Chamkhaleh |
|---|---|---|---|
| $G_s$ | 2.67 | 2.7 | 2.65 |
| $e_{max}$ | 0.82 | 0.8740 | 0.941 |
| $e_{min}$ | 0.5 | 0.5480 | 0.651 |
| $D_{50}$ | 0.34 | 0.26 | 0.17 |
| $C_c$ | 1.12 | 1.13 | 1.385 |
| $C_u$ | 1.66 | 1.9 | 1.09 |
| angularity | sub-rounded | subangular to angular | angular to very angular |

### 2.2 Loading procedure in the CDS tests

All tests were conducted using a triaxial stress path testing apparatus located in the soil mechanics laboratory of Amirkabir University of Technology.

Most tests were carried out on moist samples, but a few were also done on saturated samples. Most moist samples were first consolidated isotropically and then subjected to a dead axial load that remained constant throughout the test. The dead load was applied by placing specified weights on a round plate supported by a shaft screwed to the upper platen of the triaxial apparatus. A few samples were also consolidated anisotropically before applying the dead load.

In the tests on moist samples, following application of the dead load, the cell pressure was gradually reduced until the critical state condition was reached. In the tests on saturated samples, the soil was first subjected to a conventional strain-controlled undrained triaxial compression loading until the specified deviatoric stress was reached. Shearing was then stopped and the deviatoric load applied to the top of the sample was locked at the current value. A round metal plate was then screwed to the top of the metal shaft connected to the upper triaxial platen, and sufficient dead weight was placed on the round plate such that the same deviatoric stress that was applied by the loading frame prior to stopping the undrained test was applied to the top of the sample.

A back pressure equal to the current pore pressure was then applied and the drainage port was opened such that the pore pressure and the sample volume remained unchanged. A drained testing was then initiated by gradually increasing the back pressure while the cell pressure was kept constant. This resulted in the application of an almost constant deviatoric stress to the top of the sample, while the mean effective normal stress was being reduced under drained conditions.

## 3 TYPICAL TEST RESULTS

### 3.1 Undrained test results

Figure 2 shows typical results of an undrained test on a very loose sample of the Firoozkooh No. 161 sand consolidated to a void ratio of 0.92 and confining pressure of 248 kPa.

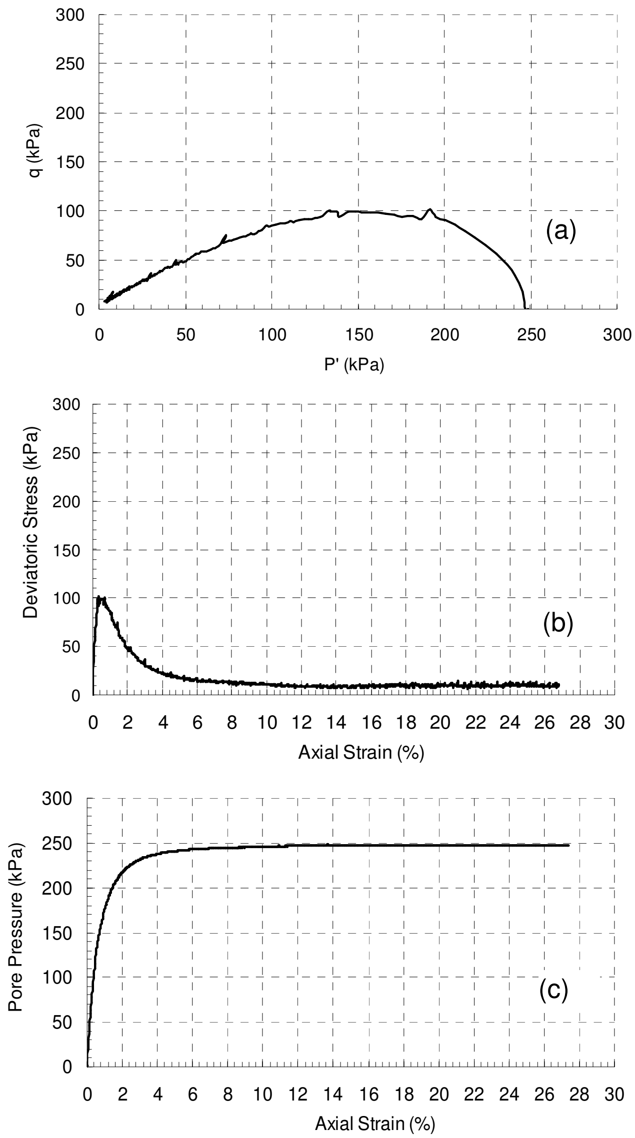


Figure 2. Results of undrained test on very loose Firoozkooh sand (a) stress path (b) stress-strain relationships (c) pore pressures generated

During loading, the deviatoric stress initially increased until it reached a maximum and then decreased sharply to a minimum at critical state, where it continued to experience shear strain without a change in deviatoric or mean effective normal stresses.

Figure 2(b) shows that before the maximum deviatoric stress was reached, the rate of development of shear strain was small, but it increased substantially afterwards. At critical state the sample was left with a small shear strength and almost “liquefied.” Such liquefied soil may behave similar to a liquid and experience large shear strains, or “flow” even under small loads.

From Figure 2(c) it may be noticed that during shearing of very loose sand, substantial pore pressure is generaed if drainage is prevented. It is expected that in a drained test, this very loose sample would contract substantially, and this volume contraction is translated into the observed large pore pressure generated in the undrained test.

### 3.2 CDS test on dry sand

Figure 3 shows typical results of a test on moist Firoozkooh No. 161 sand. The sample is consolidated isotropically to 250 kPa (point A), and subjected to a deviatoric stress of 110 kPa (point B). The mean effective normal stress was then gradually reduced by decreasing the cell pressure while the deviatoric stress remained constant.

It may be seen from the figure that during the initial stage of the CDS stress path, the sample experienced slight expansion, but as it reached a certain state prior to point C, it initially began to contract slightly. Upon further reduction of the mean effective normal stress, the rate of contraction increased substantially (at point C) until the CSL was reached at D. Sample height also changed similarly as it initially decreased at a smaller rate, and then the rate increased substantially until the critical state was reached (see Azizi 2009).

It is noted that the locations of the CSL both in the e-p plane and in the p-q plane were determined using results of undrained triaxial compression tests similar to that shown in Figure 2. The mobilized friction angle at critical state was found to be 33 degrees (see Azizi 2009).

Imam et al. (2002) showed that after application of the deviatoric stress (at B), the sample has a yield surface that passes through points B and C. While the CDS loading is applied from point B to C, the stress state remains inside this yield surface, and the sample experiences unloading associated with a volume expansion as observed in the test results. Imam et al. (2005) used similarly derived yield surfaces to model the behavior of sand under various loading conditions. As the soil state moves beyond point C, yielding restarts and the yield surface is increased in size. As a result, large volume contractions and change in height occurs as observed in Figure 3 (b).

It is of interest to note that the mobilized friction angle ($\varphi_m$) at point C where large volume contraction starts is 22 degrees, which is 11 degrees smaller than the mobilized friction angle at critical state. As shown later, in saturated sand, if this volume contraction is not allowed to occur, pore pressures may increase and loss of strength may initiate at a mobilized friction angle well below that of failure at the crictical state.

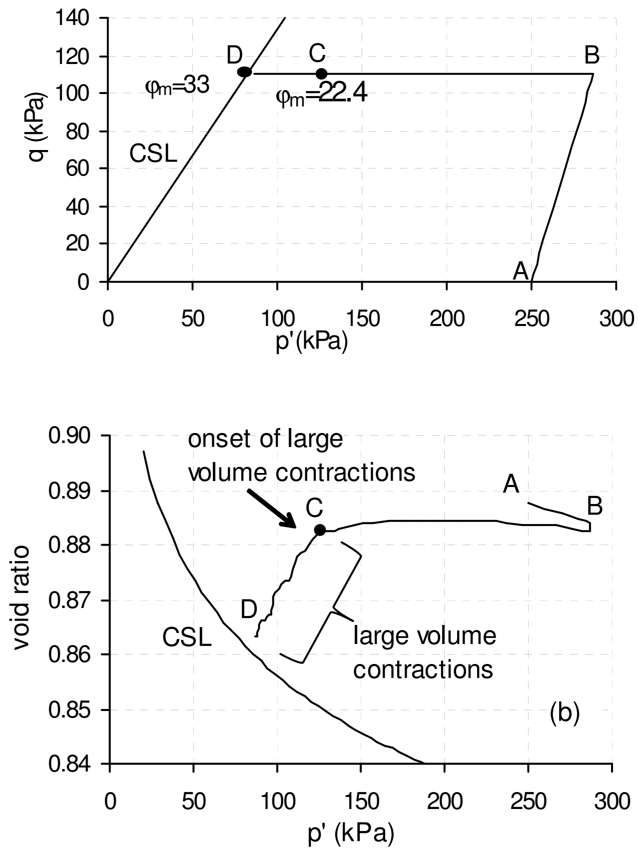


Figure 3. Typical behavior of dry sand in CDS loading: (a) stress path; (b) changes in void ratio

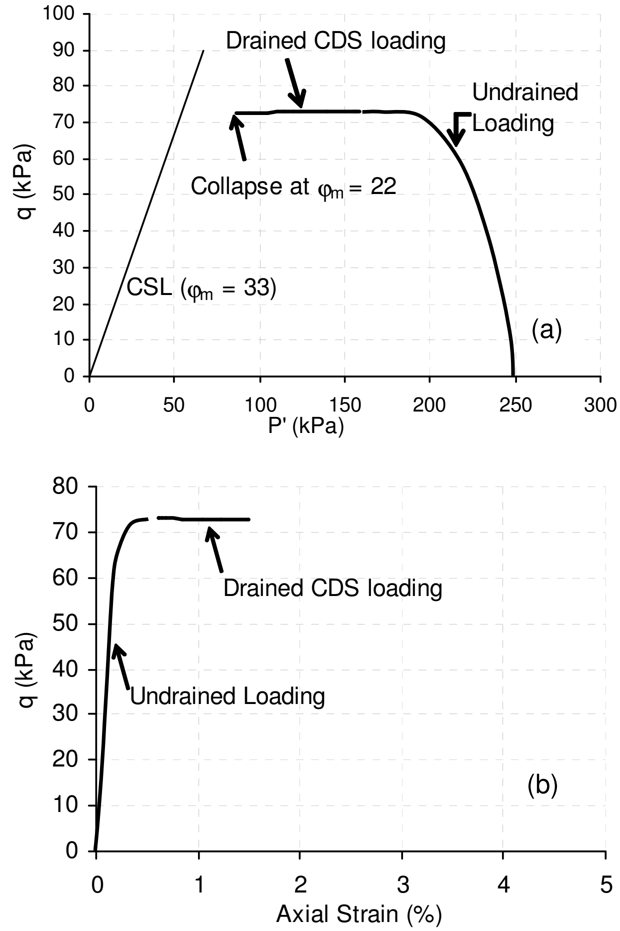


Figure 4. Behavior of saturated sand in CDS loading: (a) Stress path (b) Deviatoric stress v.s. axial strain.

### 3.3 CDS test on saturated sand

Figure 4 shows results of a CDS test on a saturated sample of the Firoozkooh No. 161 sand. The sample was consolidated isotropically to a confining pressure of 250 kPa and void ratio of 0.901. It was then subjected to undrained loading until the deviatoric stress reached 73 kPa at a mean effective normal stress of 157 kPa, and an axial strain of 0.38 percent.

A dead load equivalent to a deviatoric stress of 73 kPa was then applied to the top of the sample, and the back pressure was set equal to the current pore pressure. The mean effective normal stress was then gradually decreased starting from 157 kPa, and when it reached 86 kPa, the sample suddenly collapsed and was no longer able to sustain the applied constant deviatoric stress. The collapse occured at such high speed that no data could be recorded during this stage. At collapse, the axial strain was 1.5 percent, the void ratio 0.909 and the mobilized friction angle 22 degrees, which is substantially smaller than the mobilized friction angle at critical state.

It is of interest to relate the behavior of the dry sample with that of the saturated sample under CDS loading. As shown in Figure 3, the moist sample experiences contraction as it reaches a certain state prior to failure, and the same is expected to occur with the saturated sample, which is subjected to drained CDS loading. This can occur provided full drainage can take place such that it allows the volume contractions that would occur in a dry sample to take place. However, due to limitations in the drainage capacity of the triaxial sample and the triaxial drainage port, some of the volume changes can not occur sufficiently rapidly and, as a result, some pore pressure develops which results in the saturated sample to follow a stress path similar to that of an undrained sample, leading to a reduction in shear strength as shown in Figure 2(a). This results in the inability of the saturated sample to sustain the constant deviatoric load, and leads to its sudden collapse well before its reaching the critical state failure envelope.

Comparison can also be made between the mobilized friction angles at the onset of volume contraction of the dry sample, and at the collapse of the saturated sample. Measured friction angles were almost the same in both cases. Since these friction angles are expected to reflect the stress states at which the yield surface that was established at the start of the CDS loading (as at point B in Figure 3(a)) is reached again (point C in Figure 3(a)) (Imam et al. 2002), and this yield surface depends on both the current void ratio and the current stress state, the increased void ratio of the saturated sample (e=0.901) compared to that of the dry sample (e=0.882) is compensated for by the decreased CDS stress level applied to the saturated sample such that the mobilized friction angle at yielding remained almost the same for both samples. This may also be explained by considering the geometry of the yield surface and the way it depends on the void ratio and applied stresses (see Imam et al. 2002).

## 4 FACTORS AFFECTING SAND BEHAVIOR IN CDS LOADING

### 4.1 Void ratio

Figure 5 shows results of CDS tests on samples of Firoozkooh sand consolidated to 250 kPa and subjected to a constant deviatoric stress of 110 kPa. Void ratios at initial sample preparation ($e_p$) for all the tests are shown and void ratios after consolidation may be determined from the figure.

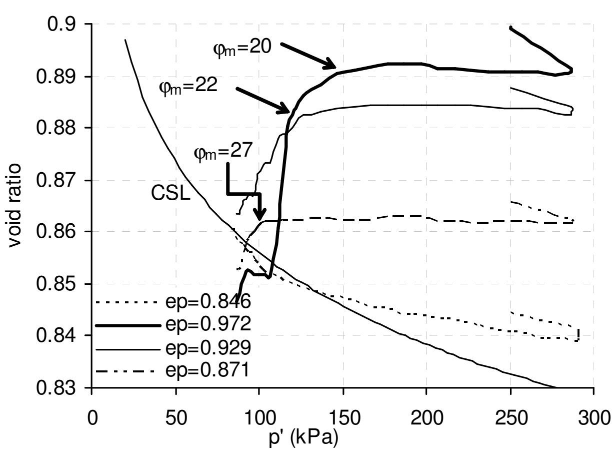


Figure 5. Effet of void ratio on the potential for volume contraction in CDS loading.

It may be noticed from the figure that upon application of the CDS loading, samples with smaller void ratios experience less volume contractions before they reach the CSL such that the sample with $e_p$ = 0.846 exhibits no volume contraction but dilates continuously throughout the CDS stress path. The onset of volume contraction also occurs at smaller mean effective stress and higher mobilized friction angle in samples with lower void ratios. This is consistent with past research indicating that static liquefaction occurs only in loose sands (see e.g. Castro, 1969). Current results indicate that as sand density increases, the potential for pore pressure generation and loss of shear strength during loads with stress paths similar to that of the CDS test decreases and it vanishes if density is increased beyond a certain value.

### 4.2 Confining pressure

Samples were prepared with the same void ratio at preparation $e_p$ = 0.972 and then consolidated to mean effective normal stresses of 150, 250, and 350 kPa. Due to differences in consolidation pressures, void ratios of the samples after consolidation were not the same. Following consolidation, all samples were subjected to the same CDS loading of 110 kPa.

Test results shown in Figure 6 indicate that the onset of volume contraction in all the samples occurs at about the same mean effective normal stress. Since the applied deviatoric stress for all the tests is also the same, mobilized friction angle at the onset of volume contraction is approximately the same for all samples.

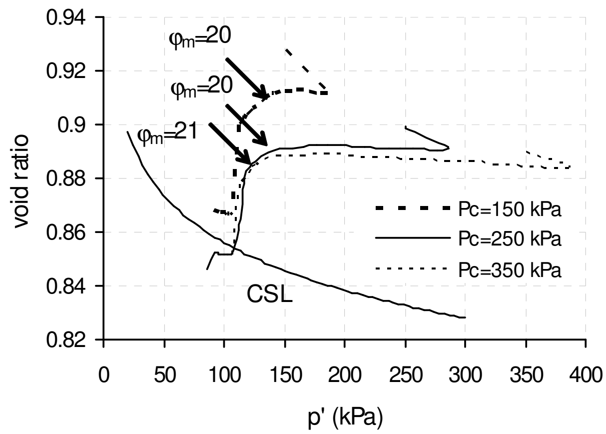


Figure 6. Effect of confining pressure on sand behavior in the CDS loading.

Figure 6 also shows that the amounts of volume contractions experienced by the samples tested are also similar. Although in the samples with higher confining pressures, the void ratios during the CDS portion of the loading path are smaller, but the higher confining pressure appears to compensate for the decrease in void ratio, resulting in about the same volume contraction and mobilized friction angle at onset of contraction. These results are consistent with the findings of Anderson and Riemer (1995), who indicated that the potential for collapse (volume contraction) is related to the state parameter of the sample. The state parameter is defined as the difference between the current void ratio and the void ratio at critical state at the current mean effective normal stress (ie. the vertical distance between the current state and the critical state line at the same mean effective normal stress in an e-p plot). It may be noticed from Figure 6 that the samples tested have similar state parameters at consolidation regardless of their confining pressure.

It may also be noticed from Figure 6 that the volume expansion during the initial stage of the CDS loading is higher in the tests with the higher confining pressures. This is also consistent with the notion that as the confining pressure increases, the size of the elastic region that develops after consolidation is larger, resulting in more unloading during the initial part of the CDS loading (Imam et al., 2002).

### 4.3 Anisotropic consolidation

Soils normally exist in an anisotropically consolidated state in the field. Therefore, study of the behavior of anisotropically consolidated soils is of particular interest when in-situ behavior of sands is considered.

Figure 7 shows results of CDS tests on isotropically consolidated and anisotropically consolidated samples of Firoozkooh No. 161 sand. The anisotropically consolidated sample was subjected to increments of major and minor principal stresses with a ratio of 2.5. The increments were applied in steps unil the minor principal stress reached 250 kPa. Increments of deviatoric stress were applied by gradually adding dead loads to the upper plate attached to the top triaxial platen described before. At the end of anisotropic consolidation, the deviatoric stress

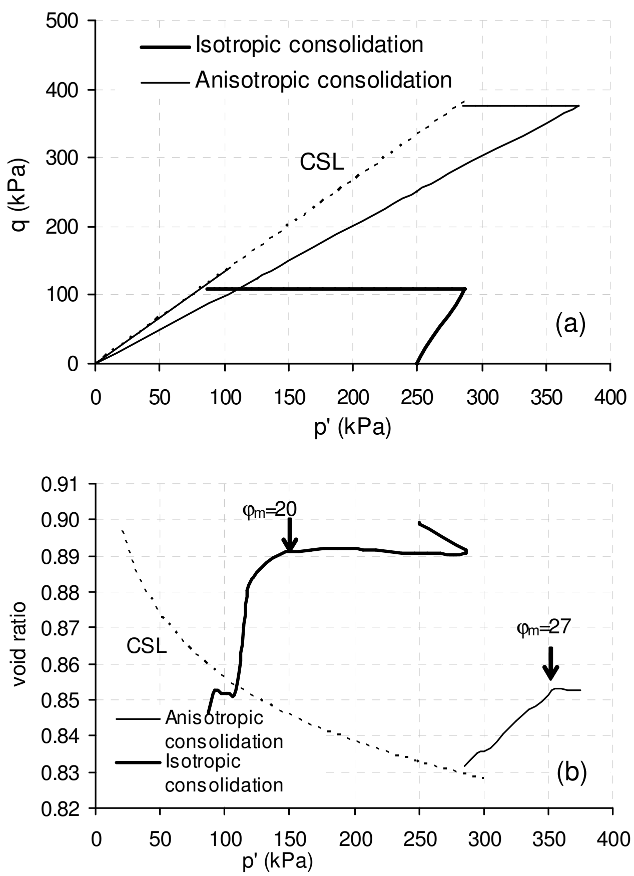


Figure 7. Effect of anisotropic consolidation: (a) stress path (b) variation of void ratio with mean normal stress.

was 375 kPa, and this stress was kept constant during the CDS portion of loading. The other sample was consolidated isotropically to 250 kPa before being subjected to a CDS loading of 110 kPa.

As shown in Figure 7, due to anisotropic consolidation, the stress state at the end of consolidation is closer to the CSL compared to that of the isotropically consolidated sample. As a result, after a small decrease in mean effective normal stress during the CDS loading, volume contractions are initiated. This behavior indicates that actual soils in the field are more prone to loss of strengh due to rise in pore pressures compared to the soil samples tested in the laboratory under isotropic consolidation.

On the other hand, for samples with the same void ratio at preparation, the higher shear stresses applied during consolidation of the anisotropically consolidated sample result in a smaller void ratio at the end of consoldation compared to the isotropically consolidated sample. As a results, during the CDS loading, the anisotropically consolidated sample will experience smaller volume contractions compared to the isotropically consolidated sample.

### 4.4 Method of sample preparation

Past experience has shown that the method of preparation of sand samples in the laboratory affects its behavior. Undrained triaxial compression tests carried out on samples prepared using the dry deposition and moist tamping methods have shown that moist tamping results in soil samples with a more contractive behavior compared to dry deposition (see Ishihara 1993)

Samples of Firoozkooh No. 161 sand with approximately the same void ratio of 0.857 were prepared using the moist tamping and dry deposition methods. The samples were first consolidated isotropically to 250 kPa and then subjected to a CDS loading with a deviatoric stress equal to 110 kPa.

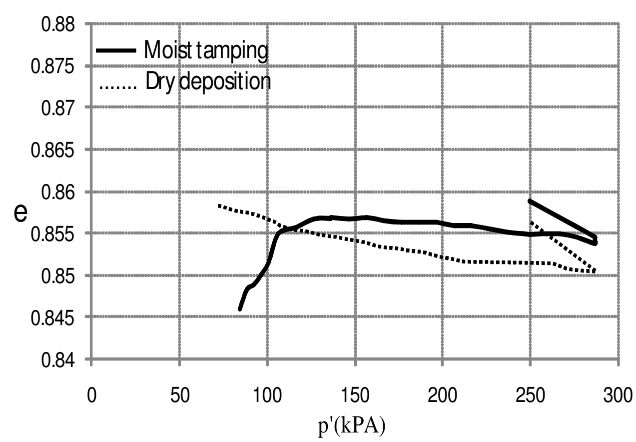


Figure 8. Behavior of moist tamped and dry deposited sand subjected to CDS loading

Figure 8 shows results of the two tests. It may be noticed that although the two samples had the same void ratio at preparation and were subjected to the same stress path, the moist tamped sample exhibited clear contraction during the latter part of the CDS loading, while the dry deposited sample remained dilative throughout the test until it reached the critical state condition. This behavior is consistent with the observations described above regarding the increased contractive properties of the moist tamped samples compared to the dry deposited samples in undrained loading.

### 4.5 Sand type

In order to compare the behaviors of various sands subjected to CDS loading, samples of Firoozkooh No. 161 sand and two other sands, namely Ottawa and Chamkhaleh sands were prepared with the same relative density, and consolidated to the same mean effective normal stress. All samples were then subjected to the same CDS stress path. The samples were all prepared to the same relative density of about zero, consolidated to a mean effective normal stress of 250 kPa and then subjected to a CDS stress path equal to 110 kPa.

Figure 9 shows results of the tests on the three sands. It may be noticed that as the mean effective normal stress was decreased following application of the deviatoric stress, the subrounded Ottawa sand started contraction earlier and at a mean effective normal stress of 122 kPa, at which the mobilized friction angle is 23 degrees, while the angular Firoozkooh sand started contraction later at a mean effective normal stress of 102 kPa, at which the mobilized friction angle is 27.1 degrees. The very angular Chamkhaleh sand, on the other hand, did not show any sign of contraction. The lack of contraction of this sand is consistent with its behavior in undrained loading, in which no contraction was observed in any undrained test under normal levels of confining stresses regardless of its void ratio (Azizi 2009).

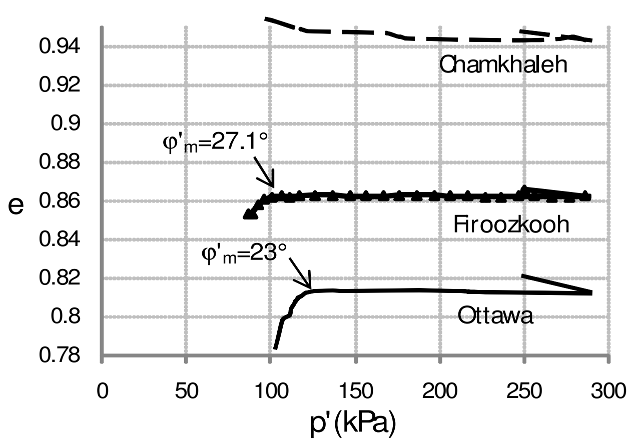


Figure 9. Behavior of three different sands with the same relative density, subjected to the same CDS stress path

It may also be observed from Figure 9 that the volume change experienced by Ottawa sand is somewhat higher than that of Firoozkooh sand. These differences may be attributed to the differences in angularity and mineralogy of the sands tests as shown in Table 1.

## 5 CONCLUSIONS

Results of constant deviatoric stress (CDS) tests carried out on sample of a uniform, fine grained, subangular sand called the Firoozkooh No. 161 sand indicated that volume contractions of the dry sand in CDS loading initiate at mobilized friction angles considerably smaller than the friction angle at failure. Such volume contractions can lead to loss of shear strength and instability of the same sand in a saturated state. These results indicate that conventional limit equilibrium stability analyses can not guarantee saftey of structures made of saturated, loose granular materials.

In looser samples, volume contractions in CDS loading are more, and occur at a smaller mobilized friction angle (i.e. at a higher mean effective normal stress). Also, for samples consolidated to similar values of state parameter, the mean effective normal stress at the onset of volume contractions, and also the amount of volume contraction is similar. Moreover, in anisotropically consolidated samples, onset of volume contractions occurs after a smaller decrease in mean effective normal stress. Such samples, which represent more closely stress state of soils in the field, may be on the verge of volume contraction and collapse at their current anisotropically consolidated state.

Examination of the effects of sample preparation and sand type on the potential for volume contraction during CDS loading showed that the potential for contraction is consistent with the behavior of the sand in undrained shearing. Samples showing higher strain softening during undrained shearing exhibit greater volume contraction during the CDS loading.

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