EXPERIMENTAL INVESTIGATION ON MONOTONIC SHEAR RESISTANCE OF GRANULAR MATERIAL: EFFECT OF THE RECONSTITUTION METHOD AND OTHER PARAMETERS

INVESTIGATION EXPERIMENTALE SUR LA RESISTANCE AU CISAILLEMENT MONOTONE D'UN SOL GRANULAIRE: EFFET DE LA METHODE DE DEPOSITION ET D'AUTRES PARAMETRES

DELLA Noureddine[1*], ARAB Ahmed[1], BELKHATIR Mostefa[1], MISSOUM Hanifi[2]
[1] *Laboratoire des Sciences des Matériaux et Environnement (LSME)*
Faculté des sciences et Sciences de l'Ingénieur
Département de Génie Civil
Université de Chlef – Algérie
[2] *Laboratoire de Construction Transport et Protection de l'Environnement (LCTPE)*
Faculté des Sciences et Sciences de l'Ingénieur
Département de Génie Civil
Université de Mostaganem – Algérie

**Auteur correspondant: Noureddine DELLA, Email: nour_della@yahoo.fr*

***ABSTRACT** : The effects of confining pressure, relative density and sample preparation methods on the shearing strength of Chlef sand were studied in this article. For this purpose, the results of drained and undrained monotonic triaxial compression tests performed on samples with initial density of 0.29 and 0.80 under initial confining pressures ranged from 50 to 200 kPa are presented. The specimens were prepared by two depositional methods namely dry funnel pluviation and wet deposition. It was found that there was a marked difference in the undrained behavior even though the density and stress conditions were identical. The conclusion was that the soil fabric was responsible for this result. The results indicated also that at low confining pressures, the specimens reconstituted by the wet deposition method exhibited complete static liquefaction (zero effective confining pressure and zero stress difference). As confining pressures and densities were increased, the effective stress paths indicated increasing resistance to liquefaction by showing increasing dilatant tendencies. The same trends were observed in drained tests results in the form of an increase in the volumetric strain and the rapid transition from the contractancy phase to the dilatancy phase.*

***KEYWORDS** : sand, drained, undrained, dry funnel pluviation, wet deposition, confinement, density, volumetric strain.*

RESUME : *Les effets de la pression de confinement, la densité relative et les méthodes de préparation des échantillons sur la résistance au cisaillement du sable de Chlef ont été étudiés dans cet article. A cet effet, les résultats des essais monotones drainés et non drainés de compression triaxiale sur des échantillons avec une densité initiale de 0,29 et 0,80 à des pressions de confinement variant de 50 à 200 kPa, sont présentés. Les échantillons ont été préparés par deux méthodes de dépôt, nommément pluviation à sec et placement humide. Il a été constaté qu'il y avait une différence marquée dans le comportement non drainé, même si les conditions de densité et de contrainte étaient identiques. La conclusion était que la structure des échantillons engendrée était responsable de ce résultat. Les résultats indiquent également qu'à de faibles pressions de confinement, les spécimens reconstitués par la méthode de placement humide montrent une liquéfaction statique complète (pression effective de confinement et différence de contraintes nulles). Lorsqu'il y'a une augmentation des pressions de confinement et des densités, les chemins de contrainte effective indiquent une résistance croissante à la liquéfaction en montrant une augmentation des tendances de dilatance. Les mêmes tendances ont été observées dans les résultats des tests de drainés sous la forme d'une augmentation de la déformation volumétrique et la transition rapide de la phase de contractance à la phase de dilatance.*

***MOTS-CLEFS** : Sable, drainé, non drainé, pluviation à sec, placement humide, confinement, la densité, déformation volumétrique.*

1. Introduction

During static or cyclic loading, the shaking of the ground may cause saturated cohesionless soils to lose their strength and behave like a liquid. This phenomenon is called soil liquefaction and will cause settlement or tipping of buildings, failures of earth dams, earth structures and slopes. The modern study of soil liquefaction has been triggered by numerous liquefaction-induced failures during the 1964 Niigata, Japan earthquake. Therefore, it is necessary to obtain a proper understanding of the effect of parameters such as soil properties and the nature of the loading on the severity of the soil liquefaction.

The region of Chlef situated near the Mediterranean Sea to the North of Algeria about 200 km to the west of the capital Algiers , by its proximity of the contact of the continental European and African plates as it is shown by Fig. 1, is constantly a very instable zone subjected to an intense seismic activity.

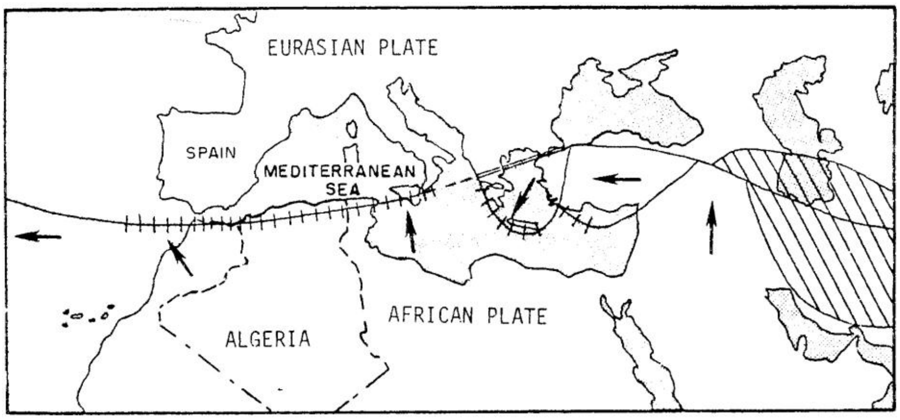


Figure 1. Movement of the Europe-African plate

On October 10th, 1980 at 13h25 (local time), the region was hit by a disastrous earthquake of a magnitude of 7.3 according to the calculations of Papastamatiou (1980), followed by strong aftershocks of magnitudes 6 and 6.1 some hours afterwards and numerous aftershocks appeared during several months (Ouyed, 1981). The main Shock generated an important inverse fault about 40 km long appearing on the surface (Ambraseys, 1981). The epicenter of this earthquake was localized in the North East of El-Asnam.

The disaster of October 10, 1980 provoked numerous losses in human lives (about 3000 deaths), the destructions of large number of buildings, important damages to the linking infrastructures and to public equipment and generated a certain number of geodynamic phenomena at the surface of the ground: movements of ground of variable nature and size, and especially the liquefaction of the sandy soils following a loss of shearing resistance. The phenomenon of liquefaction appeared on a vast alluvial valley crossed by the Chlef River and to the zone of confluence of this river with the Fodda River as it is shown in Fig. 2 (Durville and Meneroud,1982).

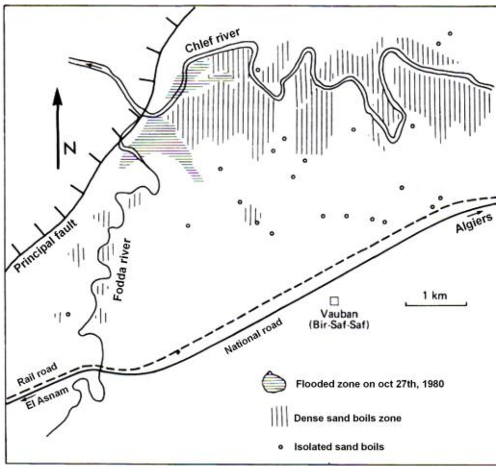


Figure 2. Valley of the Chlef River and localization of sand boils due to the liquefaction phenomenon

2. Prior studies

Numerous studies have reported that the behavior of sands can be greatly influenced by the initial state of the soil. Polito and Martin (2003) asserted that the relative density and skeleton void ratio were factors that seemed to explain the variation in different experimental results. Yamamuro and Lade (1997), Yamamuro and Lade (1998) and Yamamuro and Covert (2001) concluded that complete static liquefaction (zero effective confining pressure and zero effective stress difference) in laboratory testing is most easily achieved in silty sands at very low pressures. Kramer and Seed (1988) also observed that liquefaction resistance increased with increasing confining pressure.

Several specimen reconstitution techniques, tamping and pluviation being the most common, are in use in current practice. The objective in all of these is to replicate a uniform sand specimen at the desired void ratio and effective stresses to simulate the sand mass in-situ. However the effect of the preparation method of the samples has been subject to controversial researches. Many studies have reported that the resistance to the liquefaction is more elevated for samples prepared by the method of sedimentation than for samples prepared by dry funnel pluviation and wet deposition (Zlatovic and Ishihara, 1997); other studies have found that the specimens prepared by dry funnel pluviation method tend to be less resistant than those reconstituted by wet deposition method (Mulilis et al., 1977; Yamamuro and Wood, 2004). Other researchers indicated that the tests prepared by dry funnel pluviation are more stable and dilatant than those prepared by wet deposition (Benahmed et al., 2004; Canou, 1989; Ishihara, 1993). Vaid et al. (1999) confirm this result while showing that the wet deposition encourages the initiation of the liquefaction in relation to a setting up by pluviation under water. Yamamuro et al. (2008) concluded after their laboratory investigation, that the method of dry pluviation supports the instability of the samples contrary to the method of sedimentation. Wood et al. (2008) found that the effect of the method of deposition on the undrained behavior decreases, when the density increases. They also found that this influence decreases with the increase of the fines content, particularly with the lower densities. The focus of this study is to identify the differences in drained and undrained triaxial compression behavior that can result from using different reconstitution techniques to create silty sand specimens.

3. Experimental methods

Presented below is an experimental study of the behavior of loose and dense sand under static loading conditions. Both drained and undrained tests were performed.

3.1. Sand tested

Silty sand samples were collected from liquefied layer of the deposit areas at a depth of 6.0 m (Fig. 3) close to the Chlef earthquake epicentre (October 10th, 1980). Fig. 4 Shows craters of liquefied ground on banks of Chlef River. Fig. 5 illustrates typical subsidence location of the liquefied soil and sample collection. All tests in the present study were performed on sand from Chlef (Algeria). The sand contains 0.5% of silt of the River of Chlef. The grain size distribution curve of this sand is given in Fig. 6. It is medium sand with rounded grains of medium diameter D50=0.45mm and predominant minerals are feldspar and quartz. The silt component is non plastic with a plasticity index of 5.81%. The index properties of the sand used this laboratory research work are presented in Table 1. The specimens were reconstituted at two densities (ID = 0.29 and 0.80) representing the loose and dense states.

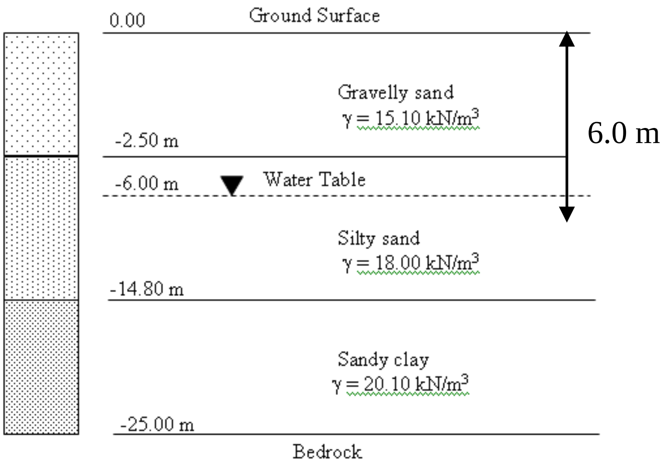


Figure 3. Geotechnical profile of the soil deposit at the site

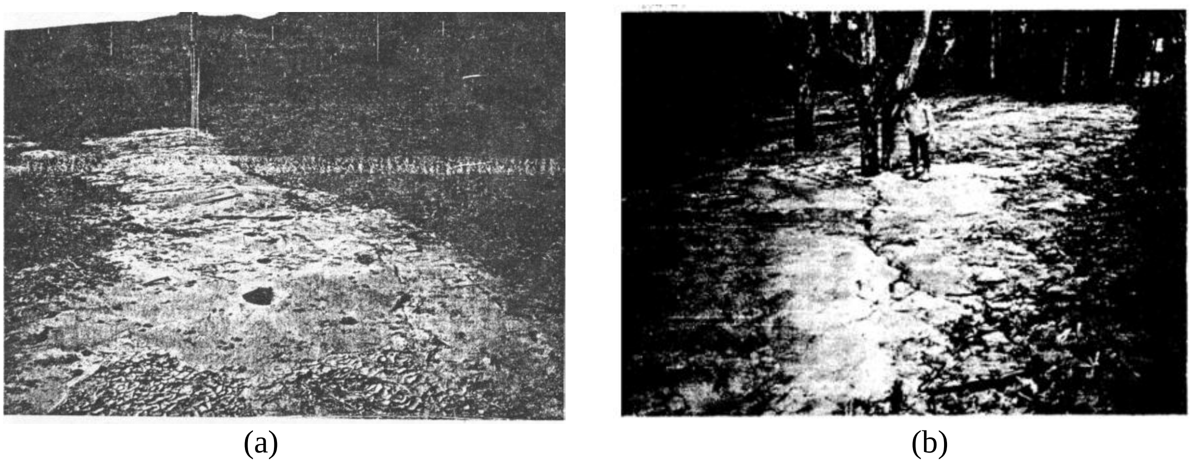

(a) (b)

Figure 4. Craters of liquefied soil on banks of the Chlef River

Figure 5. Subsidence of the Chlef River Banks due to liquefaction

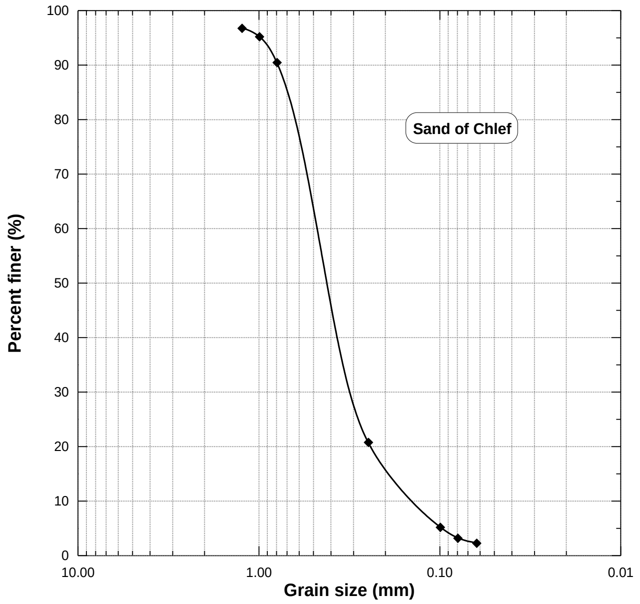


Figure 6. Grain-size distribution curve of tested material

Table 1. Index properties of the used sand

Material	e_{min}	e_{max}	γ_{dmin} (kN/m^3)	γ_{dmax} (kN/m^3)	γ_s (kN/m^3)	Cu (D_{60}/D_{10})	D_{50} (mm)	D_{10} (mm)	Grains shape
O/Chlef	0.54	0.99	13.4	17.3	26.7	3.2	0.45	0.15	Rounded

3.2. *Testing equipment*

An advanced automated triaxial testing apparatus type Bishop and Wesley was used to conduct the monotonic drained and undrained tests (Fig. 7).

Figure 7. The triaxial system set up

3.3. *Specimen preparation*

In this study, two methods of sample preparation, which included the dry funnel pluviation and the wet deposition, were utilized. The methods are briefly described below.

In the first method, the dry soil is deposited in the mould with the help of a funnel by controlling the height; this method consists in filling the mould by tipping in rain of dry sand. To have loose samples, it is necessary that the height of fall is almost nil.

The second method consists of mixing the previously dried sand with a small quantity of water (3%) and then depositing the humid soil in the mould in a as much as possible homogeneous manner. The soil was placed in successive layers. A constant number of strokes was applied to get a homogeneous and isotropic structure.

Triaxial tests were performed on cylindrical specimens measuring 70 mm in diameter and 140 mm in height (H/D = 2.0). The mass of sand to put in place is determined according to the desired density (the initial volume of the sample is known). The state of density of the sample was defined by the density index:

$$I_D = (emax - e) / (emax-emin) \qquad (1)$$

Where emin and emax indicate the minimum void ratio and the maximum void ratio, respectively; e is the target void ratio and ID density index.

After the specimen has been formed, the specimen cap is placed and sealed with O-rings, and a partial vacuum of 15 to 25 kPa is applied to the specimen to reduce the disturbances.

3.4. Saturation and consolidation

Saturation of the specimens was accomplished by flushing the specimen with carbon dioxide for approximately 20 min (Lade and Duncan, 1973), after which deaired water was slowly added from the bottom of the specimen. Application of a back pressure improves the degree of saturation which was estimated by calculating Skempton's B-parameter as the ratio of measured pore water pressure increase induced by an increase in cell pressure in undrained conditions and the corresponding increase in cell pressure. The B value was measured to test specimen saturation, and a minimum value greater than 0.96 was obtained for all tests. The triaxial test samples were isotropically consolidated under confining pressures ranging from 50 to 200 kPa prior to static loading.

3.5. Shear loading

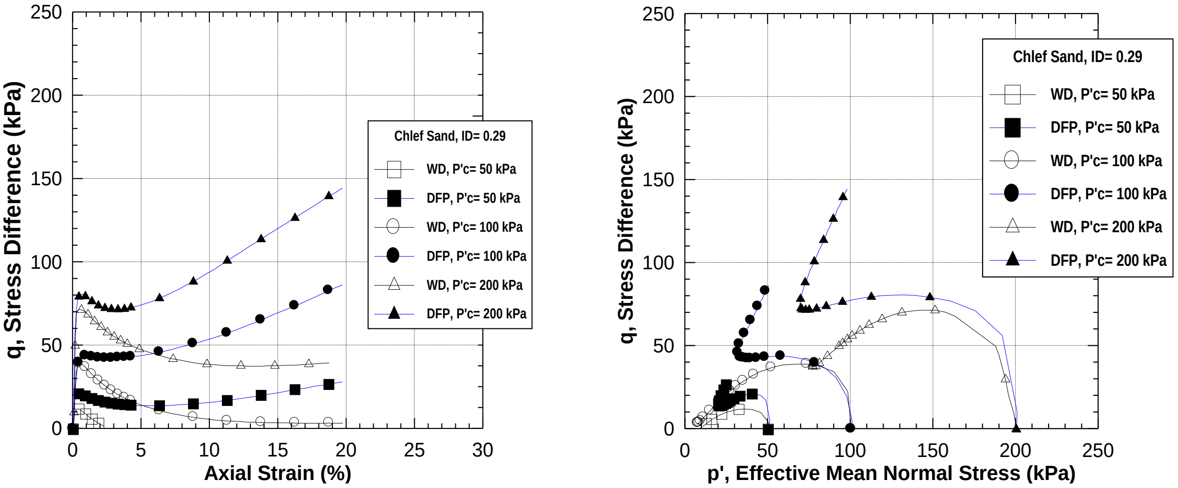
All drained and undrained triaxial tests for this study were carried out at a constant strain of 0.167% per minute, which was slow enough to allow pore pressure change to equalize throughout the sample with the pore pressure measured at the base of sample. All the tests were continued up to 20% axial strain.

4. Results of undrained triaxial compression tests

4.1. Effect of confining pressure and density

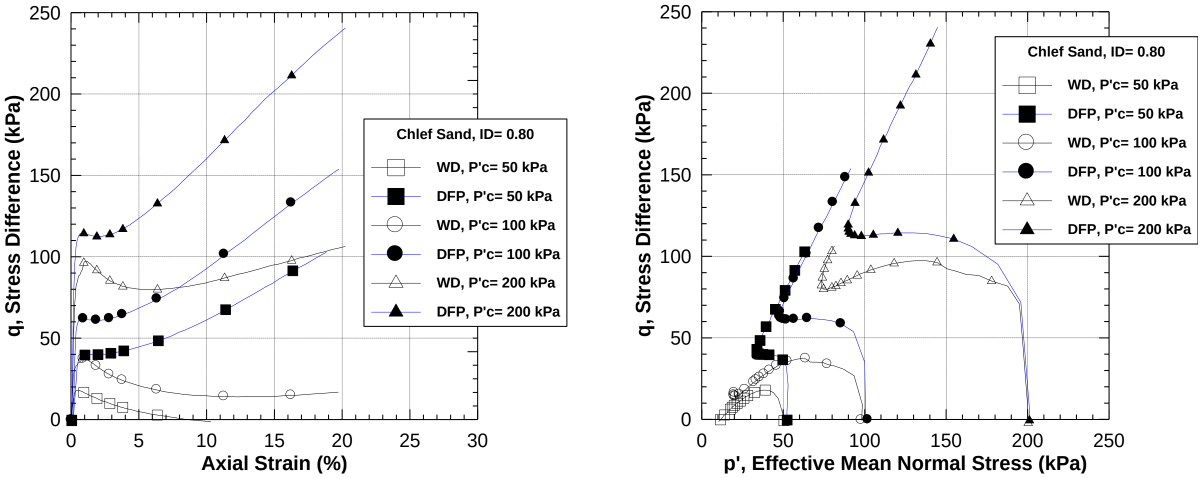
For the purpose of studying the effect of variation of effective confining pressure on liquefaction resistance, a series of tests were conducted. Figs. 8 and 9 show the results of the undrained triaxial compression tests performed in this study. All tests were performed on specimens composed of Chlef sand and each specimen was monotonically loaded in compression under undrained conditions. Figs. 8a and 9a present the undrained stress-strain curves, while Figs. 8b and 9b show the effective stress paths on the Cambridge p'-q diagram in which $p'=(\sigma'1+2\sigma'3)/3$ and $q= \sigma'1- \sigma'3$. It is noticed that as the confining pressures increased, the liquefaction resistance (deviatoric stress) increased for both dry funnel pluviation and wet deposition methods. As can be seen, for the samples reconstituted by the wet deposition method, complete static liquefaction occurred in two tests at the lowest confining pressure (50 kPa) irrespective of sand densities. Static liquefaction was coincidental with the formation of large wrinkles in the membranes surrounding the specimens. At confining pressure of 100 kPa the specimens undergo temporary liquefaction characterized by the condition where the undrained stress difference first achieves an initial peak, after which it declines to a minimum value. Finally at confining pressure of 200 kPa the résistance to liquefaction increases for both loose and dense samples.

In Figs. 8 and 9 for the dry funnel pluviation method it is clear that, when the initial confining pressure is increased from 50 kPa to 200 kPa specimens with density index of either 0.29 (loose) or 0.80 (dense) exhibit behavior that is characterised by increasing stability or increasing resistance to liquefaction. The effect of increasing confining pressure is to increase in dilatant tendencies in the soil.



(a) (b)

Figure 8. Undrained tests on loose sand:
deviator stress-strain curve, (b) stress path

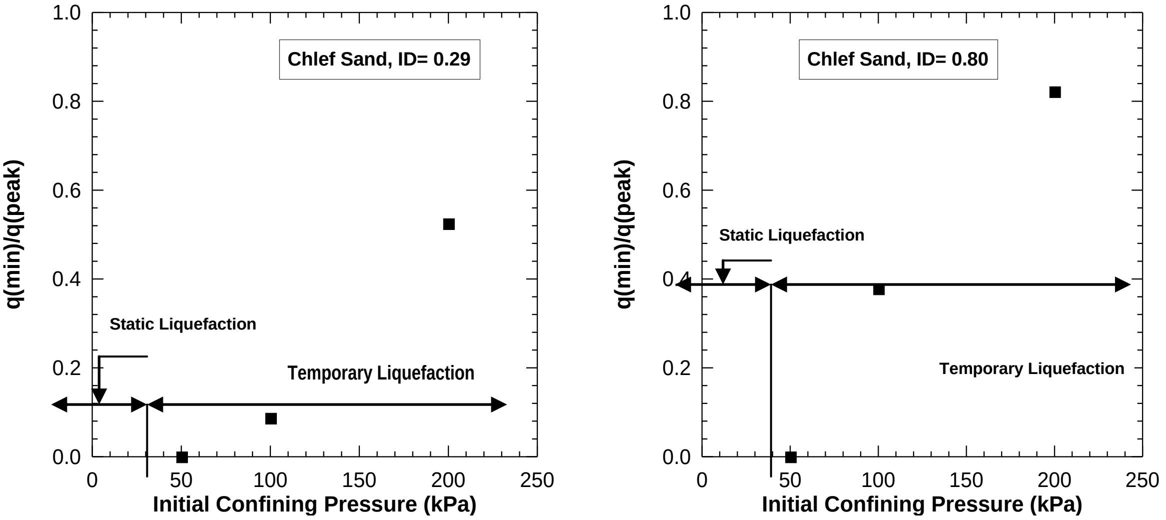


(a) (b)

Figure 9. Undrained tests on dense sand:
deviator stress-strain curve, (b) stress path

Temporary liquefaction is described as the condition where the undrained stress difference first achieves an initial peak, after which it declines to a minimum value. This is caused by rapidly rising pore pressure which decreases the effective stresses.

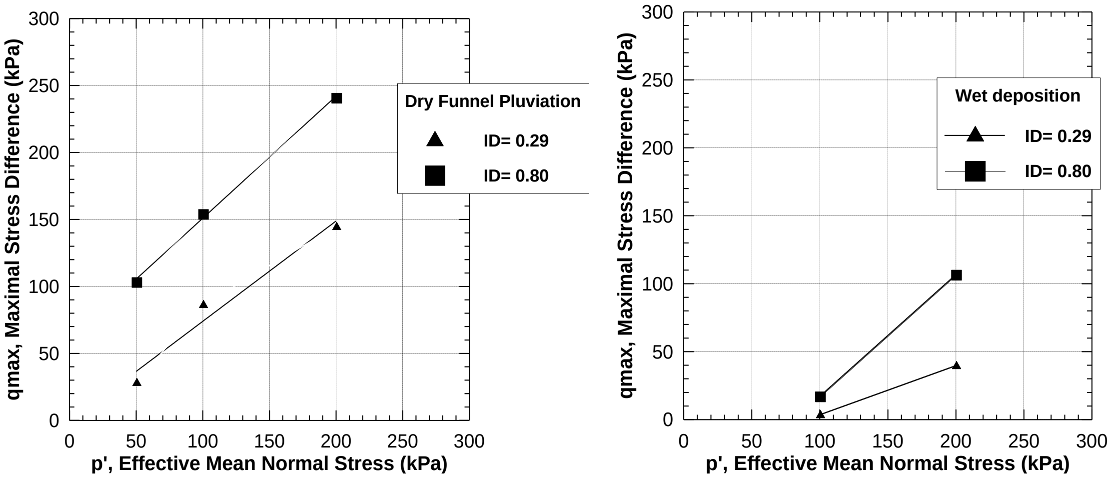
Increasing dilatancy or resistance liquefaction can also be observed by examining the ratio of the minimum stress difference to the initial peak stress difference (q(min)/q(peak)) shown in Fig. 10 for the wet deposition method. A q(min)/q(peak) ratio of zero indicates complete liquefaction, and a q(min)/q(peak) ratio of unity represents completely stable behavior. The inset diagrams in Figs. 10a and 10b show that this ratio is zero at initial confining pressure of 50 kPa, indicating complete static liquefaction. The ratio then increases at initial confining pressures from 100 to 200 kPa, indicating that the specimen exhibits more dilatancy and, thereby, more resistant to liquefaction.



(a) (b)

Figure 10. Resistance to liquefaction for the wet deposition method:
Loose state, (b) Dense state

Fig. 11 illustrates the variation of the maximal undrained shear strength (qmax) with the initial density (ID) at various confining pressures. It is clear from this figure that an increase in the relative density results in an increase in the maximal strength at a given confining pressure for both dry funnel pluviation and wet deposition, with a more pronounced increase for the method of dry funnel pluviation (Fig. 11a), contrary to the case of wet deposited samples where the evolution of the resistance is less pronounced (Fig. 11b). Thevanayagam et al. (1997) and Sitharam et al. (2004) report similar behavior of increasing undrained shear strength with increasing relative density.



(a) (b)

Figure 11. Effect of the relative density on the undrained response of sand

4.2. *Effect of the method of deposition*

Fig. 12 shows the variation of the undrained shear strength at the peak (qpeak) with the effective confining pressures using two methods of deposition. It can be seen from this figure that the dry funnel pluviation method show higher values of the deviator at peak strain, therefore a much higher resistance to liquefaction, contrary to the wet deposition method where we note some lower values of the deviator at peak for low densities (loose state for ID = 0.29) with progressive stabilization around a very small or nil ultimate stationary value representing liquefaction of the sample.

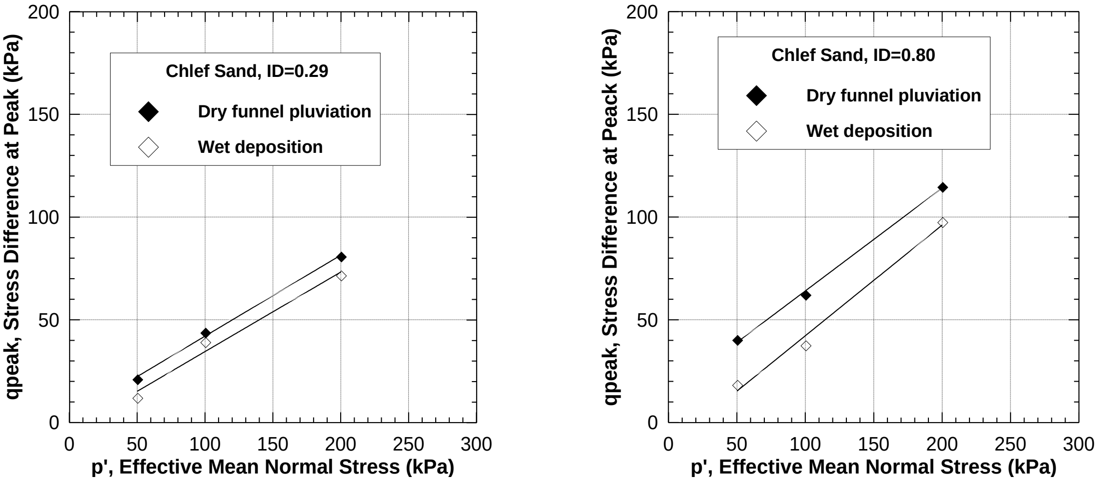


Figure 12. Effect of the deposition methods on the undrained shear strength at the peak

When loose and medium dense sandy soils are subjected to undrained loading beyond the point of peak strength, the undrained shear strength declines to a near constant value over large deformation. Conventionally, this shear strength is called the undrained steady-state shear strength or residual shear strength. However, if the shear strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction. Even limited liquefaction may result in a significant strains and associated drop in resistance. Ishihara (1993) defined the residual shear strength Sus as:

$$Sus = (qs/2)\cos\phi s = (M/2)\cos\phi s(ps') \qquad (2)$$

$$M = (6\sin\phi s)/(3-\sin\phi s) \qquad (3)$$

Where qs, ps' and ϕs indicate the deviator stress ($\sigma 1' - \sigma_3'$), the effective mean principal stress ($(\sigma_1' + 2\sigma_3')/3$) and the mobilized angle of inter-particle friction at the quasi-steady state (QSS) respectively. For the undrained tests conducted at a constant confining pressure and various initial relative densities and fines content, the deviatoric stress (q_s) was estimated at quasi-steady state point along with the mobilized internal friction angle (Fig. 13). Furthermore, the residual shear strength was calculated according to the relation (2).

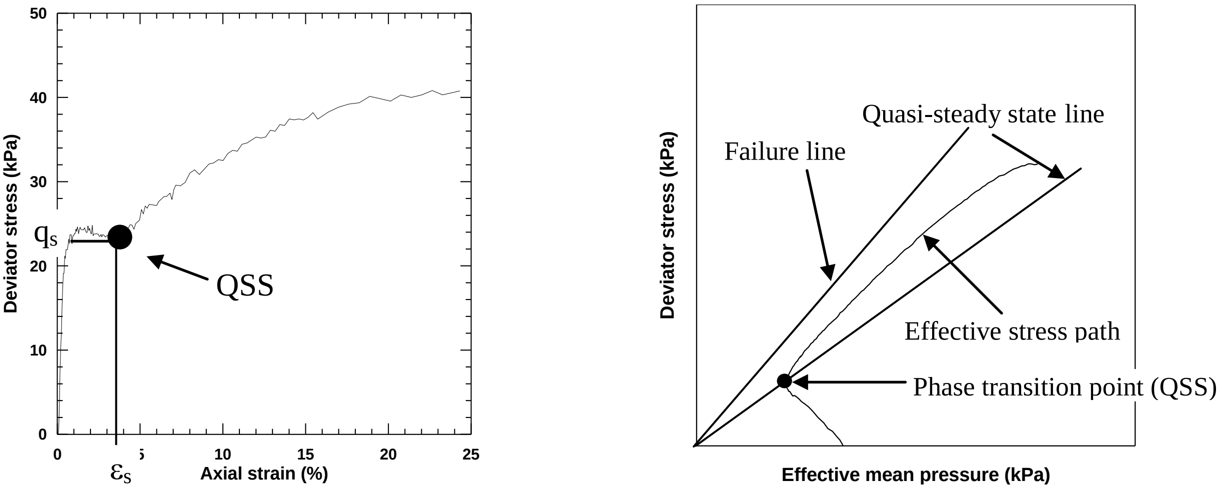


Figure 13. Determination of the phase transition point

Fig. 14 shows the evaluated undrained residual shear strength (Sus) and its variation with confining pressures and the reconstitution methods. It is clear from this figure that the samples preparation method considerably affects the evolution of the residual strength. Indeed this residual strength is nil for the samples prepared by wet deposition to a confinement of 50 kPa because of collapse of samples, but for confinements of 100 and 200 kPa, the samples prepared by dry funnel pluviation mobilize a more significant residual strength than those prepared by wet deposition.

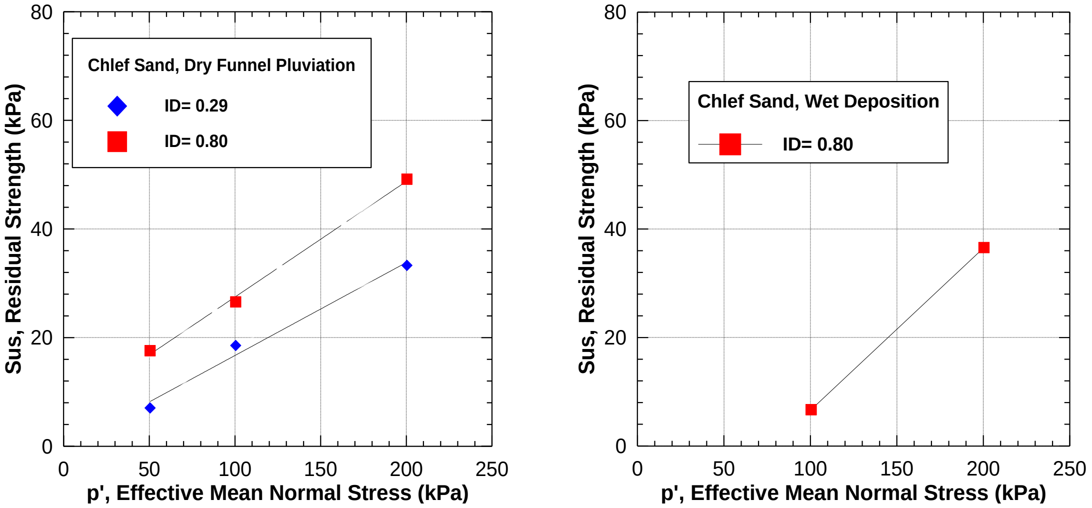
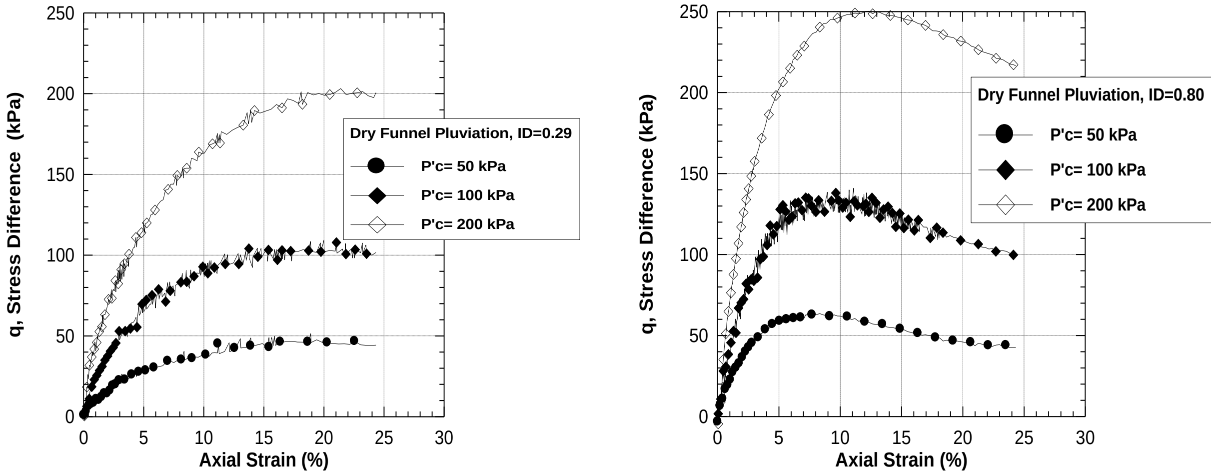


Figure 14. Effect of the deposition method on the undrained residual shear strength

5. Results of drained triaxial compression tests

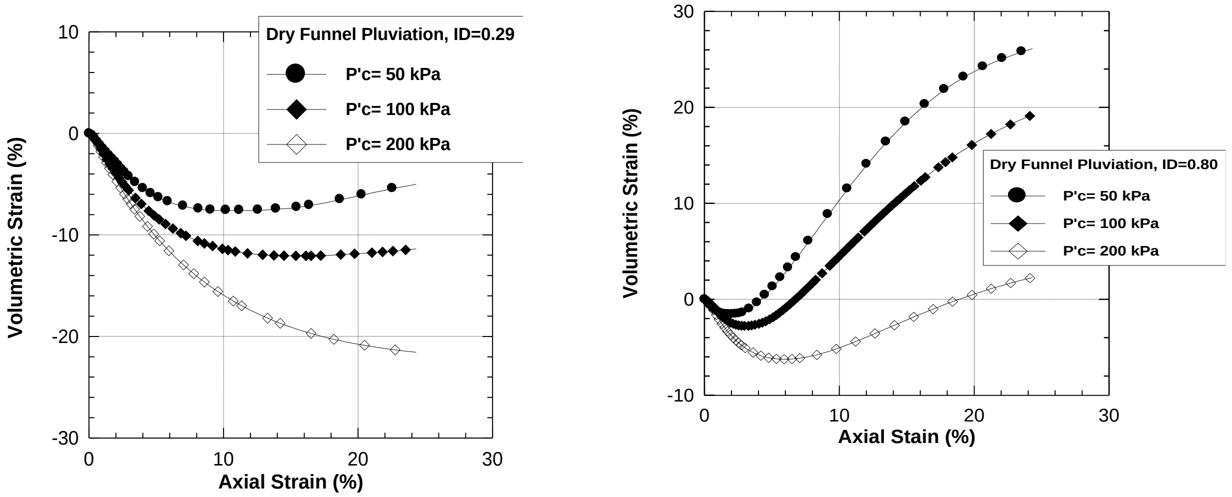
Figs. 15 and 16 show the results of the drained tests on samples prepared by the method of dry funnel pluviation with two densities (I_D=0.29 and 0.80).¶ It can be noticed from Fig. 15, that the resistance to liquefaction representing by deviatoric stress increases with increase in the confining pressure and in density¦. ¶Fig. 16 shows the evolution of the volumetric strain versus the axial strain. ¶We note that the increase in the density accelerates the transition from the contractancy phase to the dilatancy phase.¶



(a) (b)

Fig. 15. Evolution of the deviatoric stress versus axial strain

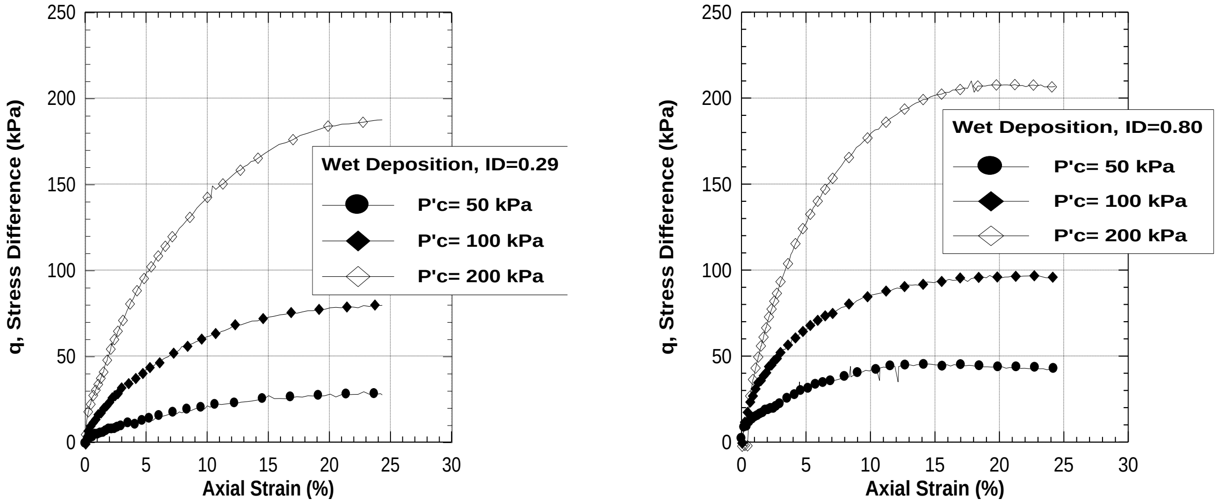
(a) Loose state (ID =0.29), (b) Dense state (ID =0.80)



(a) (b)

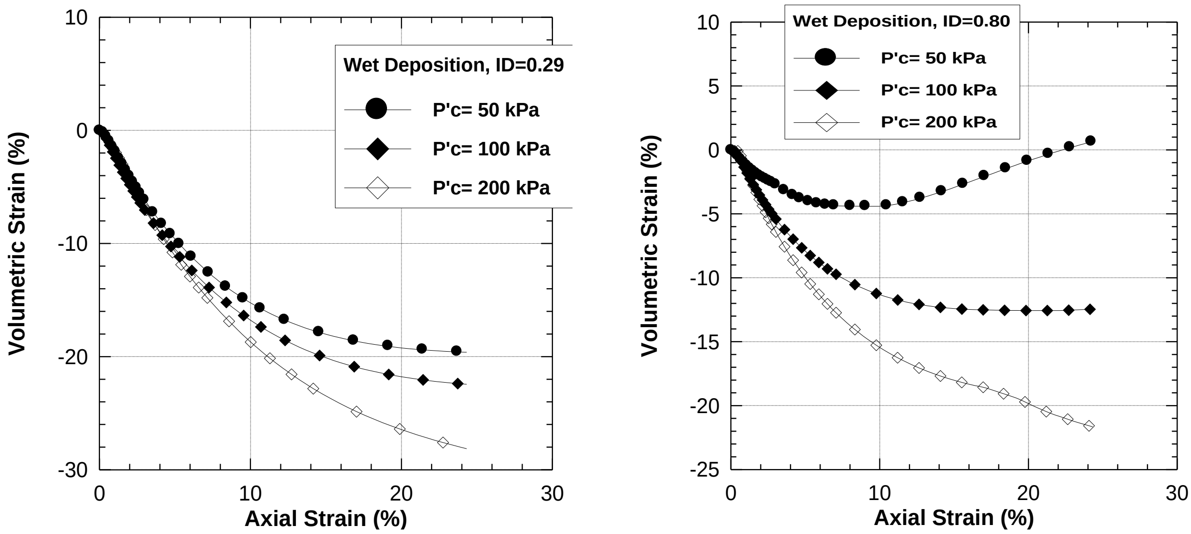
Figure 16. Evolution of the volumetric strain versus axial strain
(a) Loose state (ID =0.29), (b) Dense state (ID =0.80)

The same tendencies can be observed in Figs. 17 and 18 which show the results of the drained tests on specimens reconstituted by the wet deposition method¦. As can be seen from Fig. 17,¶ the resistance to liquefaction representing by the deviatoric stress, increases with increase in the confining pressure and in density. ¶Fig. 18 shows the evolution of the volumetric strain versus axial strain. It can be noticed¶ that the method of wet deposition increases the phase of ¦contractancy. This increase in the phase of contractancy is highly marked for the loose specimens (Fig. 18a).



(a) (b)

Figure 17. Evolution of the deviatoric stress versus axial strain
(a) Loose state (ID =0.29), (b) Dense state (ID =0.80)



(a) (b)

Fig. 18. Evolution of the volumetric strain versus axial strain
(a) Loose state (ID =0.29), (b) Dense state (ID =0.80)

By comparing the results of Figs. 15, 16, 17 and 18, we concluded that the specimens reconstituted by dry funnel pluviation method were more dilatant than those prepared by wet deposition method.

6. Conclusion

A series of drained and undrained triaxial compression tests in monotonic loading conditions were performed on silty sand samples retrieved from liquefied sites at Chlef River banks (Algeria). The effects of sample preparation methods and other parameters were studied. The study included drained and undrained triaxial tests that have been prepared at densities of 0.29 and 0.80 for confinements of 50,100 and 200 kPa. Based on the experimental results presented, the following conclusions can be drawn:

1. Complete static liquefaction occurred at low confining pressure (50 kPa) for the wet deposition method.
2. As the confining pressure increased, the liquefaction resistance of the sand increased for both dry funnel pluviation and wet deposition. This observation correlates with most historic cases of apparent static and earthquake-induced liquefaction.
3. An increase in the density resulted in an increase in the maximal undrained shear strength of the sand in undrained tests and accelerates the transition from the contractancy phase to the dilatancy phase in drained tests.
4. The peak and residual shear strengths of sand are sensitive to the sample preparation methods. Dry funnel pluviation method gives higher values of the peak and residual shear strengths than wet deposition method.
5. The results reveal also that the method of reconstitution has a detectable effect on the drained behavior of the sand in terms of volumetric strains. Dry funnel pluviation method appeared to indicate a more volumetrically dilatant or stable response, while wet deposition method appeared to exhibit a more contractive or unstable behavior.

7. References

Ambraseys N.N., *The El-Asnam earthquake of 10 october 1980: conclusions drawn from a field study*, Q. J. Engineering Geology, London, 14, 143-148, 1981.

Benahmed N., Canou J., Dupla J.C., *Structure initiale et propriétés de liquéfaction statique d'un sable*, Comptes Rendus Mécanique 332, 887-894, 2004.

Canou J., *Contribution l'étude et à l'évaluation des propriétés de liquéfaction d'un sable*, Thèse de Doctorat de l'Ecole Nationale Des Ponts et Chaussées, Paris, 1989.

Durville J. L., Meneroud J. P., *Phénomènes géomorphologiques induits par le séisme d'El-Asnam, Algérie*, Bull. Liaison Labo. P. et Ch., 120, juillet-août, 13-23, 1982.

Ishihara K., *Liquefaction and flow failure during earthquakes*, Géotechnique, 43/3, 351-415, 1993.

Kramer S.L., SeedEED H.B., *Initiation of soil liquefaction under static loading conditions*, Journal of Geotechnical Engineering, 114/4, 412-430, 1988.

Lade P.V., Duncan J.M., *Cubical triaxial tests on cohesionless soil*. Journal Soil Mechanics and Foundations Division ASCE, 99/SM10, 793-812, 1973.

Mulilis J.P., Seed H.B., Chan C.K., Mitchel J.K., Arulanada K., *Effects of sample preparation on sand liquefaction*, Journal of Geotechnical Engineering Division, ASCE, 103, 91-108, 1977.

Ouyed M., *Le tremblement de terre d'El-Asnam du 10 octobre 1980: étude des répliques*, Ph.D. Thesis, University of Grenoble, Grenoble, 200 p., 1981.

Papastamatiou D., *El-Asnam, Algeria earthquake of October 10, 1980: field evidence of ground motion in the epicentral region*, Geognosis Ltd., London, 1980.

Polito C.P., Martin II J.R., *A reconciliation of the effects of non-plastic fines on the liquefaction resistance of sands reported in the literature*. Earthquake Spectra, 19/3, 635-651, 2003.

Sitharam T.G., Govinda R.L., Srinivasa M.B.R., *Cyclic and monotonic undrained shear response of silty sand from Bhuj region in india*, ISET Journal of Earthquake Technology, Paper 450/41, 249-260, 2004.

Thevanayagam S., Ravishankar K., Mohan S., *Effects of fines on monotonic undrained shear strength of sandy soils*, ASTM Geotechnical Testing Journal, 20/1, 394-406, 1997.

Vaid Y.P., Sivathayalan S., Stedman D., *Influence of specimen reconstituting method on the undrained response of sand*, Geotechnical Testing Journal, 22/ 3, 187-195, 1999.

Wood F.M., Yamamuro J.A., Lade P.V., *Effect of depositional method on the undrained response of silty sand*, Canadian Geotechnical Journal, 45/11, 1525–1537, 2008.

Yamamuro J.A., Covert K.M., *Monotonic and cyclic liquefaction of very loose sands with high silt content*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127/4, 314-324, 2001.

Yamamuro J.A., Lade P.V., *Static liquefaction of very loose sands*, Canadian Geotechnical Journal, 34 /6, 905-917, 1997.

Yamamuro J.A., Lade P.V., *Steady state concepts and static liquefaction of silty sands*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124/9, 868-877, 1998.

Yamamuro J.A., Wood F.M., *Effect of depositional method on the undained behavior and microstructure of sand with silt*, Soil Dynamics and Earthquake Engineering, 24, 751-760, 2004.

Yamamuro J.A., Wood F.M., Lade P.V., *Effect of depositional method on the microstructure of silty sand*, Canadian Geotechnical Journal, 45/11, 1538–1555, 2008.

Zlatovic S., Ishihara K., *Normalized behavior of very loose non-plastic soils: effects of fabric*, Soils and Foundations, 37/4, 47-56, 1997.