

DILATANCY BASED IDENTIFICATION OF LIQUEFACTION SUSCEPTIBILITY

by

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ABSTRACT

DILATANCY BASED IDENTIFICATION OF LIQUEFACTION SUSCEPTIBILITY

This thesis focuses on liquefaction and dilatancy, both of which still remain as complex and controversial phenomena in geotechnical engineering. Even though the theoretical link between liquefaction susceptibility and dilatancy characteristics is known in geotechnical literature, quantification and practical definition of this relationship could not be possible due to the difficulties associated with the measurement of dilatancy angle. Thus, the main goal of this study is to develop a simple method which would allow the computation of liquefaction susceptibility based on the dilatancy of the soil. In this study, liquefaction initiation is achieved by cyclic loading which results in cyclic mobility of cohesionless soil samples, using constant-volume direct simple shear tests. Over 275 tests on 3 different sand types were conducted. The individual effects of test characteristics; such as cyclic frequency, cyclic stress ratio and consolidation pressure on test results were investigated. Moreover, the effect of average particle shape and size, both on dilatancy and liquefaction susceptibility was also investigated. Relationship between the peak dilation angle and number of loading cycles to liquefy were obtained from the results of direct simple shear tests and in order to link dilatancy to cyclic mobility, a novel empirical equation was used. This equation enabled the estimation of peak dilatancy from soil's relative density and mean effective stress, using the data obtained from consolidated drained triaxial tests, conducted on the sand types under investigation.

ÖZET

ZEMİNLERİN SIVILAŞMA YATKINLIĞININ GENLEŞİME BAĞLI OLARAK BELİRLENMESİ

Zemin genleşimi ve zemin sıvılaşması, günümüz zemin mekaniğinde bile karmaşık ve tartışmaya yola açan kavramlar olarak kalmaya devam etmektedirler. Zemin genleşiminin sıvılaşma üzerindeki etkileri, literatürde yaygın bir şekilde kabul edilmektedir. Ancak genleşim açısını, dinamik geoteknik deneylerini kullanarak bulabilmenin zorluklarından dolayı, zemin genleşiminin sıvılaşma üzerindeki etkisi, şu ana dek teorik olarak incelenmiş ve niteliksel yaklaşımlar geliştiren çalışmalar henüz oluşturulmamıştır. Bu sebeplerle, bu çalışmanın amacı zemin genleşiminin sıvılaşma üzerine etkilerini basit ve rakamlara dökülebilen bir yaklaşım geliştirerek tanımlayabilmektir. Bu çalışmada, kohezyonsuz numunelerde sıvılaşma çevrimsel yükleme ile başlatılmış ve numunenin çevrimsel mobilitesi, sabit hacimli çevrimsel basit kesme deneyleri kullanılarak sağlanmıştır. 3 farklı kum türünde 275'den fazla deney yapılmıştır. Deney karakteristiklerinden olan çevrimsel frekans, çevrimsel kayma gerilmesi oranı ve düşey gerilme değerlerinin deney sonuçlarındaki etkileri incelenmiştir. Bunlarla birlikte, kullanılan kum türlerinin ortalama dane şekli ve boyutunun zemin genleşimi ve sıvılaşması üzerindeki etkileri de ele alınmıştır. Zemin genleşimi ve sıvılaşması arasındaki ilişkiyi yakalamak için, en büyük genleşim açısını zeminin göreceli sıkılık ve ortalama efektif gerilme değerlerini kullanarak hesaplayabilen orijinal ampirik bir formül kullanılmıştır. Bu formülde kullanılacak veriler incelenen kum türleri üzerinde yapılmış konsolidasyonlu drenajlı üç eksenli deney sonuçlarından sağlanmıştır. Alınan sonuçlar hem grafiklerle hem de formüllerle sunulmuştur.

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LIST OF SYMBOLS

A	Cross sectional area
A_o	Initial sample area
A_c	Corrected sample area
A_m	Area of the membrane
B	Ratio of specimen pore pressure change to cell pressure change
C_u	Uniformity Coefficient
C_c	Curvature Coefficient
D_c	Specimen diameter after consolidation
e	Void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
E_m	Young's modulus
F	Force Applied to Stretch the Membrane
F_b	Buoyancy force
G_s	Specific gravity
h_o	Initial specimen height
h_c	Specimen height after consolidation
I_D	Relative density
K_o	Ratio of horizontal stress to vertical stress at-rest
L	Unstretched Length of the Membrane
L_M	Length of the major axis
m_{ψ}	Dilatancy constant value for Cinicioglu & Abadkon (2015) Relationship
p	Perimeter
p_a	Atmospheric pressure
p'	Mean effective stress
pi'	Pre-shear confinement pressure
R	Empirical Line Fitting Parameter
r_u	Pore water pressure ratio

Q	Empirical Line Fitting Parameter
tm	Membrane Thickness
V	Volume
V_c	Sample Volume after the Consolidation Phase
I_e	Difference between Maximum void ratio and Minimum void ratio
α_ψ	Dilatancy constant value for Cinicioglu & Abadkon (2015) Relationship
ϕ'_{cs}	Critical State Friction Angle
σ'_n	Effective vertical stress
σ_c	Cell pressure
ΔL	Change in Length of the Membrane
ΔV_s	Volumetric Change of the sample during the Shearing Phase
Δh	Change in Height of the Specimen
ε_v	Volumetric Strain
ε_1	Axial Strain
Δu	Change in pore water pressure
$\Delta \sigma_3$	Change in cell pressure
γ_w	Unit weight of water
γ_s	Unit weight of soil
W_s	Dry unit weight of soil
Γ	Critical State Material Parameter
λ	Critical State Material Parameter

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ASTM	American Society for Testing and Material
CD	Consolidated Drained
CF	Cyclic Frequency
CSR	Cyclic Stress Ratio
CSS	Cyclic Simple Shear
LM	Light Microscope
LVDT	Linear Variable Differential Transformer
NCL	Number of Cycles to Liquefy
NGI	Norwegian Geotechnical Institute
NIH	National Institute of Health
NNCL	Normalized Number of Cycles to Liquefy
SEM	Scanning Electron Microscope
SGI	Swedish Geotechnical Institute
WSSSF	Weighted Single Sand Shape Factor

1. INTRODUCTION

Saturated loose cohesionless soils lose their initial stiffness and strength at liquefied state, as a result of increase of pore water pressure caused by undrained shear loading (Kramer, 1996). In geotechnical literature, it is a commonly accepted fact that, increased dilatant properties help soil to have greater liquefaction resistance. However, owing to problems associated with the measurement of dilatancy angle in cyclic tests, influence of dilatancy on liquefaction susceptibility has not been clearly established. Thus, the main aim of this study is to investigate the link between dilatancy and liquefaction potential.

Dilatancy angle is a function of soil density and effective stress state. Moreover, it should be made clear that dilatancy angle varies with strain level and the value of the dilatancy angle that influences behaviour is the peak (Abadkon 2012). Since peak dilatancy angle can only be measured at the instance of failure, it is not possible to obtain this value in cyclic tests. Accordingly, this study attempts to overcome this difficulty by calculating the peak dilatancy angle from the initial density and effective stress state of the soil. A specially calibrated empirical equation (Çinicioglu and Abadkon, 2015) is used for this purpose.

Once the constants of the dilatancy equation are obtained through K_o consolidated-drained (CD) triaxial tests, peak dilatancy angles of each cyclic simple shear samples' peak dilation angle can be calculated. Cyclic simple shear tests were conducted to measure the liquefaction resistance corresponding to different peak dilatancy angle values. Tests were repeated with different cyclic stress ratio and cyclic frequency values. As a result, figures were prepared explaining the influences of stress ratio amplitude and cyclic period on peak dilatancy-number of cycles to liquefaction relationship.

On the other hand, particle shape characteristics and its impacts on liquefaction susceptibility is also studied in geotechnical literature. Dodds (2013) states that global form or shape of a sand particle is at first determined during formation and later by

the effects of weathering. Vaid *et al.* (1985) compared sands with the same gradation and different angularity. It was suggested that liquefaction susceptibility for rounded sand was higher than for angular sand at low confining pressures. Cho *et al.* (2006) concluded that increasing angularity resulted in increase in critical state friction angle. Cox (2008) used both light microscope and scanning electron microscope for the determination of particle shape characteristics and suggested that the differences between the results of two microscopes did exist however were not significant.

In this study, particle shape characteristics were determined using image processing software and light microscope. Average circularity and roundness values of three different sand types were examined. Moreover, the effect of these factors on overall test results are discussed.

2. LITERATURE REVIEW

2.1. Liquefaction

Liquefaction is one of the complex and widely discussed topics in geotechnical engineering field, which continues jeopardizing public safety and causing financial losses. Year of 1964 was a turn of events for geotechnical earthquake engineers' research point of view towards liquefaction. Two devastating earthquakes causing liquefaction occurred in 1964, in March, Good Friday Earthquake, Anchorage, Alaska ($M_w = 9.2$) and in June, Niigata Earthquake, Niigata Japan ($M_s = 7.5$).

General definition of liquefaction in the literature emphasizes on undrained loading conditions causing excess pore water pressure generation. Undrained rapid loading of loose saturated cohesionless soils leads to a state of densification. Since water cannot escape (undrained), pore water pressure increases, consecutively causing effective stresses to decrease.

A quick sand tank filled with loose sand, having piezometers at various levels is a good way to observe liquefaction. With the sudden increase of pressure, soil mass liquefies and starts acting like a dense liquid which can be easily stirred with a stick. Liquefied mass has no effective stress between sand grains, leading to a zero shear resistance (Holtz and Kovacs, 1981).

Flow liquefaction and cyclic mobility are two main categories of liquefaction phenomena. Flow liquefaction is observed when required static shear stress which maintains the equilibrium of soil mass, is greater than the residual shear strength of soil in its liquefied state. Flow liquefaction is actually driven by static shear stresses and character traits of this type of liquefied mass are suddenness, speediness and moving large distances. Flow liquefaction can be a consequence of several events, such as monotonic loading, pile driving, train traffic, geophysical exploration and blasting. On the other hand, deformations in cyclic mobility which is a seismic phenomenon, develops

gradually due to buildup pore water pressure. These deformations are a result of both static and cyclic shear stresses (Kramer, 1996).

Another important difference between flow liquefaction and cyclic mobility is that most likely, flow liquefaction takes place in uniform, fine, clean and loose sands. However if the cyclic stresses are large enough for a sufficient period of time, cyclic mobility could be observed in any sand, in any state in the laboratory (Holtz and Kovacs, 1981).

Most common approach to assess triggering point of liquefaction for laboratory tests is evaluating liquefaction potential based on the constant amplitude of cyclic shear stresses and number of cycles applied. “Initial liquefaction”, defined by (Seed and Lee, 1966), is the point where $r_u = 100\%$, meaning that excess pore water pressure is equal to the initial effective stress. Another definition to pinpoint liquefaction failure is to consider a limit of peak-to-peak axial strain.

2.2. Cyclic Simple Shear Device

In order to mimic cyclic response of specimens in laboratory, cyclic triaxial and cyclic direct simple shear tests are frequently used by many researchers. The cyclic direct simple shear test can reproduce earthquake stress conditions much more accurately than cyclic triaxial test (Kramer, 1996).

One of the advantages of cyclic direct simple shear test is that; soil specimen is consolidated under K_o condition with zero horizontal deformation. Zero horizontal deformation is ensured by stacked rings or wire reinforced rubber membrane, which confines the cyclic direct simple shear specimen and keeps the diameter constant.

These arrangements simulate the real subsoil behavior as can be seen in Figure 2.1, where horizontal plane cyclic shear stresses are produced with the vertical propagation of S waves. Cyclic shear stresses alternates in positive and negative directions; causing negative dilatancy and excess pore water pressure build-up, eventually causing

liquefaction (Towhata, 2008).

Moreover, “constant volume” cyclic simple shear testing helps specimen to maintain its volume throughout the shearing phase, simulating undrained conditions which is of importance for cyclic loading. Dyvik *et al.* (1987) compared results of truly undrained tests and constant volume drained tests on normally consolidated clay specimens and concluded that two methods are equivalent for all practical purposes.

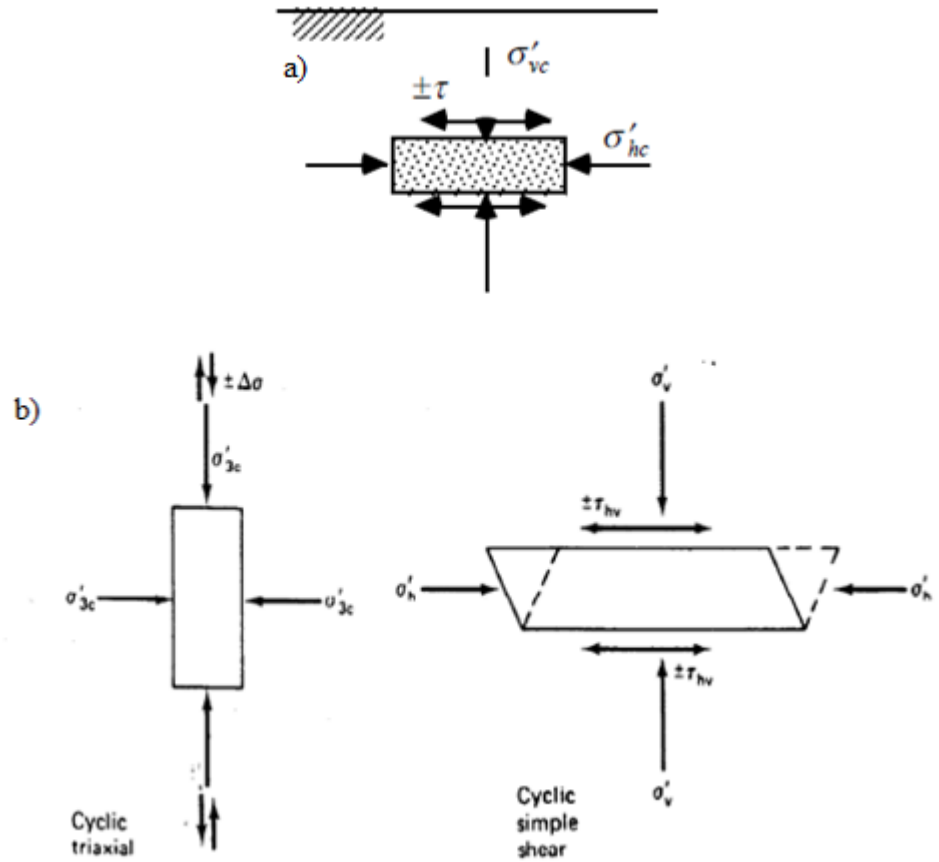


Figure 2.1. a) Seismic Stress State in Ground Level, (Towhata, 2008), b) (Holtz and Kovaks, 1981).

Constant volume is achieved by restricting vertical displacement and fixing the top and bottom loading caps to resist vertical movement. Additionally, change in pore water pressure is calculated from the change in total stress, which results from the attempt of keeping the specimen height constant (ASTM D6528).

Some disadvantages of this device is that; its inability to test undisturbed soil samples and boundary effects, which create non-uniformity of stress state within the specimen, near the specimen boundaries (Cole, 1967). Based on the 3D finite element analysis on NGI type DSS specimen by Lucks *et al.* (1972) it is shown that 70% of the specimen was subjected to uniform stress field. According to Shen *et al.* (1978), boundary effects in DSS configuration are closely related to the height to diameter ratio of specimen (H/D) and decrease in this ratio helps the improvement of boundary conditions.

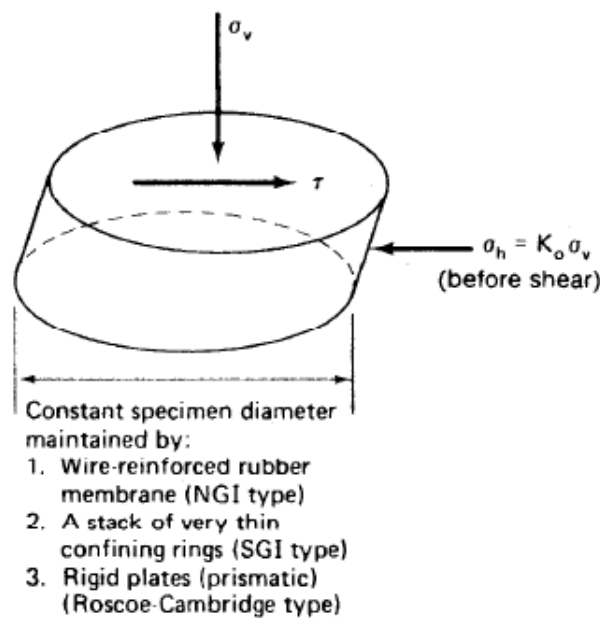


Figure 2.2. Simple Shear Test Sample (Holtz and Kovaks, 1981).

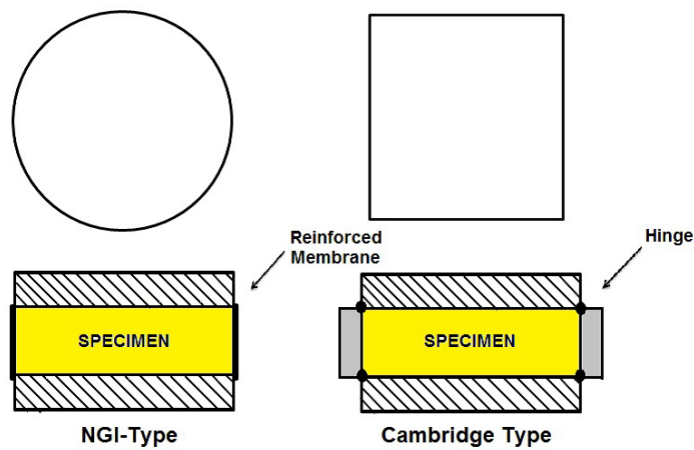


Figure 2.3. Different Types of Simple Shear Apparatus (Akin, 2014).

“Cambridge-type” and “Norwegian (NGI) and Swedish (SGI) Geotechnical Institute - type” are two common types of cyclic direct simple shear device, which are widely being used in research. As it can be seen from Figure 2.3, the main differences between these two types are, the cross section and confinement type of the specimen. Cambridge type device has its cuboidal specimen confined by rigid boundary platens which can rotate by their two hinged ends. SGI-type was first proposed by (Kjellman, 1951) and this type was modified by Norwegian Geotechnical Institute with a cylindrical sample of soil surrounded by a wire-reinforced rubber membrane (Bjerrum and Landva, 1966). If wire-reinforced rubber membrane was used as confinement, the device was called NGI-type and if stacked metal rings were used, it was called a SGI-type. Later, the distinction between SGI-type and NGI-type disappeared, and now they both are called NGI-type.

2.3. Dilatancy and Its Influence on Liquefaction Susceptibility

Dilatancy is another important soil behavior, which can be described by the tendency of soils to dilate or to contract during drained shear or pore water pressure changes during undrained shear. These shear stresses at the boundary of soil sample causes the disturbance of arrangement of grains. Accordingly, the rearrangement of soil grains leads to a change of volumetric packing of the sample.

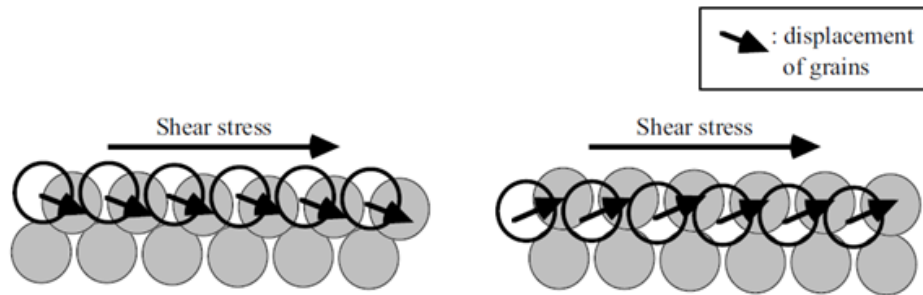


Figure 2.4. Displacement of Grains (Towhata, 2008).

As it is shown in the Figure 2.4, shear stress causes loose sands to compress, because of down movement of grains towards large voids, which leads to decreasing

height of sandy deposit. On the other hand, dense sand increase in volume due to the fact that the soil grains tend to climb top of each other. However, it should be noted that the dilatant behavior for dense sands is closely related with confining stress. For example, dense sands increase in volume under low effective stresses. On the contrary, it is also possible to encounter a decrease in sample volume under higher stresses for the same sand (Towhata, 2008).

According to Youd (1977), because of packing rearrangements caused by cyclic loading, sandy deposit will go through a repetitive shear-dilation process as shown in Figure 2.5. These steps of dilatant behavior leads to instances of pore pressure reduction where a regain in shear stiffness and strength occurs.

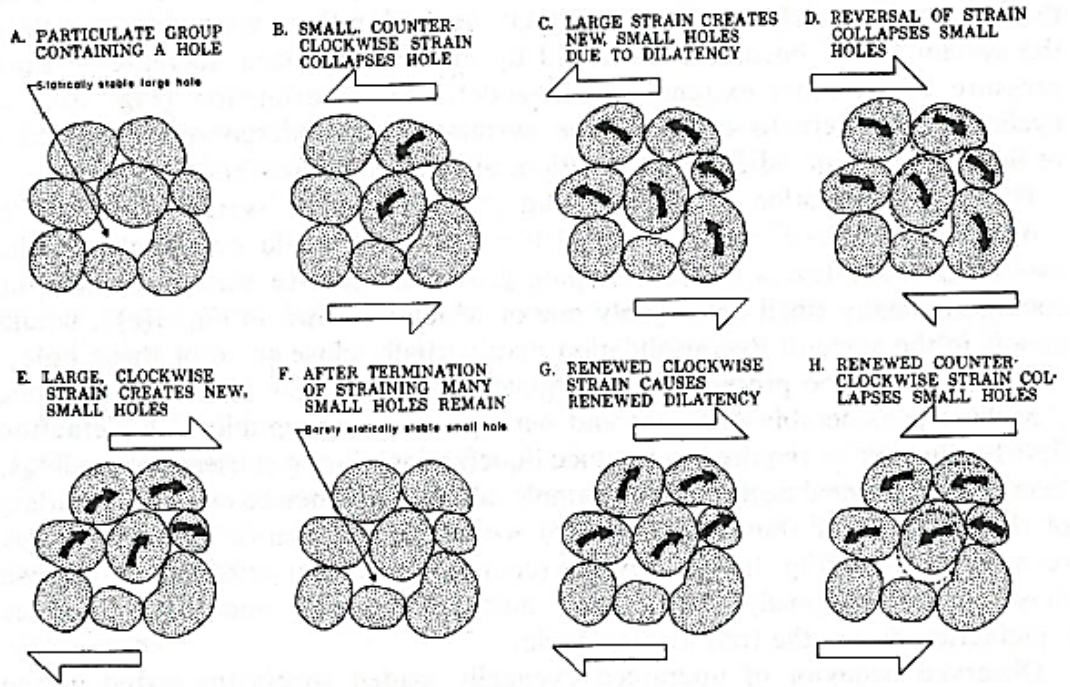


Figure 2.5. Repetitive Shear-Dilation Process under Cyclic Loading (Youd, 1977).

Bishop (1972), suggested that peak angles of shearing resistance (ϕ'_c) and dilation (ψ_p) had a linear relationship. After collating the available data of seventeen different sand types from the literature, in his pioneering work, (Bolton, 1986) managed to simplify the relationship between ϕ'_p and dilation (ψ_p) and suggested the linear relationship

which was first hypothesized by Bishop.

$$\phi'_p = \phi'_c + r\psi_p \quad (2.1)$$

For the equation above, r is the line fitting parameter and ϕ'_c is the critical state friction angle.

Bolton (1986) also proposed the following equation, which was first to allow the quantification of influences of relative density and confining pressure on peak angles of shearing resistance (ϕ_p') and dilation (ψ_p).

$$\left(-\frac{d\varepsilon_v}{d\varepsilon_1}\right)_{max} = 0.3 \left[I_D (Q - \ln p'_f) - R \right] \quad (2.2)$$

where I_D = relative density, Q and R = empirical line fitting parameters that depend on inherent soil characteristics and the unit chosen for mean effective stress at peak (p'_f). Left hand of the equation can be defined as dilatancy rate in triaxial test. Later, based on Chakraborty and (Salgado, 2010) work, showed that the line fitting parameter, Q is not constant and varies with pre-shear consolidation pressure, leading Equation 2.1 to be only used for plane strain conditions.

Based on triaxial test results, (Schanz and Vermeer, 1996) proposed the following equation to be used for axisymmetric samples.

$$\sin\psi = -\frac{d\varepsilon_v/d\varepsilon_1}{2 - d\varepsilon_v/d\varepsilon_1} \quad (2.3)$$

For the above equation, $d\varepsilon_v$ is volumetric strain increment and $d\varepsilon_1$ is axial strain increment.

Inspired mainly by work of researchers mentioned above, (Abadkon and Cini-cioğlu, 2015) conducted conventional drained triaxial compression tests consolidated under K_0 condition to investigate the uncoupled effects of pre-shear mean effective

stress (p'_0) and volumetric state as defined by pre-shear relative density (I_D) on peak dilation angle ψ_p . They grouped the test results into several small I_D ranges showed that for constant I_D values, $p'_i/p_a - \tan \psi_p$ relationship is linear.

$$\tan \psi_p = \alpha_\psi \left(\frac{p'_i}{p_a} \right) + \beta_\psi \quad (2.4)$$

where β_ψ is a constant value depending on I_D . So the equation above can be written as:

$$\tan \psi_p = \alpha_\psi \left(\frac{p'_i}{p_\alpha} \right) + m_\psi I_D \quad (2.5)$$

where α_ψ and m_ψ are empirical constants which can be obtained by conducting a few triaxial tests.

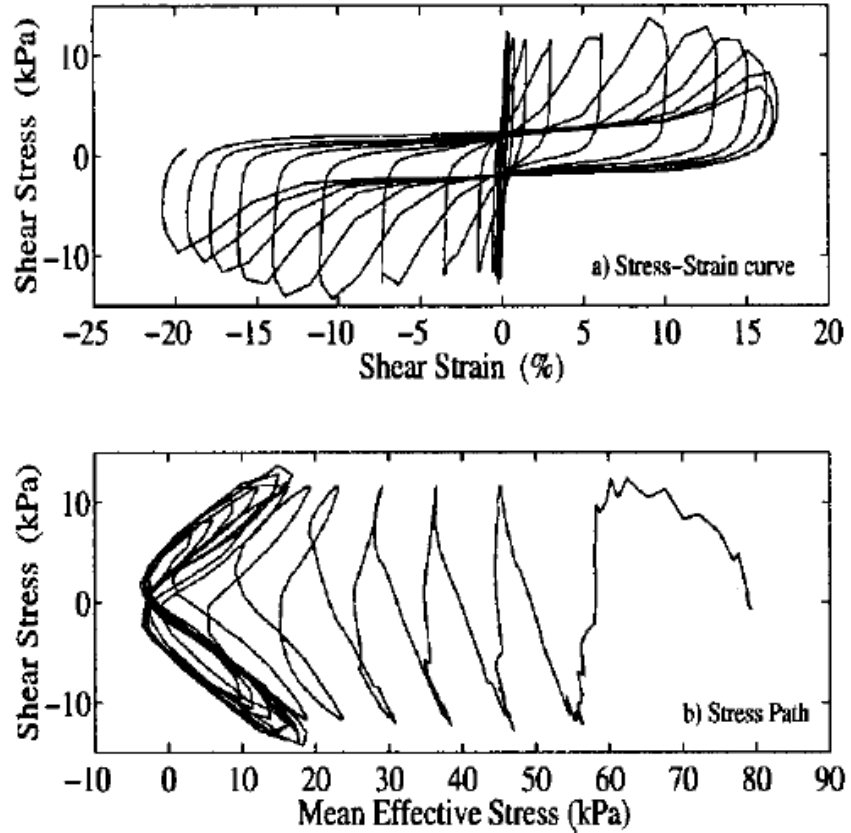


Figure 2.6. Stress-Strain and Stress-Path Response for Nevada Sand in a Stress-Controlled, Undrained Cyclic Simple Shear Test (Arulmoli *et al.*, 1992).

Based on Figure 2.6, stress-controlled undrained cyclic simple shear response of sand under investigation suggests that, with increasing number of cycles, soil loses its initial shear stiffness gradually with increasingly larger shear strain values (cyclic mobility). Rapid development of cyclic shear strain is observed at low shear stress and mean effective stress values. However, these high cyclic shear strain values are accompanied with increase in mean effective stress, which also means a decrease in pore water pressure. Reason behind this behavior is the dilation of sand grains, that results in an increase in shear stiffness and strength (Arulmoli *et al.*, 1992).

According to Elgamal *et al.* (1998), cyclic shear strain behavior of clean sands ($D_r > 35\%$, which generally is the case for liquefiable soils in practice), is dominantly affected by soil dilation during liquefaction. Moreover, residual strength (before dilation and its regain effect in shear stiffness and strength) found to be of minor importance for clear sands. On the other hand, for silty and clayey sands, residual stress may be a more important element. In the state of cyclic mobility (r_u approaching to 1), extensive and more destructive cyclic lateral deformation due to high values of cyclic shear strain can be restricted by the help of regain in shear strength due to dilatancy.

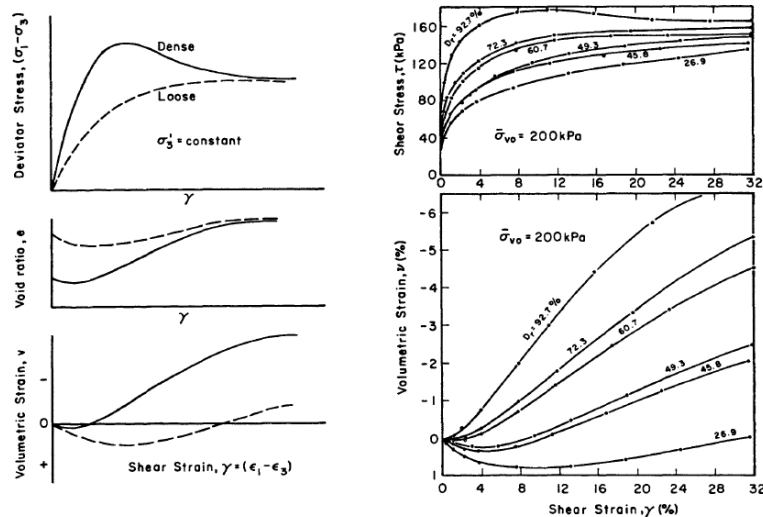
Based on consolidated drained cyclic triaxial test results on Fulung sand (which has a 5% clay content), (Ueng 1986) found that the dilatancy rate of a sand, could be used satisfactorily as a parameter for evaluating the liquefaction potential of the sand and came up with a relationship between dilatancy rate and liquefaction resistance of sand under investigation. Moreover, other factors affecting the liquefaction potential, which are relative density, confining pressure, overconsolidation, period of sustained loading and fines content were also investigated.

Figure 2.7 shows the behavior of Ottawa sand subjected to drained triaxial compression with constant confining pressure (left) and drained simple shear (right). For triaxial compression, initially both loose and dense samples go through volume expansion (dilation), however dense sand reacts with larger dilation rate to increasing shear strain, when compared with loose sand. Although both test results seems to be quite similar, an important difference is observed for dense sand's behavior. For triaxial test

result, dense sand shows a distinctive peak followed by strength reduction. However for simple shear test result, a relative flatness is observed in the shear strain versus stress curve. The reason behind this, can be explained by the fact that; the stress ratio with which sand deforms is constant under simple shear conditions and consecutively dilation angle also remains constant for a considerable strain range (Vaid *et al.*, 1981).

Vaid *et al.* (1981) showed that sand's volume expansion rate or dilation angle are uniquely related with confining pressure and sand's relative density. Additionally liquefaction resistance can be deduced to SPT blow count, dilation angle value and relative density. It is also stated that cyclic test results on undisturbed specimens and field observations show a reasonable agreement for this approach.

Elgamal *et al.* (1998) states that water migration through the additional pore space created by the rearrangement of soil grains is the main cause of dilatant behavior of saturated sands. Relatively fast rate of loading or low soil permeability will lead to a restraint of water migration, resulting in a sharp reduction in pore water pressures, consequently a jump in effective confinement and possibly a sudden decrease in shear strain values.



2.4. Influence of Particle Shape and Size on Dilatancy and Liquefaction Susceptibility

Before the development of image capturing and processing, standard two dimensional charts were used for morphology measurements. Particles from samples were compared to the standardized shapes in these charts. As an example, Powers' Qualitative Sphericity - Roundness chart can be found is given as Figure (Powers, 1953).

Grain size distribution has a considerable effect on soil cyclic liquefaction resistance. For low relative densities (D_r less than 45%), poorly graded water-deposited sand leads to lower cyclic strength when compared to well-graded sands at the same relative density. On the contrary, for samples with higher relative densities, well gradation may result in reduced cyclic resistance. In this study, for all the sand types mean grain size (D_{50}) is the same (Vaid *et al.*, 1990).

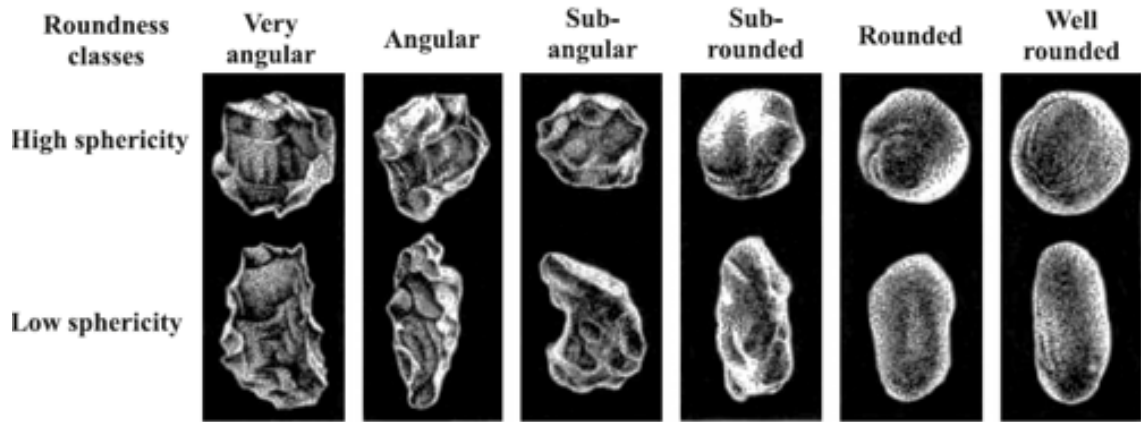


Figure 2.8. Powers' (1953) Qualitative Sphericity-Roundness chart.

Tsomokos and Georgiannou, (2010) investigated undrained response, four fine to medium sands ($D_{50} = 0.15- 0.29$ mm) which have similar or identical gradation curves but different grain angularity. Based on torsional hollow cylinder test results, undrained behavior of rounded sands goes through a systematic weakening, however for angular sands, a stable response with continuous increase in strength is observed after the peak.

Using three different sizes of silica sands with digital microscopy, (Alshibli and Alsaleh, 2004) conducted a detailed analysis of the surface roughness, roundness, and sphericity. Increase of friction and dilation angles with increasing surface roughness was observed.

Based on the study of Vaid *et al.* (1985), under 200 kPa confining pressure, for the entire range of relative densities, cyclic simple shear resistance of angular tailings sand is greater than those of rounded tailings sand. On the contrary, for higher confining pressures angular sand resistance can be greater or smaller depending on the relative density range, when compared to that of rounded sand. Under the highest confining pressure (2500 kPa), angular sand is much less resistant even at relative densities near 100%. Rounded sand, however is unlikely to liquefy for relative densities 80-85% under similar confining pressures.

According to Cho *et al.* (2006), increased particle irregularity results in an increase in e_{max} , e_{min} and $I_e(e_{max} - e_{min})$, increase compressibility under zero-lateral strain loading (C_c), an increase in critical state friction angle φ_{cs} and intercept Γ with a weak effect on slope λ . Depending on triaxial test results on two different types of sand, (Guo and Su, 2007), determined that particle angularity has a significant influence on peak friction angle and dilatant behavior.

In order to investigate the influence of grain shape on dilatancy, (Cox, 2008) came up with Weighted Single Sand Shape Factor (WSSSF), based on weighted factor analysis of six distinctive grain shape parameters of different sands, which are Circularity, Roundness, Compactness, ModRatio, Sphericity, and Aspect Ratio. Equation shows the correlation developed between dilation and WSSSF.

$$\frac{\left(\frac{\alpha_p}{\phi'_{cs}}\right)}{D_R} = -0.117 \ln \left(\frac{\sigma'_n}{p_a} x WSSSF \right) + 0.457 \quad (2.6)$$

where D_R is relative density, φ_{cs} is critical state friction angle, α_p is peak dilation angle, σ_n is effective vertical stress and p_a is atmospheric pressure.

Cox, (2008) also compared the grain images taken by both light microscope (LM) and scanning electron microscope (SEM) using ImageJ as image processing program. It was concluded that LM was capable of producing images of sufficient quality for grain shape observing purposes. Figure 2.9 shows the comparison of values obtained from SEM (hatched bars) and LM (shaded bars) for four different sand types. It is concluded that LM is better at providing accurate relative grain shape values rather than accurate absolute grain shape values.

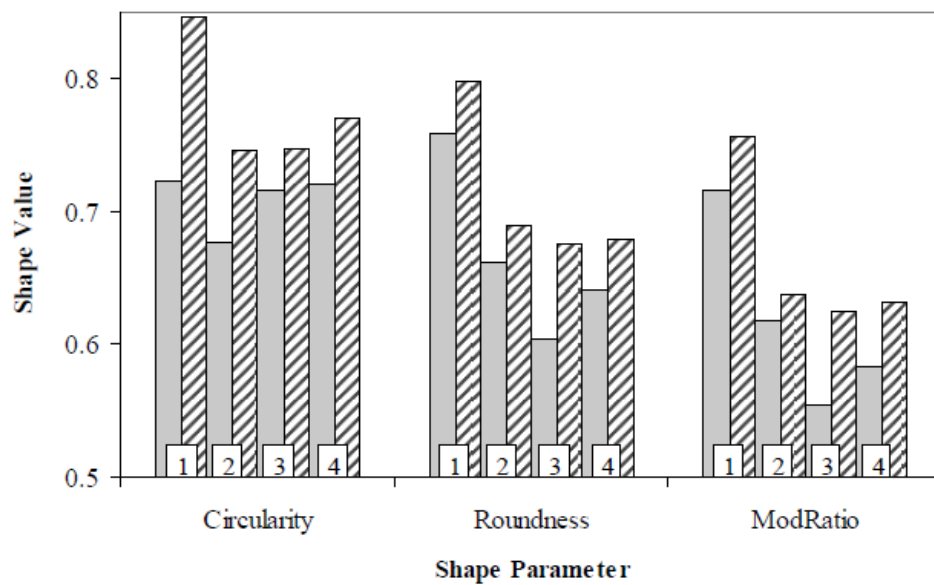


Figure 2.9. Comparison of Grain Shape Parameter by LM and SEM images (Cox, 2008).

2.5. Influence of Sample Preparation for Reconstituted Sands in Laboratory Testing

It is a common practice to use reconstituted specimens of sandy soil in geotechnical laboratories, owing to the cost limitations and difficulty in obtaining high quality undisturbed sandy soil deposits. Primary objective of sample preparation with sandy soils, is to have a good estimation of in-situ properties.

Pluviation (raining) and densification methods are widely used for sample preparation. Figure 2.10 shows different techniques of densification, such as hammer impact,

surface tamping and horizontal vibration. Generally, densification is applied layer by layer, until the specimen reaches the desirable relative density.

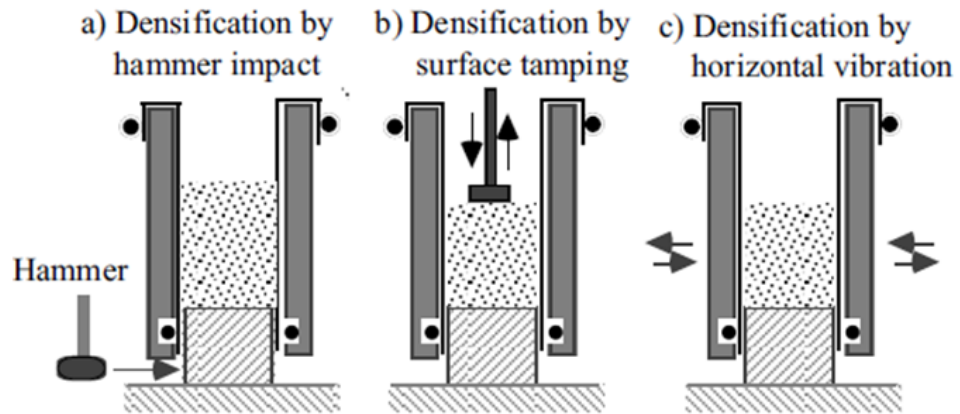


Figure 2.10. Densification Methods for Sample Preparation (Towhata, 2008).

On the other hand, in air pluviation method as shown in Figure 2.11, dry sand is allowed to fall through a funnel with a constant fall height in order maintain uniformity throughout the specimen. Denser specimens can be obtained by increasing height of the fall, which leads to an increase the potential energy of the sand grains or reducing the rate of fall to cause less energy loss due to minimized grain to grain collision. Reducing rate of fall can be achieved in two ways, reducing the rate of pouring the sand particles or the cross sectional area of the funnel's opening.

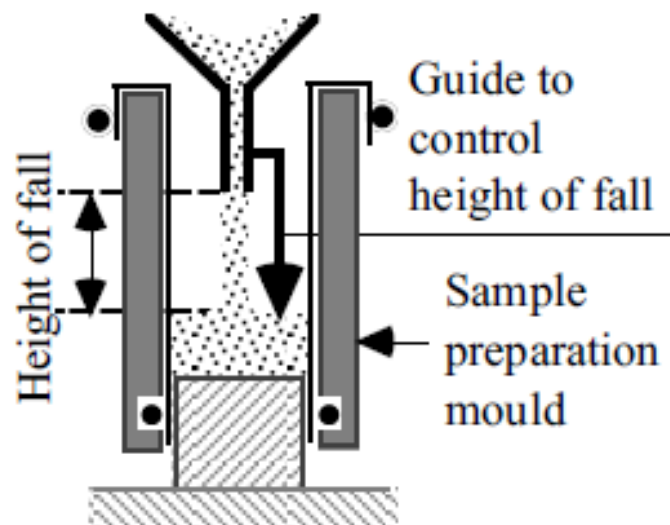


Figure 2.11. Air Pluviation Method for Sample Preparation (Towhata, 2008).

Pluviators are categorized into two by Fretti *et al.*, (1995); stationary pluviators and travelling pluviators. In the case of stationary pluviators, the soil grains exit from the hopper through holes or nozzles that are opened simultaneously. The grain diffusion over the area of interest (In Fretti *et al.*'s study, a cylindrical specimen with a 1.2 m in diameter and 1.5 m height was used) is achieved by means of sieves or meshes. On the other hand, for traveling pluviators, the soil grains exit from the hopper through an opening or nozzle that is moved over sample area. Although usage of travelling pluviator leads to more uniform specimens compared to stationary pluviators, it is not possible to reconstitute relative densities greater than 70-90% with a travelling pluviator (Fretti *et al.*, 1995).

Vaid *et al.*, (1999), compared the responses of specimens reconstituted with different sample preparation methods to undisturbed specimens and concluded that techniques in specimen-reconstituting effects the static undrained behavior of sand samples. Specimens for undrained static and cyclic simple shear were prepared by moist tamping, air pluviation, and water pluviation; while specimens for static undrained triaxial tests were prepared using water pluviation and moist tamping methods.

Figure 2.12 shows the one - dimensional compressibility of Syncrude sand specimens reconstituted by moist tamping, air pluviation and water pluviation. As it can be seen from the figure, moist-tamping method results in a very high initial void ratio, however compressibility of these moist-tamped samples are greater in comparison with samples prepared with pluviation methods. This can be explained with the potentially collapsible fabric caused by moist tamping method. It is also notable that air and water pluviation methods leads to a very similar one dimensional compressibility behavior (Vaid *et al.*, 1999).

Figure 2.13 shows the comparison of undrained simple shear behavior for two Fraser River Sand specimens having the same relative density, one air pluviated and the other water pluviated which have the same void ratio and both are under the confining stress of 100 kPa. The results show that while air pluviation leads to steady state type of liquefaction failure, water pluviated sample shows a dilative response with

no strain softening.

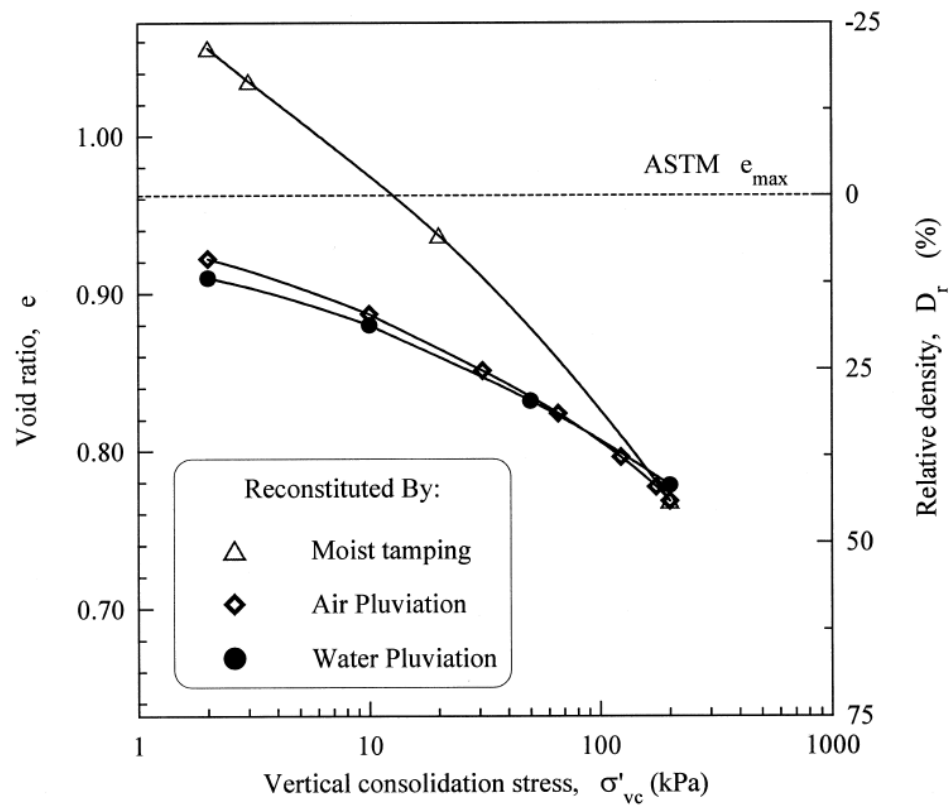


Figure 2.12. One - Dimensional Compressibility of Syncrude Sand Specimens Reconstituted by Different Methods (Vaid *et al.*, 1999).

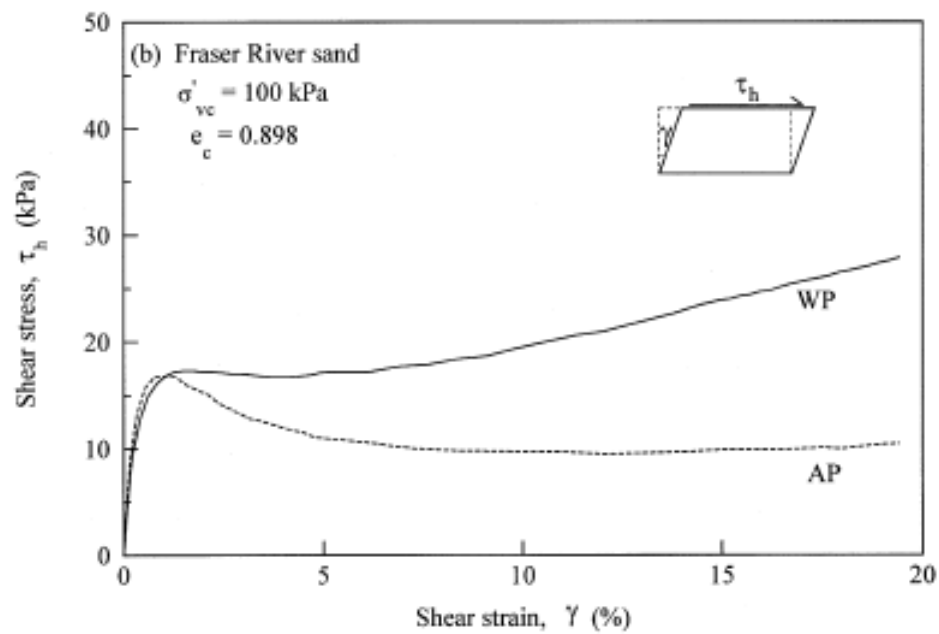


Figure 2.13. Undrained Simple Shear Response of Specimens Reconstituted by Different Techniques: Fraser River Sand (Vaid *et al.*, 1999).

Comparing undisturbed samples versus pluviated samples for both triaxial and simple shear specimens led to the conclusion that; the specimen response difference was more noticeable for triaxial testing rather than simple shear.

Rad and Tumay, (1987) state that, pluviation method leads to specimens relatively uniform and most similar to natural soil fabric, when compared to other techniques. The most preferred method by (Towhata, 2008) is air pluviation with controlled height of sand fall, because of its stable and repeatable results. Vaid *et al.* (1999) shows that, specimens reconstituted using moist tamping method will lead to unjustifiable results because of non-uniformity of these samples compared to samples prepared with water pluviation. Lo Presti *et al.*, (1992) states that achievement of higher dry densities, avoidance of particle crushing, reduced segregation and better repeatability can be counted as advantages of the pluviation method with addition to its less time consuming nature compared to vibration method.

3. METHODOLOGY

3.1. Particle Shape Determination

Particle shape determination for the sands under investigation was done using an image processing software. ImageJ is an open sourced, Java based tool developed by Rasband of Research Services Branch, National Institute of Mental Health, with the aim of analyzing human cells for the medical field. There are thousands of different plugins available to perform different tasks. In this study, circularity and roundness values of grains were obtained by using ImageJ “Shape Descriptor” plugin (Chinga, 2006). This plugin uses the following formulas to calculate circularity and roundness, where A is the area of the grain in square pixels, P is the perimeter of the outside boundary of the grain and LM is the length of the major axis measured for a grain.

$$Circularity = \frac{4\pi \times A}{P^2} \quad (3.1)$$

$$Roundness = \frac{4 \times A}{L_M^2} \quad (3.2)$$

A light microscope was used to investigate the shape characteristics of sand grains. Photos were taken using an MSHOT MD50 digital light microscope camera. In order to come up with shape parameters, binary images are needed for ImageJ to process. Accordingly, binary versions of these photos were created as shown in Figure.

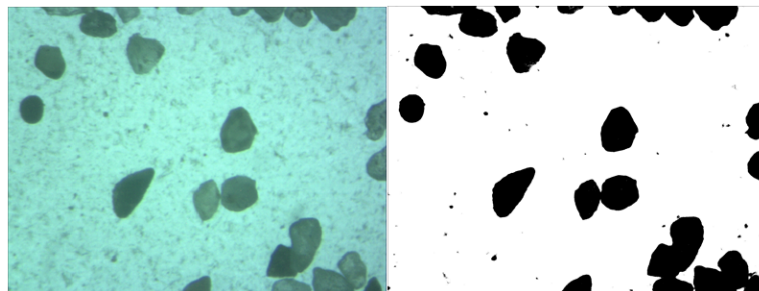


Figure 3.1. Original and Binary Version of Microscope Images.

Separation of grains were important before taking the photo, because results of overlapping grains cannot be used. Since shape parameter calculations for every particle in the image were undertaken by ImageJ, clear and separated particles were selected manually.

Ranjpour, (2015) processed images of 350 individual particles and showed that using the average circularity and roundness values of 50 particles would correspond to the minimum standard deviation for both parameters and same particle shape characterization techniques. Accordingly, 50 grains were processed for each sand type under investigation.

3.2. Testing Materials

Three different sand types were used in this research. Grain size distributions of these sands is given in Figure 3.2. Sieve analysis was done according to ASTM-D422. ASTM D-4253 and ASTM D-4254 standards were used to calculate minimum and maximum void ratios of sands.

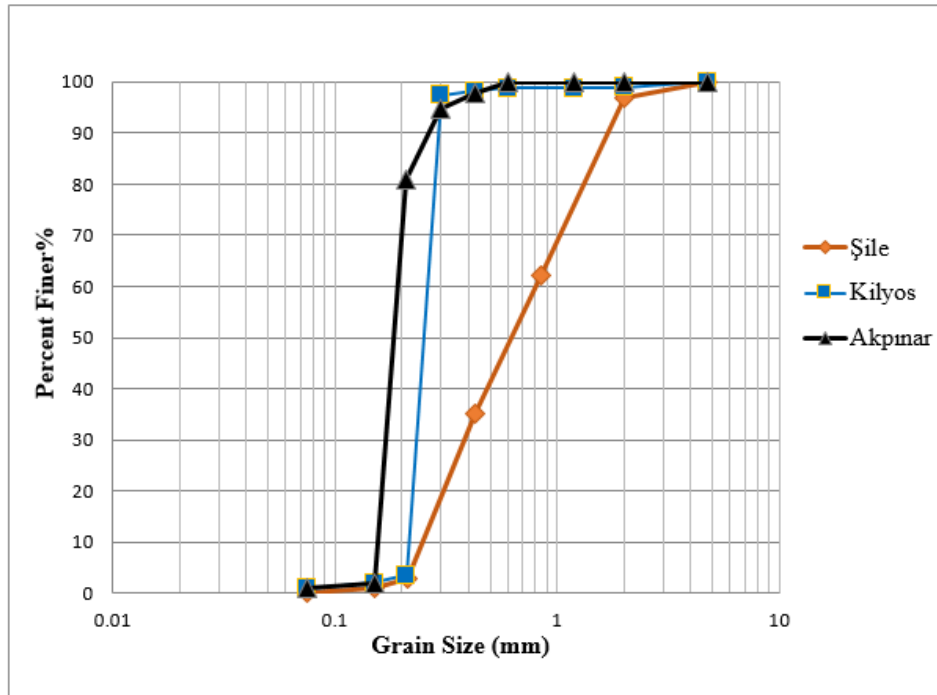


Figure 3.2. Grain Size Distribution Curves.

Table 3.1. Basic Properties of Sands under Investigation.

Sand Type	Specific Gravity	Mean Grain Size (D_{50})	Uniformity Coefficient	Curvature Coefficient	Maximum Void Ratio	Minimum Void Ratio
Kilyos	2.66	0.26	1.17	0.93	0.77	0.44
Sile	2.61	0.64	3.2	0.76	0.78	0.52
Akpınar	2.63	0.19	1.19	0.95	0.87	0.58

According to Unified Soil Classification System (USCS), if the both criteria of $C_u \geq 6$ and $1 < C_c < 3$ are met, sand can be defined as well-graded. This means all three sands are poorly graded. However, in comparison Sile is relatively more well-graded when compared to Kilyos and Akpınar sands. Since for all three sand types, uniformity coefficient is less than 4 ($C_u \leq 4$), all sands can be classified as uniform. Kilyos and Sile sands are more uniform in comparison to Sile sand.

3.3. Triaxial Testing Program

In order to go through dilatancy calculations based on (Cinicioglu and Abadkon, 2015) relationship, dilatancy constants α_ψ and m_ψ should be calculated for each sand under investigation.

$$\tan \psi_p = \alpha_\psi \left(\frac{p'_i}{p_a} \right) + m_\psi I_D \quad (3.3)$$

Geocomp fully automated system (LoadTrac II / FlowTrac II) was used for conducting triaxial tests based on the ASTM Standard of D7181 (Method for Consolidated Drained Triaxial Compression Test for Soils Tests). These CD tests were performed under K_0 condition on normally consolidated sand samples.

Automated triaxial test system goes through four phases, which are initialization, saturation, consolidation and shearing phases.

Initialization phase aims to make sure if the device is functioning properly. This step takes approximately 20 minutes, where small magnitudes of horizontal stress, vertical stress and pore pressure are applied.

Main aim of saturation phase, is to make sure that the sample is saturated before the test continues. ASTM Standard of D7181 states that B ratio, which is defined by the following equation, should have a minimum value of 0.95, before continuing to consolidation phase. Here Δu is the change of pore water pressure of the sample due to cell pressure change while specimen drainage valves are closed and $\Delta\sigma_3$ is the change in cell pressure.

$$B = \frac{\Delta u}{\Delta\sigma_3} \quad (3.4)$$

Consolidation is the phase where the sample is consolidated under K_0 zero lateral strain condition. This condition is achieved by preventing lateral strain of the sample by cell pressure adjustments.

All the samples were sheared with a constant shear strain rate of $2.5 \times 10^{-2}\%$ / min under drained conditions. This rate is calculated as given in the following equation as stated in ASTM D7181.

In order to have reliable triaxial test results, there are three important test correction to be considered, these are membrane, piston area and sample area corrections.

ASTM D-4767 standard states that membrane thickness has an influence on the radial stress applied. In order to correct the principal stress from membrane effects, equation below is suggested by ASTM.

$$\Delta(\sigma_1 - \sigma_3) = (4E_m t_m \varepsilon_1) / D_c \quad (3.5)$$

Here $\Delta(\sigma_1 - \sigma_3)$ is the correction to be subtracted from the measured deviator stress, D_c is the diameter of the sample after consolidation, E_m is the Young's Modulus for the membrane material, t_m is the membrane thickness and ε_1 is the axial strain.

Young's modulus of the membrane material (E_m) can be calculated using the equation below. In Equation, F is the force applied to stretch the membrane, L is unstretched length of the membrane, ΔL is the length change of the membrane and A_m is the area of the membrane.

$$E_m = \frac{\left(\frac{F}{A_m}\right)}{\left(\frac{\Delta L}{L}\right)} \quad (3.6)$$

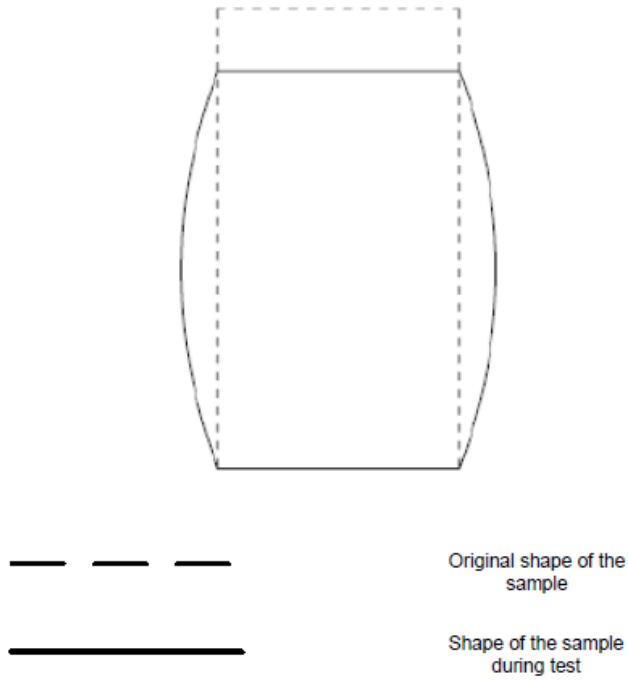


Figure 3.3. Parabolic Area Correction.

For the proper sample area correction calculation, it is important to consider the sample shape during the shearing phase. In this study, all of the tests resulted in a bulging parabolic sample Figure 3.3. All the sample area correction were undertaken

using Equation 3.7.

$$A_c = A_0 \frac{1 - \varepsilon_v}{1 - \frac{5}{3}\varepsilon_1} \quad (3.7)$$

In this equation A_c is the corrected sample area, A_0 is the initial area of the sample, ε_v is the volumetric strain during shearing and ε_1 is the axial strain. Here, ε_v is $\Delta V_s/V_c$, ε_1 is $\Delta h/h_c$ and h_c is $h_o - \Delta h_o$. ΔV_s is the volume change of the sample during the shearing phase, V_c is the volume of the sample after the consolidation phase, h is the change in height of the specimen during loading h_c is the height of sample after consolidation, h_o is the initial height of the sample and Δh_o is the change of height in the specimen at the end of the consolidation phase.

The reason of correction for piston area is the upload buoyancy force, which resists the vertical load applied by the piston. In order to measure the buoyancy force effect, filled the triaxial cell with water and applied pressure to piston.

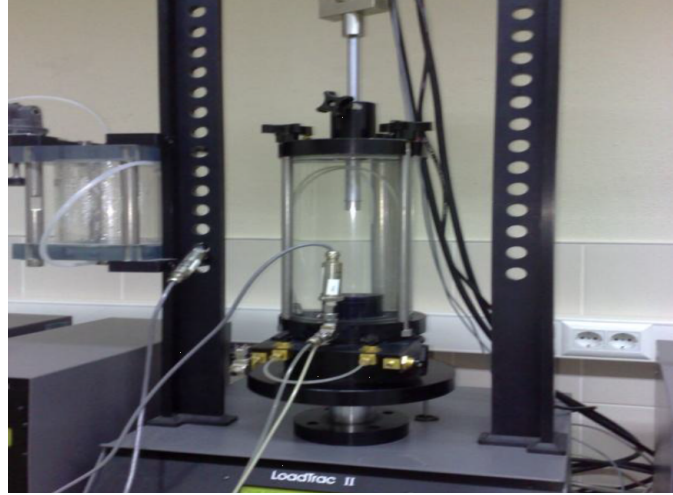


Figure 3.4. Piston area correction test (Abadkon, 2012).

According to Abadkon, (2012), the effective area of the piston was determined to be 153 mm^2 and the equation to calculate buoyancy force was given as follows.

$$F_b = 0.153\sigma_c + 6.25 \quad (3.8)$$

In this equation, F_b is the buoyancy force in Newtons and σ_c is the cell pressure in kPa. Calculated buoyancy force should be subtracted from the total load value. Calculated buoyancy force was subtracted from the total load value.

3.4. Cyclic Direct Simple Shear Testing

Cyclic direct simple testing device used for this study is a fully automated GeoComp Shear Trac II at Bogazici University Karl Terzaghi Soil Mechanics Laboratory (Figure 3.5) which is capable of going the phases of consolidation and shear automatically. Application of vertical and horizontal loads are applied via micro-stepper motors. The software of the system gives full control of test parameters and records data throughout the test.

Teflon coated stacked rings is used as confinement for 63.5 mm diameter specimens with top and bottom porous stones. Linear variable differential transformers (LVDT) are used for displacement measurements. Total load capacity of the device is 4.4 kN and displacement capacity of load pistons are 24.45 mm in vertical and ± 12.5 mm in horizontal direction. Displacement resolutions for both directions is 0.0013 mm. Figure 3.6 shows the detailed cross sectional view of the test apparatus (Kaveh Hassan Zehtab, 2010).



Figure 3.5. GeoComp Shear Trac II at Bogazici University Karl Terzaghi Soil Mechanics Laboratory.

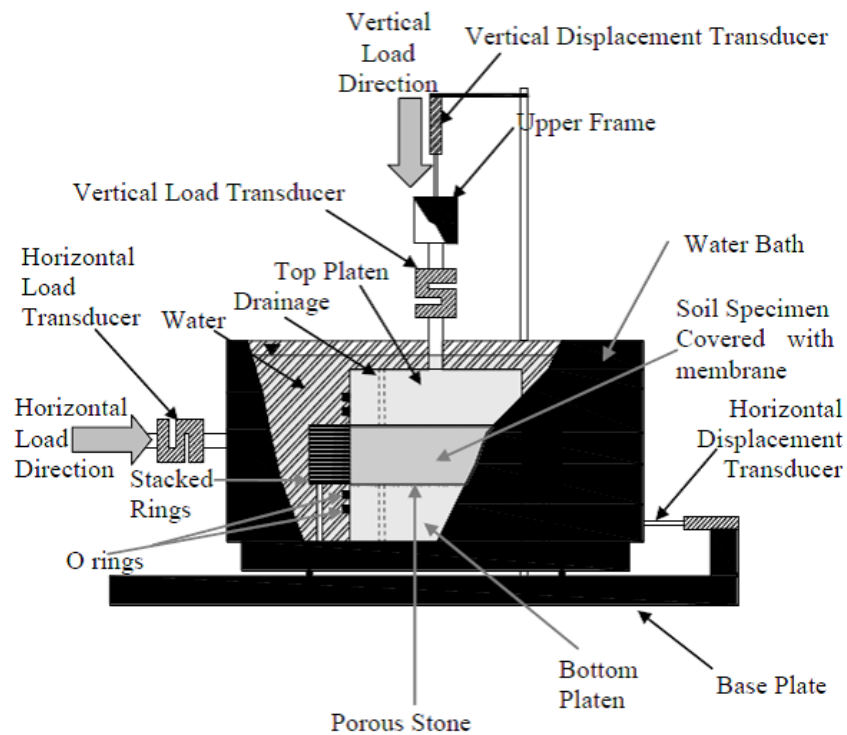


Figure 3.6. Simplified Diagram of the DSS device (Zehtab, 2010).

Every cyclic direct simple shear test has three phases, which are sample preparation and test set-up, consolidation phase and shearing phase.

3.4.1. Sample Preparation and Test Set Up

Teflon covered aluminum rings are used to wrap up the specimen, which is enclosed with a rubber membrane. A prism holder was used in order to keep these rings fixed. Air vacuum pump is used for air suction between the membrane and stacked rings. Bottom and top platens include porous stones and the two filter papers are used in order to prevent the direct contact of specimen with the porous stone. O-rings are used to fix the specimen.

Specimen heights are measured with LVDT in a two-step process. For the first measurement, an item with a known height of 28.45 mm was placed into the mold and LVDT is calibrated accordingly. And then, after sample is prepared, mold is again placed into the device and the vertical displacement reading is taken. Height of the

specimen is found by subtracting the reading from the known height (28.45 mm).

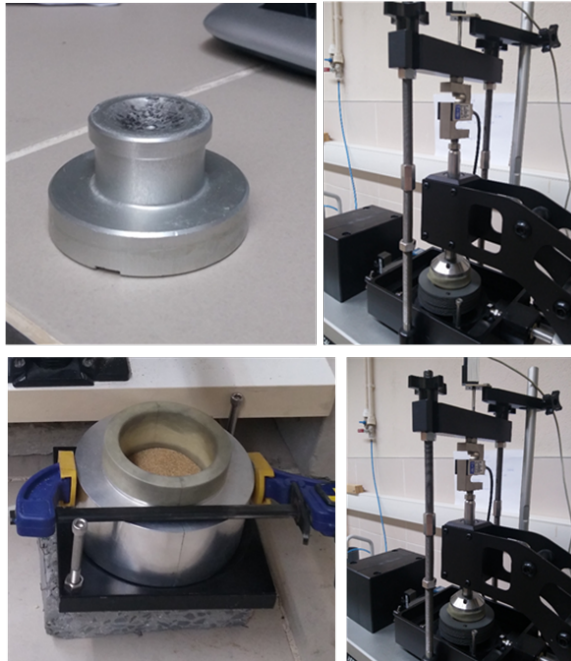


Figure 3.7. Two Step Procedure for measuring the sample height with LVDT.

All the reconstituted specimens were prepared using air pluviation method. This method is achieved by letting sand particles fall into the mold using a funnel from a predetermined height according to get the aimed relative density of the sample as shown in the figure. Looser samples are achieved by reducing the height of fall and increasing the rate of fall. On the contrary, dense samples are achieved by increasing the height of fall and reducing the rate of fall.

After the placement of the top cap, the sample is placed into the device. In order to place the rod, the sample is moved horizontally and rod is placed without disturbing the sample. Cross bar (by making sure it is horizontal) and vertical LVDT is placed and then all the bolts are tightened. After measuring the sample height via LVDT, test set up is complete and test is ready to run.

An average cyclic direct simple shear sample has 63.5 mm diameter with a height of 21 to 25 mm and weight of 120 gr.

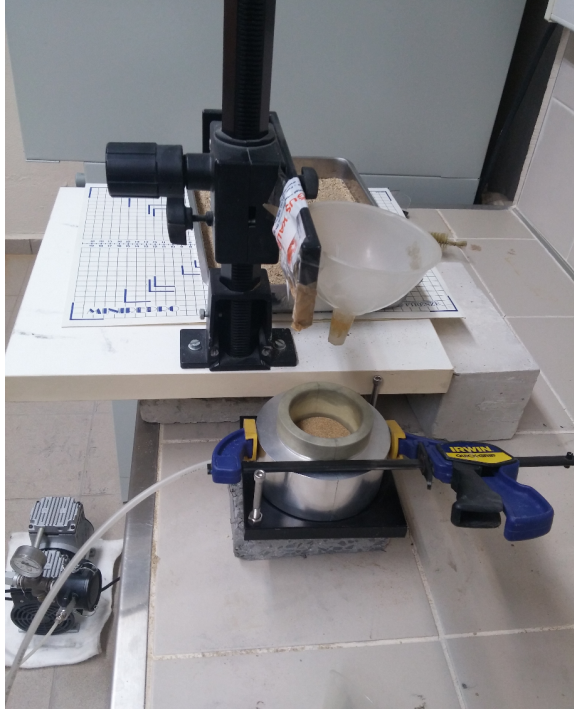


Figure 3.8. Sample Preparation Method.

Relative density (I_D) of each CSS sample was calculated as given in the Equation 3.9 where e_{min} and e_{max} are maximum and minimum void ratio of the sand, determined according to ASTM D-4253 and ASTM D-4254 standards. Additionally, e is void ratio of sample, which is calculated using the equations below, where γ_w is the unit weight of water, G_s is the sand specific gravity and γ_s is the dry unit weight of sample and where W_s is the dry weight and V is the volume of the specimen.

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad (3.9)$$

$$e = \frac{G_s \gamma_w}{\gamma_s} - 1 \quad (3.10)$$

$$\gamma_s = \frac{W_s}{V} \quad (3.11)$$

3.4.2. Consolidation Phase

Before cyclic shearing phase, all specimens were consolidated. Applied consolidation stress were chosen to be 50 kPa, 100 kPa and 200 kPa with a duration of 15 minutes. Since all the specimens are dry sands, it was observed that this time is more than enough to decrease the vertical displacement ratio to zero for all three sand types.

3.4.3. Cyclic Shearing Phase

At the end of consolidation phase, constant volume cyclic shearing phase commences and horizontal loads start to be applied to the specimen, as shown in Figure 3.9.

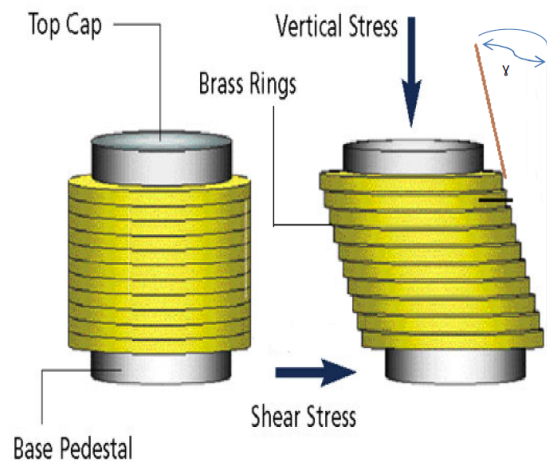


Figure 3.9. CSS Tests Vertical and Shear Stresses.

These horizontal loads are expected to be similar to a sinusoidal waveform as given in Figure 3.10. Silver (1977) states that, the condition of no spikes and insignificant ringing, which can be defined as 10% difference of amplitudes and durations between the ideal and actual waveform, is acceptable, as can be seen in Figure 3.11.

Unacceptable waveforms, which are shown in Figure 3.12, can cause the sample response to be significantly different from the actual soil response. The reason behind this, is the abnormal changes in pore water pressure, which can be observed in tests

with unacceptable waveforms.

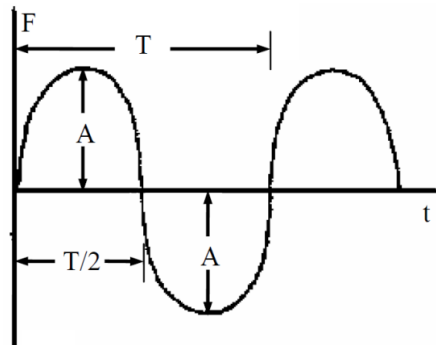


Figure 3.10. An ideal waveform (Silver, 1977).

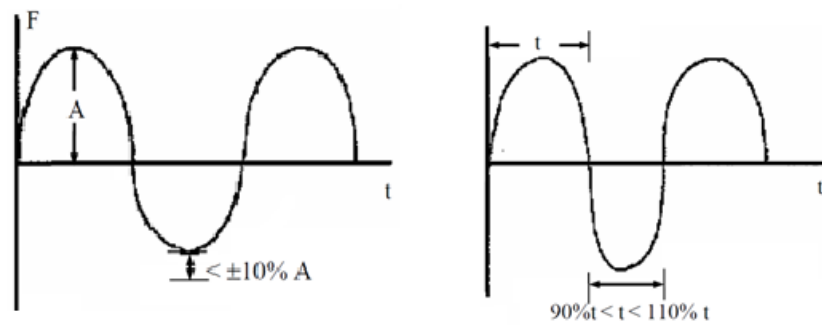


Figure 3.11. Acceptable waveforms (Silver, 1977).

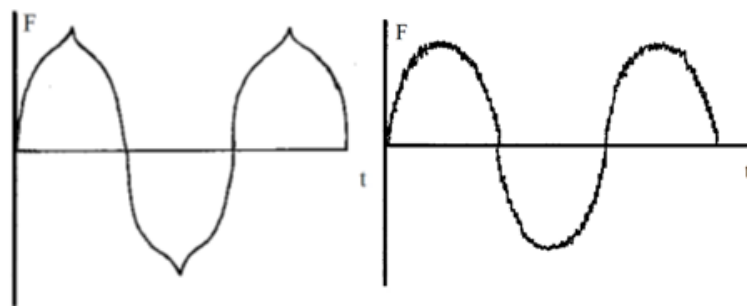


Figure 3.12. Unacceptable waveforms (Silver, 1977).

To decide the cyclic direct simple shear characteristics of the CSS testing program, literature has been reviewed.

Sriskandakumar *et al.*, (2012) conducted NGI-type cyclic direct simple shear tests using air pluviated reconstituted sand samples. Samples were consolidated to 100 kPa with a CSR of 0.1 and cyclic frequency of 0.1 Hz. It is noted in the study that, even though a period of 10 seconds is less than the period content of typical earthquakes, this value is chosen to have a better control of loading. In order to assess the point of cyclic mobility of the sample, single amplitude horizontal shear strain of 3.75% was taken into account, because this value is equivalent to reaching a 2.5% single amplitude axial strain of a triaxial specimen. This definition was also used to evaluate the cyclic resistance of sands US National Research Council (NRC, 1985).

Sadrekarami (2016) used constant volume cyclic ring shear tests to investigate cyclic shear response of Fraser River Sand. Specimens were consolidated to 100 and 200 kPa, then subjected to 0.08, 0.12, 0.15 and 0.20 with a cyclic frequency of 0.1 Hz.

Sanin and Wijewickreme (2005) used CSS test results for the evaluation of Fraser River Delta Silt cyclic shear response. Consolidation pressure for the conducted tests was 100 kPa and for the shearing phase CSR amplitudes of 0.10, 0.21 and 0.29 with 0.1 Hz cyclic frequency were applied.

El Takch *et al.* (2016) conducted ring shear tests on silt and sandy silt soils to evaluate the cyclic resistance. These tests were conducted with a consolidation pressure of 100 kPa, with different CSR values between 0.095 - 0.214 and 0.1 Hz cyclic frequency.

In order to investigate the cyclic behavior of clean, silty and clayey sands, Monkul *et al.* (2015) uses NGI-type cyclic direct simple shear test results. Applied test parameters are a CSR of 0.12, a cyclic frequency of 0.1 Hz and a consolidation pressure of 50 kPa. Tests were continued until the occurrence of $r_u = 100\%$ or 10% double amplitude shear strain.

Based on the previous studies, it is obvious that research was focused on lower cyclic frequencies. In this study, higher frequencies which are 1, 0.5 and 0.2 were used.

18 cyclic loading combinations were used for each sand type as given in Table 3.2. All the tests continued until $r_u = 1$ or a double amplitude (maximum peak to peak) strain of 20% was reached.

Table 3.2. CSS Tests, Cyclic Shearing Phase Test Combinations.

Combination	Consolidation Pressure	Cyclic Stress Ratio (CSR)	Cyclic Frequency (Hz) (CF)
1	50	0.2	1
2	50	0.2	0.5
3	50	0.2	0.2
4	50	0.3	1
5	50	0.3	0.5
6	50	0.3	0.2
7	100	0.2	1
8	100	0.2	0.5
9	100	0.2	0.2
10	100	0.3	1
11	100	0.3	0.5
12	100	0.3	0.2
13	200	0.2	1
14	200	0.2	0.5
15	200	0.2	0.2
16	200	0.3	1
17	200	0.3	0.5
18	200	0.3	0.2

4. EXPERIMENTS AND RESULTS

4.1. Shape Factor Analysis Results

As mentioned in the Methodology section, shape factors are determined using a light microscope, M-SHOT light microscope camera and an image processing software, ImageJ. Shape factor characterization is made using two factors, circularity and roundness. Circularity value helps the comparison of a sand particle with a perfect circle. In other words, as the circularity value approaches to the value of one, the particle is more and more circular. On the other hand, roundness is related with angularity. As the roundness value approaches to 1, the particle means to be less angular. Light microscope images for Akpinar, Kilyos and Sile sands can be found in Figure 4.1, Figure 4.2 and Figure 4.3.

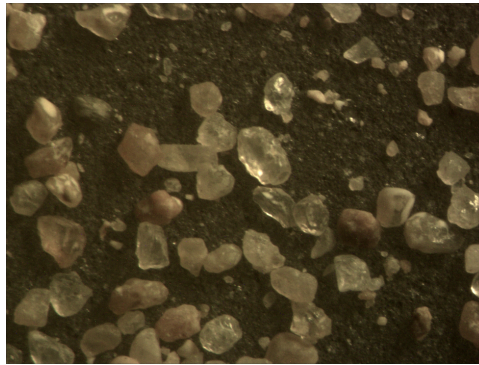


Figure 4.1. Light Microscope Image for Akpinar Sand.

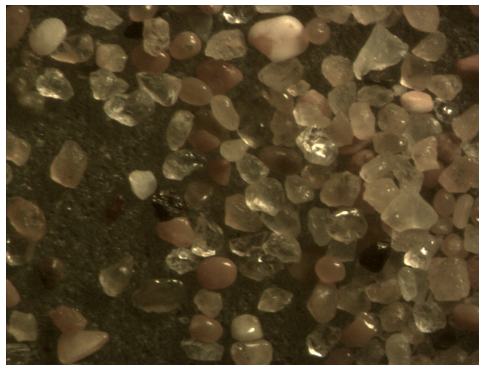


Figure 4.2. Light Microscope Image for Kilyos Sand.

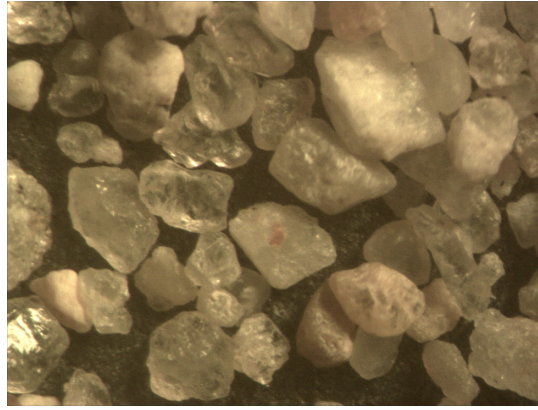


Figure 4.3. Light Microscope Image for Sile Sand.

Table 4.1. Circularity and Roundness Values for Akpınar Grains.

Particle Number	Circularity	Roundness	Particle Number	Circularity	Roundness
1	0.599	0.907	26	0.54	0.708
2	0.591	0.786	27	0.495	0.471
3	0.523	0.507	28	0.557	0.888
4	0.617	0.77	29	0.573	0.669
5	0.716	0.866	30	0.642	0.74
6	0.489	10.529	31	0.609	0.982
7	0.574	0.712	32	0.605	0.532
8	0.501	0.712	33	0.628	0.719
9	0.53	0.768	34	0.582	0.902
10	0.589	0.819	35	0.612	0.857
11	0.571	0.759	36	0.554	0.774
12	0.517	0.834	37	0.584	0.594
13	0.555	0.672	38	0.567	0.722
14	0.588	0.67	39	0.548	0.823
15	0.647	0.632	40	0.505	0.469
16	0.613	0.768	41	0.559	0.527
17	0.556	0.724	42	0.538	0.497
18	0.612	0.679	43	0.569	0.725
19	0.618	0.789	44	0.576	0.864
20	0.577	0.771	45	0.541	0.796
21	0.633	0.753	46	0.612	0.769
22	0.57	0.694	47	0.599	0.742
23	0.549	0.805	48	0.612	0.666
24	0.58	0.701	49	0.57	0.741
25	0.651	0.89	50	0.637	0.723

50 grains for each sand type were taken into account to come up with the average particle shape values, which is presented in Table 4.1, Table 4.2 and Table 4.3. According to Table 4.4, it can be deduced that Kilyos is the most circular and Akpınar is the most angular of all three sands.

Table 4.2. Circularity and Roundness Values for Kilyos Grains.

Particle Number	Circularity	Roundness
1	0.689	0.758
2	0.611	0.754
3	0.549	0.738
4	0.616	0.77
5	0.629	0.962
6	0.607	0.842
7	0.617	0.843
8	0.649	0.823
9	0.608	0.798
10	0.688	0.683
11	0.589	0.641
12	0.682	0.893
13	0.663	0.979
14	0.592	0.619
15	0.639	0.606
16	0.63	0.725
17	0.651	0.829
18	0.667	0.976
19	0.622	0.791
20	0.599	0.908
21	0.617	0.837
22	0.562	0.626
23	0.541	0.888
24	0.506	0.874
25	0.591	0.936

Particle Number	Circularity	Roundness
26	0.698	0.86
27	0.641	0.91
28	0.6	0.905
29	0.626	0.837
30	0.565	0.748
31	0.51	0.549
32	0.622	0.649
33	0.51	0.568
34	0.701	0.745
35	0.637	0.583
36	0.574	0.76
37	0.635	0.81
38	0.589	0.662
39	0.593	0.665
40	0.526	0.718
41	0.599	0.89
42	0.574	0.866
43	0.601	0.572
44	0.698	0.852
45	0.645	0.733
46	0.651	0.693
47	0.626	0.763
48	0.607	0.836
49	0.645	0.733
50	0.526	0.718

Table 4.3. Circularity and Roundness Values for Sile Grains.

Particle Number	Circularity	Roundness
1	0.567	0.66
2	0.638	0.717
3	0.577	0.633
4	0.574	0.907
5	0.588	0.848
6	0.485	0.75
7	0.589	0.742
8	0.644	0.794
9	0.546	0.926
10	0.579	0.812
11	0.487	0.674
12	0.544	0.907
13	0.562	0.914
14	0.64	0.759
15	0.59	0.767
16	0.655	0.97
17	0.521	0.741
18	0.535	0.53
19	0.621	0.711
20	0.574	0.859
21	0.59	0.786
22	0.513	0.965
23	0.586	0.864
24	0.574	0.864
25	0.57	0.912

Particle Number	Circularity	Roundness
26	0.544	0.536
27	0.533	0.636
28	0.574	0.718
29	0.532	0.654
30	0.542	0.547
31	0.553	0.647
32	0.553	0.869
33	0.607	0.618
34	0.57	0.846
35	0.601	0.91
36	0.589	0.952
37	0.559	0.896
38	0.397	0.468
39	0.498	0.656
40	0.551	0.743
41	0.501	0.894
42	0.501	0.734
43	0.437	0.593
44	0.541	0.671
45	0.593	0.893
46	0.66	0.756
47	0.552	0.622
48	0.56	0.733
49	0.606	0.783
50	0.589	0.754

Table 4.4. Average Circularity and Roundness Values for all Three Sand Types under Investigation.

Sand Type	Circularity	Roundness
Kilyos	0.612	0.775
Sile	0.562	0.763
Akpinar	0.580	0.728

4.2. Triaxial Test Results

In order to calculate the peak dilation angle according to Cinicioglu and Abadkon equation, the soil specific constants which are α_ψ and m_ψ , should be determined for each sand type.

$$\tan \psi_p = \alpha_\psi \left(\frac{p'_i}{p_a} \right) + m_\psi I_D \quad (4.1)$$

Consolidated Drained (CD) triaxial tests were conducted to determine these constants. Evaluation of triaxial test data was undertaken using Schanz and Vermeer equation, which allows the computation of dilation angle at any instant of reading during the shearing phase. Peak dilation angle is assessed by determining the peak value of the dilation angle versus axial strain relationship.

$$\sin \psi = -\frac{d\varepsilon_v/d\varepsilon_1}{2 - d\varepsilon_v/d\varepsilon_1} \quad (4.2)$$

Using the same method and apparatus, Altunbas determined α_ψ and m_ψ values for Akpinar sand. Therefore, no additional triaxial tests were conducted for Akpinar Sand. Dilatancy constants for Akpinar sand can be found in Table 4.5.

Table 4.5. Dilatancy Constants and Critical State Friction Angle for Akpinar Sand
(Altunbas, 2014).

Sand Type	α_ψ	m_ψ	Critical State Friction Angle (ϕ_c')
Akpinar	-0.0662	0.6490	33°

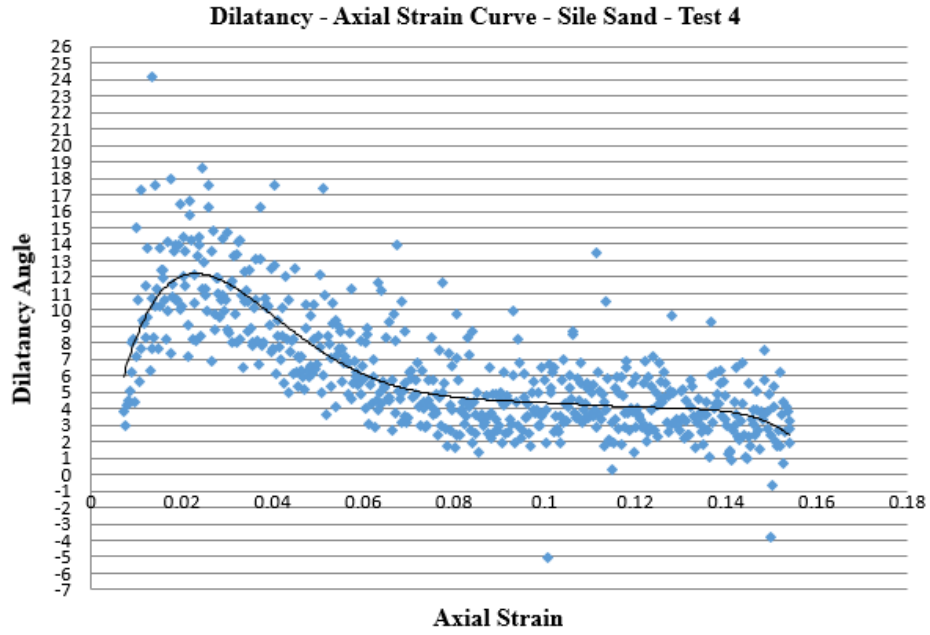


Figure 4.4. Typical Dilatancy Angle - Axial Strain Relationship based on Triaxial Test Results (Sile Sand - Test 4).

4.2.1. Sile Sand Triaxial Test Results

11 CD triaxial tests using Sile Sand, were conducted under different combinations of pre-shear confinement pressure (p_i') and relative density (I_D). The characteristics and results of these tests can be found in Table 4.6.

Dilatancy constants for Kilyos Sand is given in Table 4.7. These constants were cross checked by comparing the tangent of dilatancy angle using the Cinicioglu and Abadkon equation and tangent of dilatancy angle calculated directly from the test result using Schanz and Vermeer Equation (Equation 2.3).

Table 4.6. Sile K_o Triaxial Test Results

Sample Number	σ'_v (kPa)	p'_i (kPa)	I_D	K_o Value
1	150	79.9	0.40	0.301
2	250	137.5	0.31	0.311
3	300	210.0	0.29	0.550
4	150	87.3	0.41	0.374
5	250	133.2	0.45	0.300
6	150	90.9	0.67	0.410
7	300	173.5	0.50	0.367
8	150	108.3	0.74	0.587
9	250	133.4	0.80	0.300
10	150	100.0	0.50	0.501
11	300	169.5	0.83	0.347

Table 4.7. Dilatancy Constants and Critical State Friction Angle for Sile Sand.

Sand Type	α_ψ	$m_\psi\psi$	Critical State Friction Angle (ϕ'_c)
Sile	-0.0711	0.4665	32.9°

Figure 4.5 shows that there is a linear relationship between peak friction angle and peak dilation angle values for Sile Sand, which is known to be generally accepted in the geotechnical literature.

Calculation of peak friction angle is calculated according using triaxial test data according to Coulomb based model.

$$\sin \phi_p = \frac{(\sigma'_1)_p - (\sigma'_3)_p}{(\sigma'_1)_p + (\sigma'_3)_p} \quad (4.3)$$

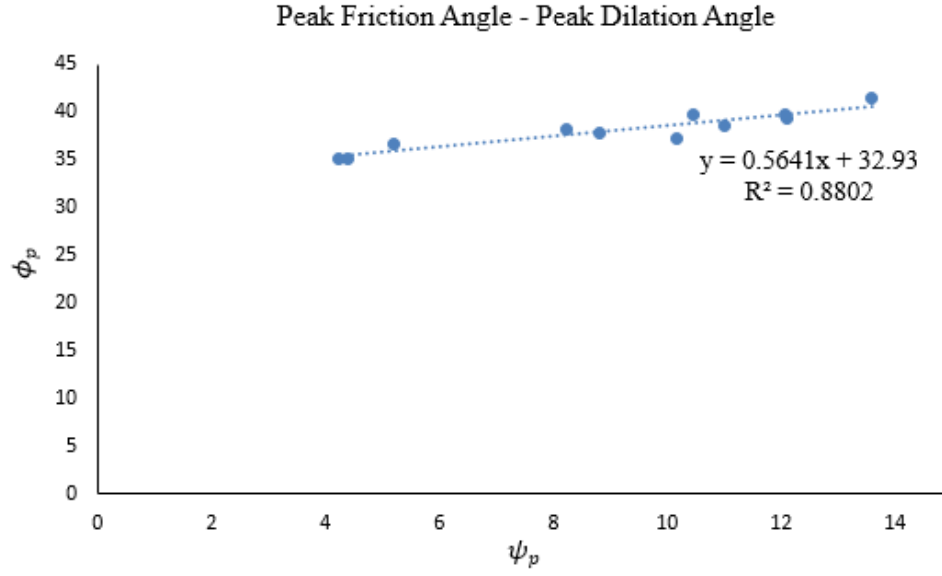


Figure 4.5. Linear relationship between peak friction angle and peak dilation angle of Sile sand.

Using the above equation, the friction angle at failure could be calculated as:

$$\phi'_p = \phi'_{cs} + r\psi_p \quad (4.4)$$

Critical state friction angle is the zero intercept of this line (Figure 4.5) and 32.9° for Sile sand.

4.2.2. Kilyos Sand Triaxial Test Results

CD triaxial tests using Kilyos Sand, were conducted under different combinations of pre-shear confinement pressure (p'_i) and relative density (I_D). The characteristics and results of these tests can be found in Table 4.8.

Dilatancy constants for Kilyos Sand is given in Table 4.9. These constants were cross checked by comparing the tangent of dilatancy angle using the Cinicioglu and Abadkon equation and tangent of dilatancy angle calculated directly from the test

result using Schanz and Vermeer Equation (Equation 2.3).

Table 4.8. Kilyos K_o Triaxial Test Results.

Sample Number	σ'_v (kPa)	p'_i (kPa)	I_D	K_o Value
1	250	133.1	0.43	0.300
2	250	133.0	0.24	0.299
3	150	111.1	0.75	0.612
4	150	119.2	0.56	0.693
5	300	168.6	0.58	0.343
6	150	115.4	0.43	0.656
7	250	133.4	0.63	0.299
8	150	104.8	0.60	0.552

Table 4.9. Kilyos Sand - Dilatancy Constants and Critical State Friction Angle.

Sand Type	α_ψ	m_ψ	Critical State Friction Angle (ϕ'_c)
Kilyos	-0.0007	0.6264	33.3°

Figure 4.6 shows that, there is a linear relationship between peak friction angle and peak dilation angle values for Kilyos Sand, which is known to be generally accepted in the geotechnical literature.

To summarize, the results for the dilatancy constants and critical state friction angles for all sands are given in Table 4.10.

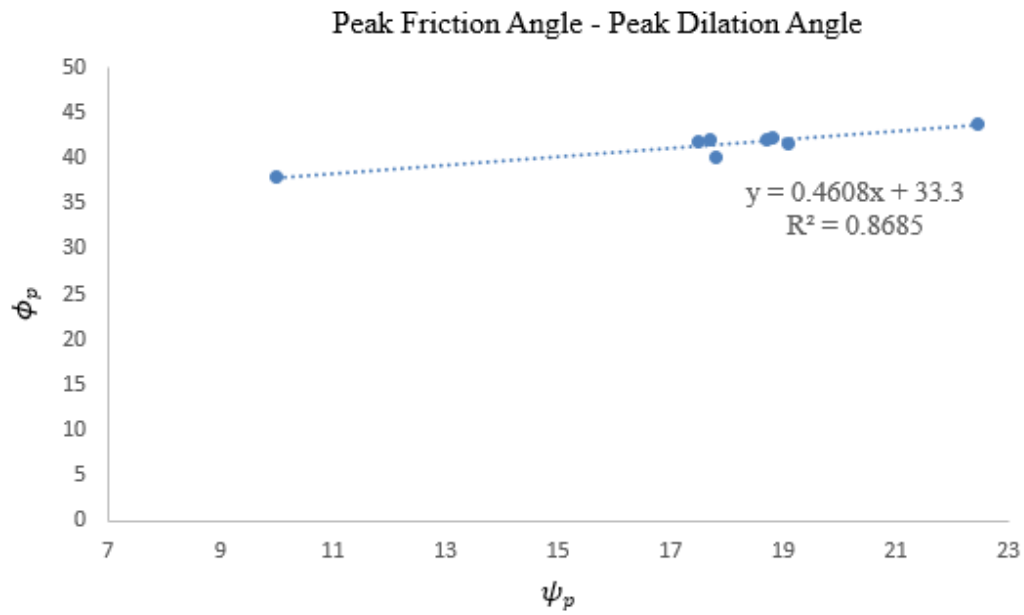


Figure 4.6. Linear relationship between peak friction angle and peak dilation angle of Kilyos sand.

Table 4.10. Dilatancy Constants and Critical State Friction Angle Values for all Three Sand Types under Investigation.

Sand Type	σ_ψ	m_ψ	Critical State Friction Angle (ϕ'_c)
Kilyos	-0.0007	0.6264	33.3°
Sile	-0.0711	0.4665	32.9°
Akpınar	-0.0662	0.6490	33°

4.3. Cyclic Simple Shear Tests

As mentioned in the Methodology section, cyclic simple shear tests were conducted on reconstituted sand samples with different relative densities under 18 main different combinations of consolidation pressure, Cyclic Stress Ratio (CSR) and Cyclic Frequency (CF).

Figure 4.7 shows a typical CSS test result and same analysis has been done for each CSS test that has been conducted in this study. Figure 4.7 shows that the Kilyos

sand sample is consolidated under 100 kPa with a CSR 0.2, leading to a 20 kPa shear stress. 49 cycles was observed until sample reached a single amplitude of shear strain of 3.75%.

All the tests continued until a double amplitude (maximum peak to peak) strain of 20% was reached. Point of cyclic mobility based on NCL values was decided to be when either r_u reached 100% or the double amplitude cyclic shear strain reached 7.5% (single amplitude shear strain of 3.75%). This value, which is also suggested by National Research Council (1985), has been generally accepted as excessive shear strain, which will lead to serviceability issues for structures. These values are also generally accepted in cyclic simple shear testing literature.

The dilatancy angles of the CSS samples were calculated using the empirical equation proposed by Cinicioglu and Abadkon (Equation 4.1), where dilatancy constants are α_ψ and m_ψ were calculated using triaxial tests, as mentioned in the previous section.

Calculation of p'_i for the conducted CSS tests, requires the knowledge of the coefficient of lateral earth pressure (K_o) value. For this purpose, variations of K_o with relative density for Kilyos and Sile sands were obtained from the results of K_o triaxial tests and relationships were defined between K_o and I_D . On the other hand, for Akpınar sand, this relationship is obtained from the Altunbas' (2014) Akpınar sand retaining wall test results on Akpınar sand.

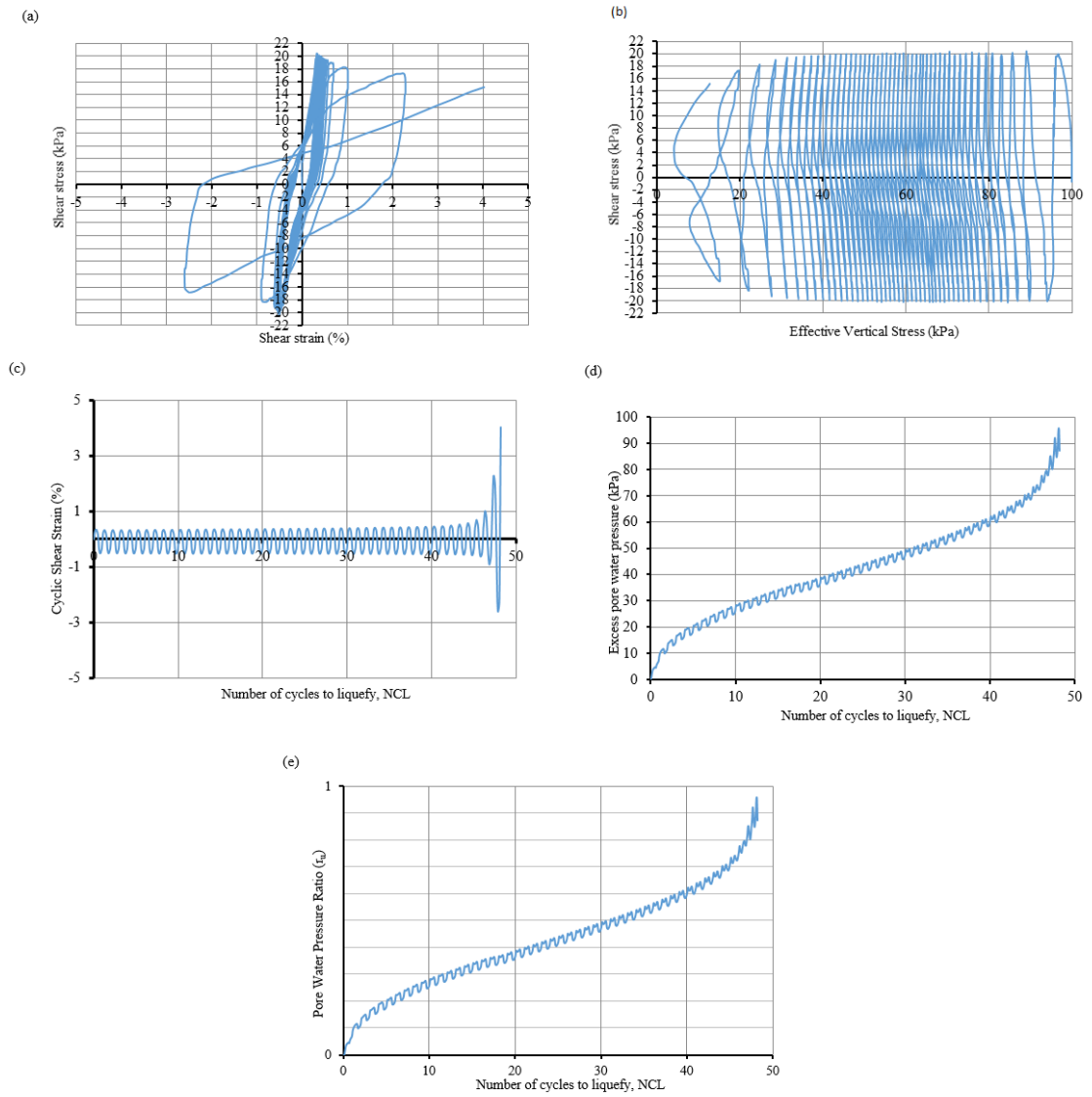


Figure 4.7. Constant Volume Cyclic Shear Response of Kilyos Sand Test 15 (100 kPa, CSR = 0.2, CF=0.2 Hz) a) Stress-Strain Response, b) Stress Path Response, c) Strain-NCL Response, d) Excess PWP-NCL Response, e) r_u -NCL Response.

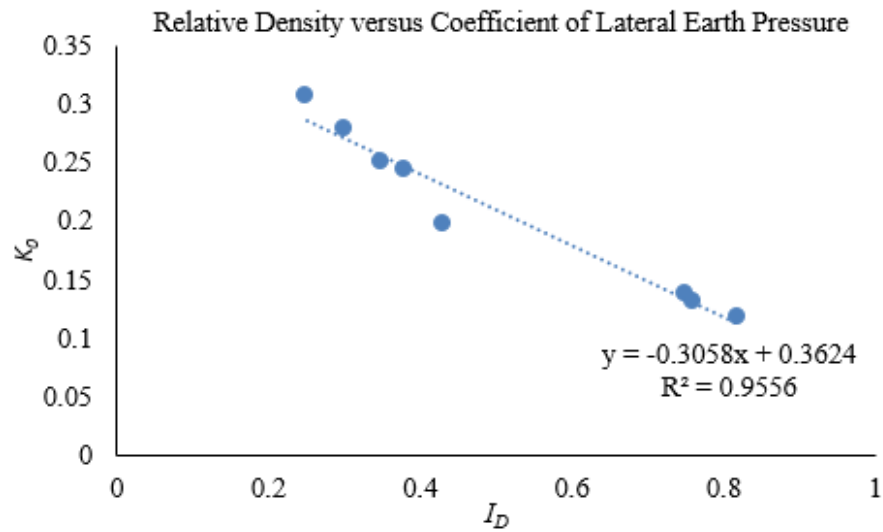


Figure 4.8. Linear relationship between relative density (I_D) and Coefficient of Lateral Earth Pressure (K_o) of Akpinar sand (Altunbas, 2014).

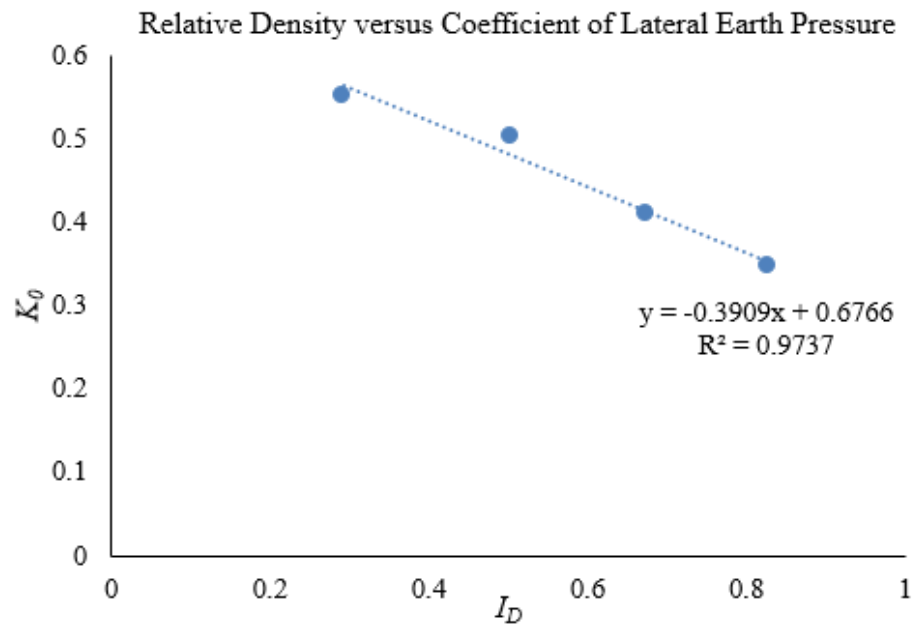


Figure 4.9. Linear relationship between relative density (I_D) and Coefficient of Lateral Earth Pressure (K_o) of Sile sand based on conducted triaxial test results.

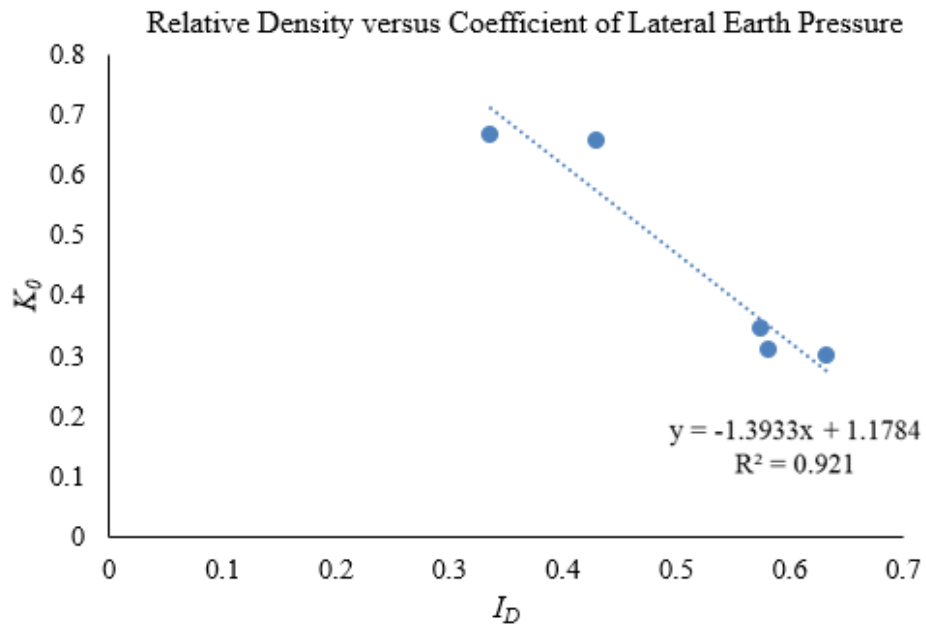


Figure 4.10. Linear relationship between relative density (I_D) and Coefficient of Lateral Earth Pressure (K_o) of Kilyos sand based on conducted triaxial test results.

Table 4.11. $I_D - K_o$ Relationships for all Three Sand Types under Investigation.

Sand Type	$I_D - K_o$ Relationship
Kilyos	$K_o = -1.393 I_D + 1.1784$
Sile	$K_o = -0.3909 I_D + 0.6766$
Akpınar	$K_o = -0.3058 I_D + 0.3624$

4.3.1. Akpınar Sand Cyclic Simple Shear Test Results

Air pluviation method was used to prepare dry sand samples for each sand under investigation. An average cyclic simple shear sample has 63.5 mm diameter with a height of 21 to 25 mm and weight of 120 gr. Relative density values differ from sample to sample, ranging between 35% up to 85%. These variations in relative density would lead to a change K_o value, and also affects the confinement pressure. As relative density and confinement pressure changes, dilation angle value would also change from sample to sample.

CSS tests were conducted on dry Akpinar sand samples with CSR (0.2 and 0.3) and CF (1, 0.5 and 0.2 Hz) combinations, under 50 kPa, 100 kPa and 200 kPa consolidation pressure. The test results are given in Table 4.12.

Table 4.12. Akpinar CSS Results for $\sigma'_v = 50$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	50	0.2	1	0.57	19.3	70
2	50	0.2	1	0.61	20.5	75
3	50	0.2	1	0.70	23.6	99
4	50	0.2	1	0.51	17.5	65
5	50	0.2	0.5	0.72	24.0	60
6	50	0.2	0.5	0.56	19.0	43
7	50	0.2	0.5	0.47	15.8	29
8	50	0.2	0.5	0.55	18.6	40
9	50	0.2	0.5	0.43	14.7	17
10	50	0.2	0.2	0.38	12.9	5
11	50	0.2	0.2	0.55	18.8	17
12	50	0.2	0.2	0.45	15.3	12
13	50	0.2	0.2	0.63	21.4	29
14	50	0.3	1	0.75	24.9	23
15	50	0.3	1	0.60	20.3	22
16	50	0.3	1	0.54	18.2	18
17	50	0.3	1	0.59	19.9	16
18	50	0.3	1	0.38	12.9	7
19	50	0.3	0.5	0.58	19.5	8
20	50	0.3	0.5	0.46	15.6	5
21	50	0.3	0.5	0.72	24	11
22	50	0.3	0.5	0.62	20.9	9
23	50	0.3	0.5	0.78	26.0	18
24	50	0.3	0.2	0.38	12.7	3
25	50	0.3	0.2	0.42	14.2	3
26	50	0.3	0.2	0.67	22.5	8
27	50	0.3	0.2	0.54	18.3	5
28	50	0.3	0.2	0.58	19.6	7
29	50	0.3	0.2	0.58	19.7	7

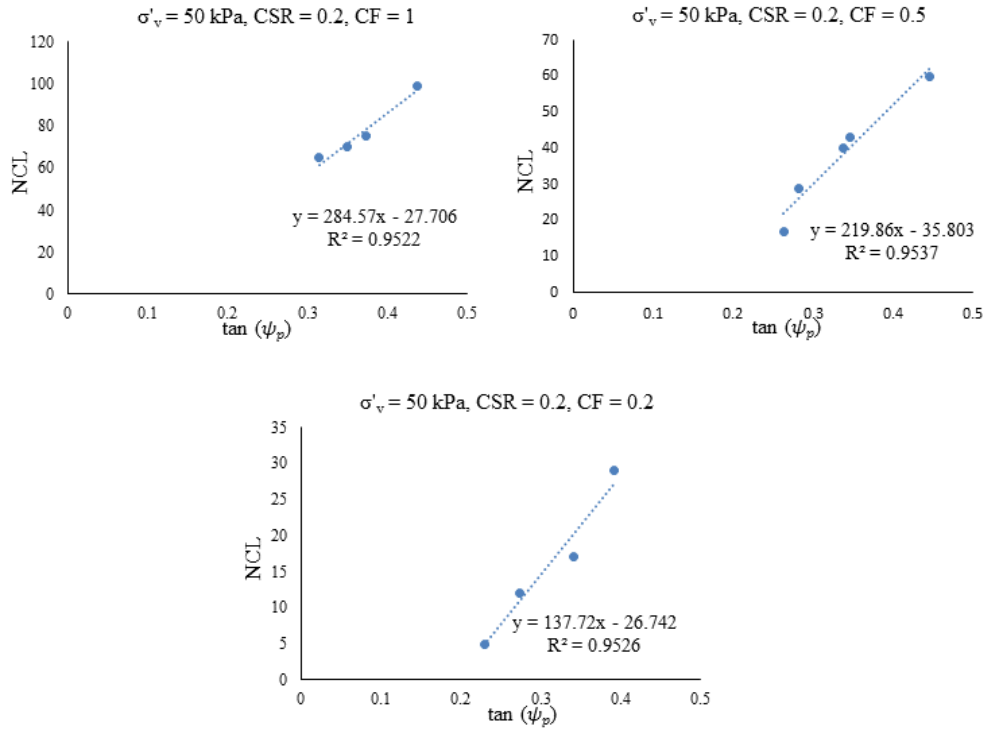


Figure 4.11. Akpinar CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.2$.

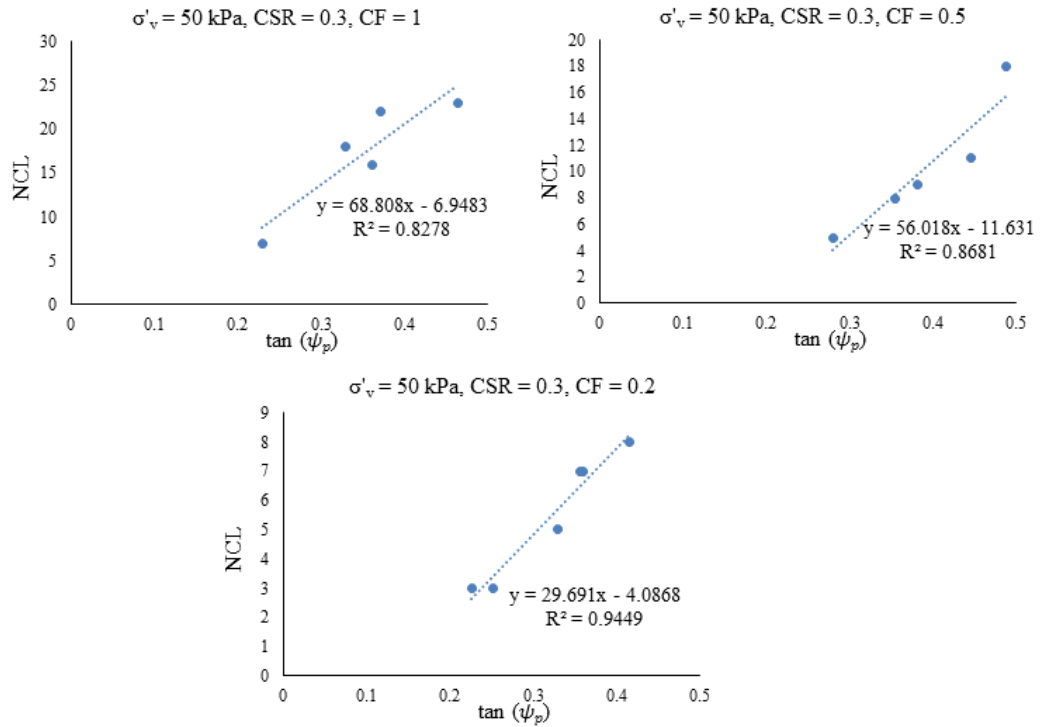


Figure 4.12. Akpinar CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.3$.

Table 4.13. Akpinar CSS Results for $\sigma'_v = 100$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	100	0.2	1	0.57	19.3	166
2	100	0.2	1	0.61	20.5	374
3	100	0.2	1	0.70	23.6	238
4	100	0.2	1	0.51	17.5	51
5	100	0.2	0.5	0.56	18.1	50
6	100	0.2	0.5	0.39	12.3	20
7	100	0.2	0.5	0.52	16.9	35
8	100	0.2	0.5	0.60	19.7	59
9	100	0.2	0.5	0.66	21.4	70
10	100	0.2	0.2	0.68	22.3	61
11	100	0.2	0.2	0.60	19.5	51
12	100	0.2	0.2	0.41	13.1	14
13	100	0.2	0.2	0.62	20.2	32
14	100	0.2	0.2	0.69	22.6	76
15	100	0.3	1	0.57	18.7	21
16	100	0.3	1	0.72	23.4	28
17	100	0.3	1	0.52	16.8	17
18	100	0.3	1	0.63	20.6	22
19	100	0.3	0.5	0.63	20.5	8
20	100	0.3	0.5	0.63	20.5	9
21	100	0.3	0.5	0.68	22.1	12
22	100	0.3	0.5	0.52	16.7	8
23	100	0.3	0.5	0.79	25.7	14
24	100	0.3	0.2	0.79	25.5	14
25	100	0.3	0.2	0.46	14.7	3
26	100	0.3	0.2	0.64	20.8	6
27	100	0.3	0.2	0.69	22.5	12
28	100	0.3	0.2	0.68	22.3	7

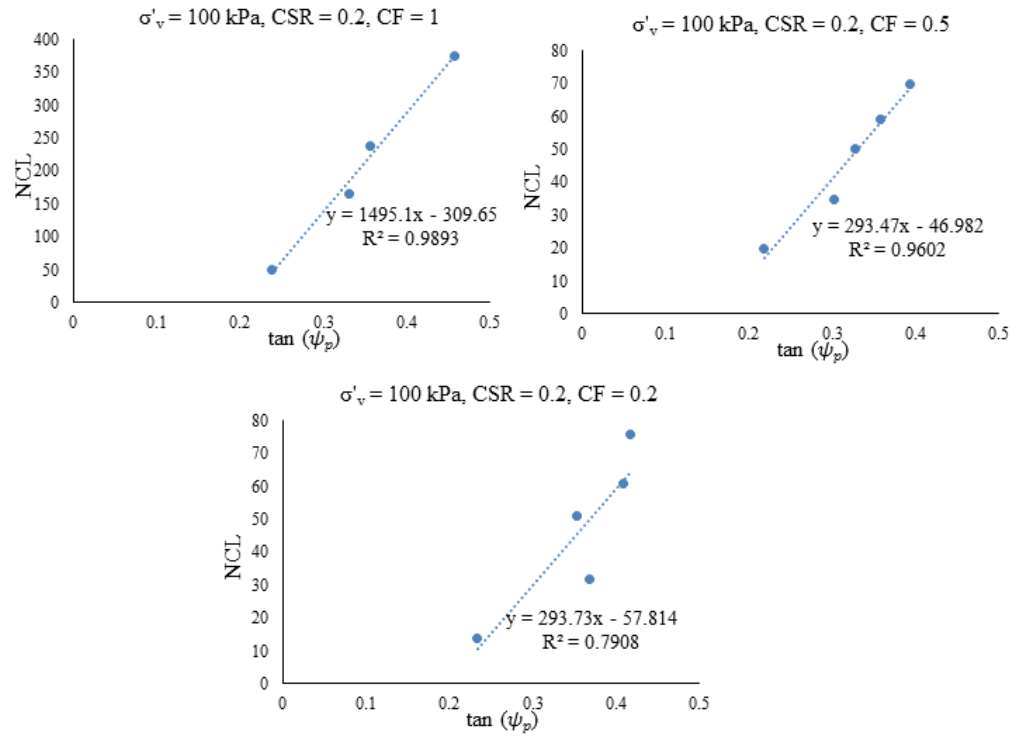


Figure 4.13. Akpinar CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.2$.

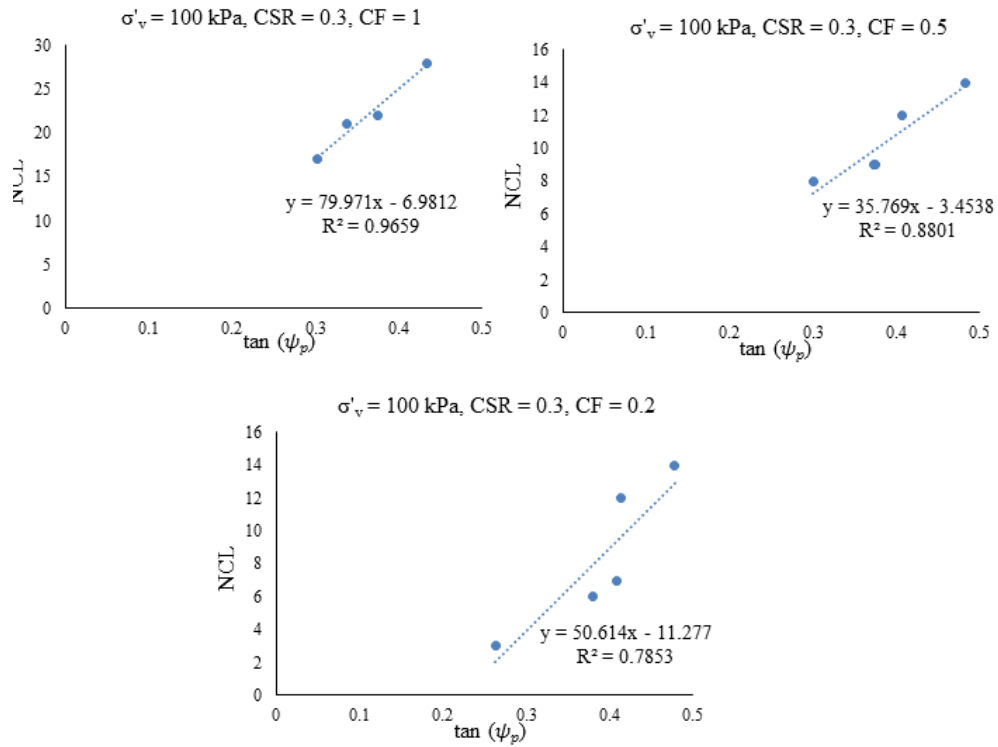


Figure 4.14. Akpinar CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.3$.

Table 4.14. Akpinar CSS Results for $\sigma'_v = 200$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	200	0.2	1	0.73	22.5	747
2	200	0.2	1	0.68	20.8	467
3	200	0.2	1	0.68	20.8	616
4	200	0.2	1	0.60	18.0	220
5	200	0.2	1	0.52	15.2	96
6	200	0.2	1	0.86	26.5	689
7	200	0.2	0.5	0.73	22.3	195
8	200	0.2	0.5	0.67	20.3	151
9	200	0.2	0.5	0.72	22.2	151
10	200	0.2	0.5	0.52	15.4	77
11	200	0.2	0.5	0.64	19.4	135
12	200	0.2	0.2	0.68	20.7	110
13	200	0.2	0.2	0.77	23.6	117
14	200	0.2	0.2	0.81	25.1	176
15	200	0.3	0.2	0.57	17.1	55
16	200	0.3	0.2	0.42	11.6	19
17	200	0.3	1	0.61	18.6	23
18	200	0.3	1	0.54	15.8	14
19	200	0.3	1	0.77	23.7	101
20	200	0.3	1	0.67	20.5	40
21	200	0.3	1	0.70	21.4	40
22	200	0.3	1	0.94	28.9	132
23	200	0.3	1	0.86	26.6	83
24	200	0.3	0.5	0.57	17.1	8
25	200	0.3	0.5	0.63	19.1	16
26	200	0.3	0.5	0.61	18.4	11
27	200	0.3	0.5	0.65	19.9	12
28	200	0.3	0.5	0.74	22.9	21
29	200	0.3	0.5	0.78	24.0	24
30	200	0.3	0.2	0.72	22.0	16
31	200	0.3	0.2	0.70	21.5	14
32	200	0.3	0.2	0.74	22.7	32
33	200	0.3	0.2	0.84	25.9	40

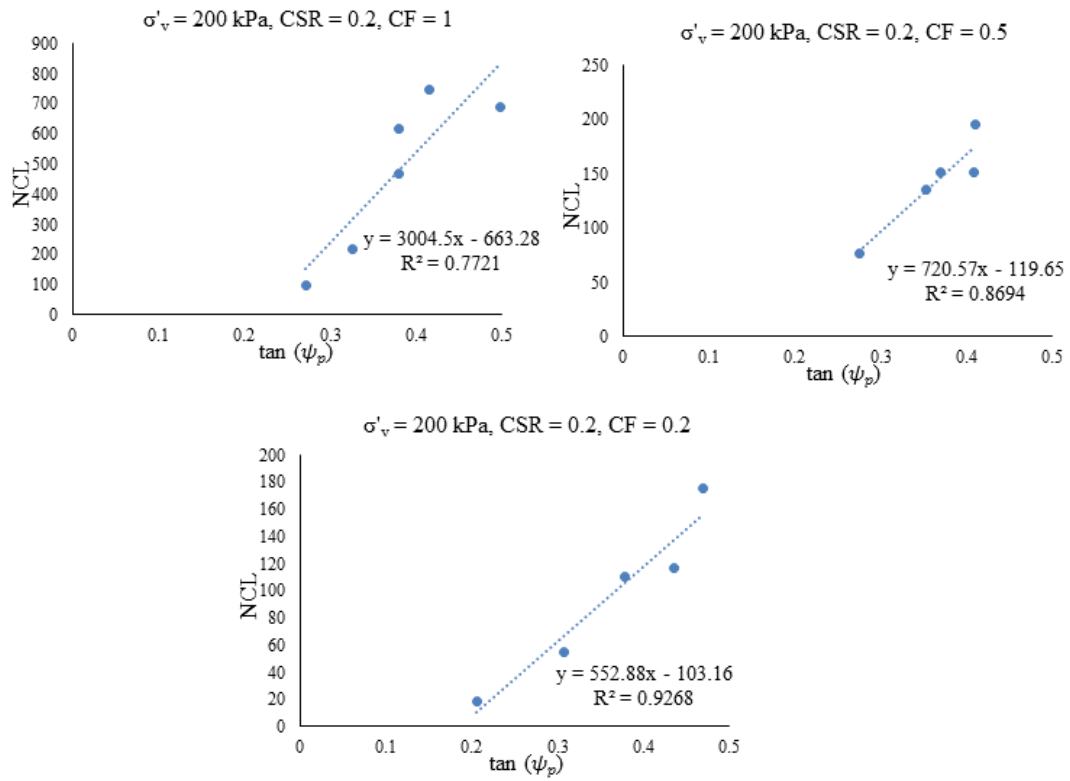


Figure 4.15. Akpinar CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.2$.

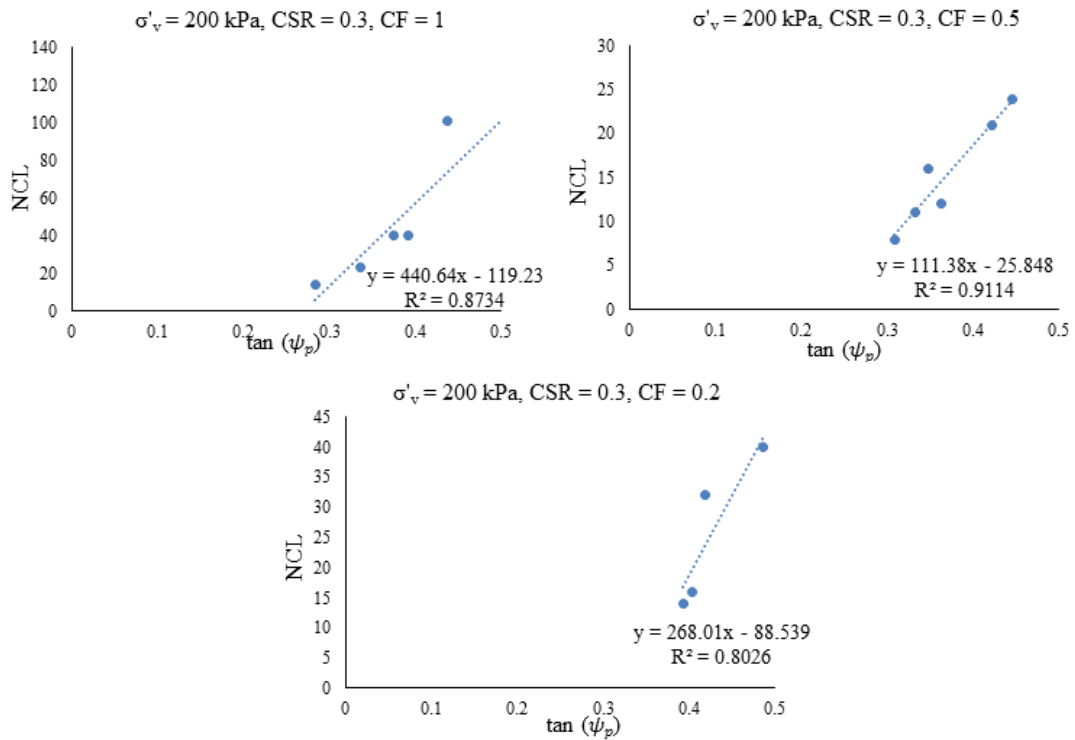


Figure 4.16. Akpinar CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.3$.

Table 4.15. Akpınar Sand $\tan\psi_p$ - NCL Linear Relationships.

Combination	Test Characteristics	Relationship
1	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 1	NCL = $284.57 \tan\psi_p - 27.706$
2	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 0.5	NCL = $219.86 \tan\psi_p - 35.803$
3	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 0.2	NCL = $137.72 \tan\psi_p - 26.742$
4	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 1	NCL = $68.808 \tan\psi_p - 6.9483$
5	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.5	NCL = $56.018 \tan\psi_p - 11.631$
6	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.2	NCL = $29.691 \tan\psi_p - 4.0868$
7	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 1	NCL = $1495.1 \tan\psi_p - 309.65$
8	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 0.5	NCL = $293.47 \tan\psi_p - 46.982$
9	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 0.2	NCL = $293.73 \tan\psi_p - 57.814$
10	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 1	NCL = $79.971 \tan\psi_p - 6.9812$
11	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 0.5	NCL = $35.769 \tan\psi_p - 3.4538$
12	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 0.2	NCL = $50.614 \tan\psi_p - 11.277$
13	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 1	NCL = $3004.5 \tan\psi_p - 663.28$
14	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 0.5	NCL = $720.57 \tan\psi_p - 119.65$
15	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 0.2	NCL = $552.88 \tan\psi_p - 103.16$
16	$\sigma'_v = 200$ kPa, CSR = 0.3, CF = 1	NCL = $440.64 \tan\psi_p - 119.23$
17	$\sigma'_v = 200$ kPa, CSR = 0.3, CF = 0.5	NCL = $111.38 \tan\psi_p - 25.848$
18	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.2	NCL = $268.01 \tan\psi_p - 88.539$

4.3.2. Sile Sand Cyclic Simple Shear Test Results

CSS tests were conducted on dry Sile sand samples with CSR (0.2 and 0.3) and CF (1, 0.5 and 0.2 Hz) combinations, under 50 kPa, 100 kPa and 200 kPa consolidation pressure. The test results are given as follows.

Table 4.16. Silo CSS Results for $\sigma'_v = 50$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	50	0.2	1	0.58	13.9	92
2	50	0.2	1	0.72	17.5	133
3	50	0.2	1	0.43	10.0	44
4	50	0.2	1	0.51	12.1	60
5	50	0.2	1	0.66	15.9	108
6	50	0.2	0.5	0.54	12.8	28
7	50	0.2	0.5	0.68	16.5	41
8	50	0.2	0.5	0.46	10.8	20
9	50	0.2	0.5	0.68	16.6	43
10	50	0.2	0.5	0.57	13.6	28
11	50	0.2	0.2	0.63	15.3	32
12	50	0.2	0.2	0.60	14.5	26
13	50	0.2	0.2	0.49	11.7	13
14	50	0.2	0.2	0.38	8.9	9
15	50	0.3	0.2	0.82	20.0	47
16	50	0.3	1	0.75	18.3	14
17	50	0.3	1	0.52	12.5	9
18	50	0.3	1	0.62	14.9	10
19	50	0.3	1	0.42	9.9	8
20	50	0.3	1	0.49	11.7	9
21	50	0.3	0.5	0.66	16.0	9
22	50	0.3	0.5	0.57	13.6	8
23	50	0.3	0.5	0.46	10.8	6
24	50	0.3	0.5	0.71	17.4	10
25	50	0.3	0.5	0.63	15.3	7
26	50	0.3	0.2	0.59	14.2	4
27	50	0.3	0.2	0.32	7.2	2
28	50	0.3	0.2	0.67	16.3	7
29	50	0.3	0.2	0.47	11.2	3

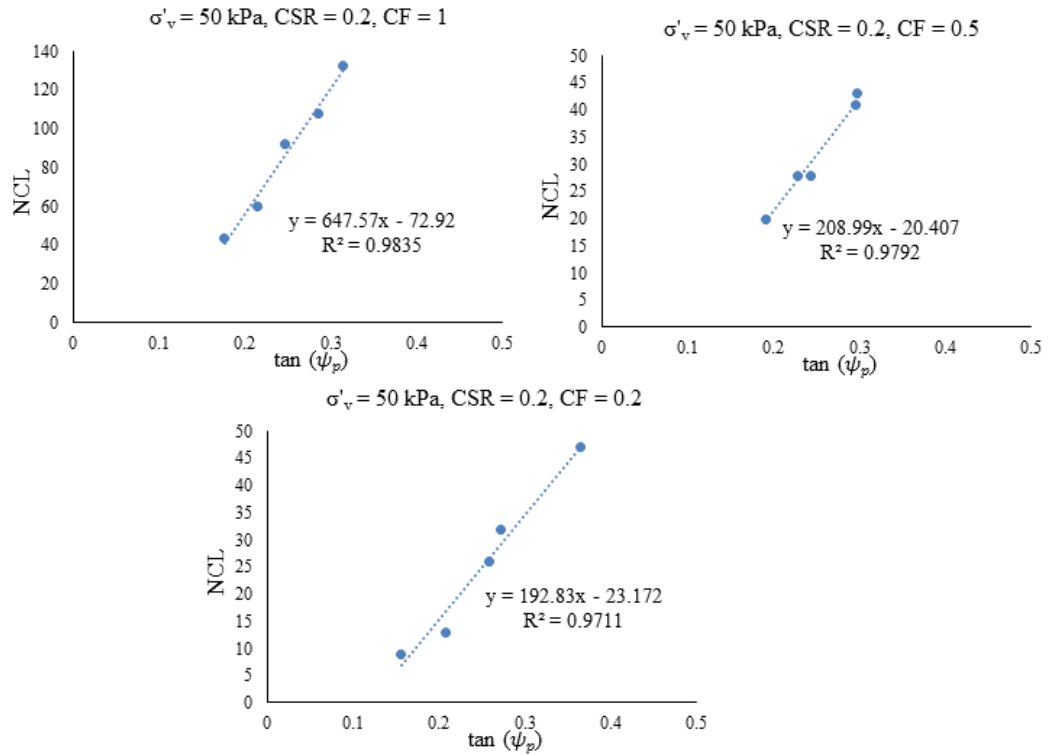


Figure 4.17. Sile CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.2$.

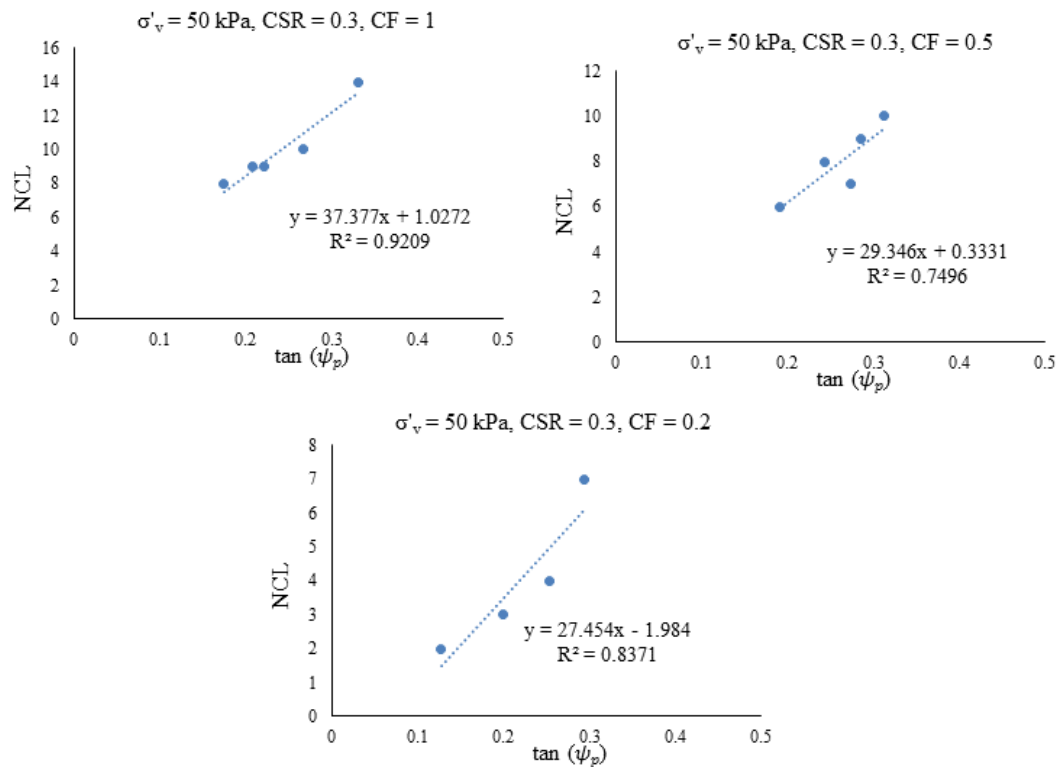


Figure 4.18. Sile CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.3$.

Table 4.17. Sile CSS Results for $\sigma'_v = 100$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	100	0.2	1	0.72	16.4	293
2	100	0.2	1	0.60	13.3	188
3	100	0.2	1	0.53	11.5	152
4	100	0.2	1	0.66	14.9	236
5	100	0.2	1	0.47	9.9	120
6	100	0.2	0.5	0.67	15.2	76
7	100	0.2	0.5	0.49	10.5	21
8	100	0.2	0.5	0.63	14.2	63
9	100	0.2	0.5	0.62	13.9	59
10	100	0.2	0.5	0.70	15.9	83
11	100	0.2	0.2	0.63	14.2	45
12	100	0.2	0.2	0.39	7.6	10
13	100	0.2	0.2	0.54	11.7	30
14	100	0.2	0.2	0.78	18	80
15	100	0.3	0.2	0.64	14.4	54
16	100	0.3	1	0.54	11.7	12
17	100	0.3	1	0.67	15.0	16
18	100	0.3	1	0.81	18.8	24
19	100	0.3	1	0.71	16.3	20
20	100	0.3	1	0.56	12.3	13
21	100	0.3	0.5	0.72	16.5	13
22	100	0.3	0.5	0.49	10.3	10
23	100	0.3	0.5	0.77	17.6	19
24	100	0.3	0.5	0.9	20.9	27
25	100	0.3	0.2	0.78	14.7	6
26	100	0.3	0.2	0.82	13	4
27	100	0.3	0.2	0.69	10.3	4
28	100	0.3	0.2	0.62	17.3	8

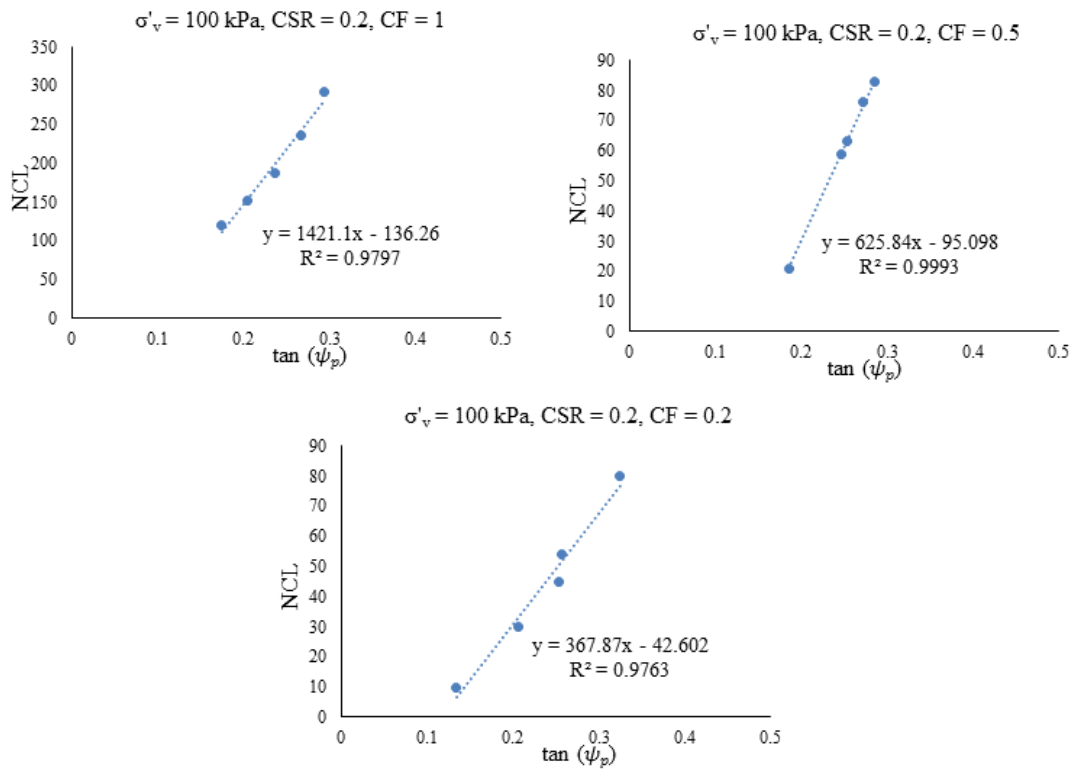


Figure 4.19. Sile CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.2$.

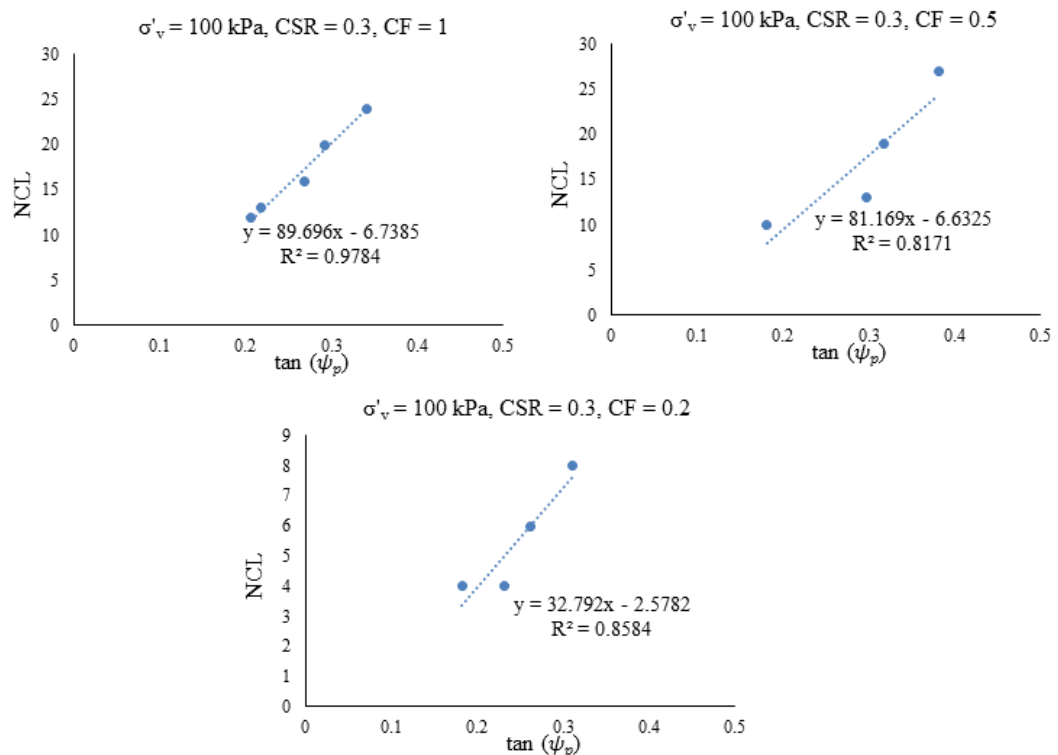


Figure 4.20. Sile CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.3$.

Table 4.18. Sile CSS Results for $\sigma'_v = 200$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	200	0.2	1	0.79	16.0	590
2	200	0.2	1	0.65	12.4	277
3	200	0.2	1	0.46	7.1	99
4	200	0.2	1	0.54	9.3	260
5	200	0.2	0.5	0.52	8.7	179
6	200	0.2	0.5	0.64	12.1	65
7	200	0.2	0.5	0.76	15.2	105
8	200	0.2	0.5	0.78	15.9	124
9	200	0.2	0.5	0.76	15.3	155
10	200	0.2	0.2	0.46	7.1	34
11	200	0.2	0.2	0.74	16.8	84
12	200	0.2	0.2	0.64	12.1	62
13	200	0.2	0.2	0.87	18.1	141
14	200	0.2	0.2	0.78	15.8	96
15	200	0.3	1	0.51	8.3	28
16	200	0.3	1	0.75	14.9	36
17	200	0.3	1	0.61	11.2	29
18	200	0.3	1	0.85	17.8	51
19	200	0.3	1	0.76	15.4	55
20	200	0.3	0.5	0.47	7.4	24
21	200	0.3	0.5	0.78	15.8	47
22	200	0.3	0.5	0.82	16.9	40
23	200	0.3	0.5	0.69	13.4	17
24	200	0.3	0.5	0.62	11.4	20
25	200	0.3	0.2	0.56	9.8	9
26	200	0.3	0.2	0.64	12.1	11
27	200	0.3	0.2	0.78	15.9	25
28	200	0.3	0.2	0.66	12.7	11
29	200	0.3	0.2	0.70	13.6	15

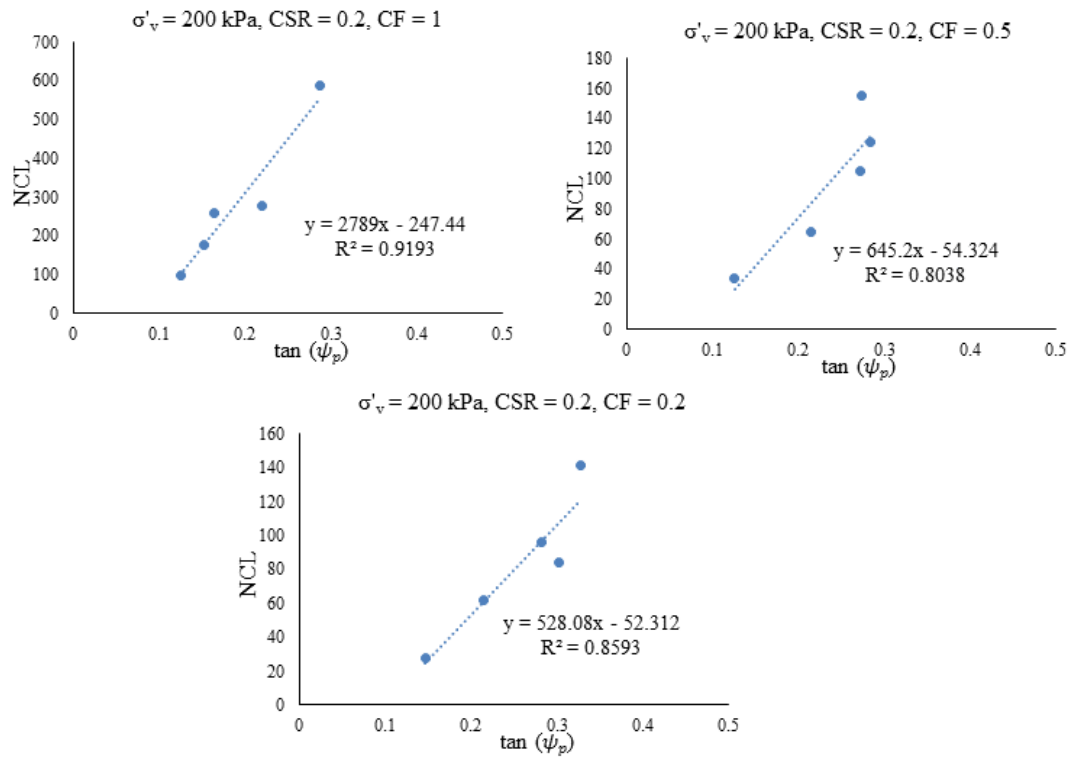


Figure 4.21. Sile CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.2$.

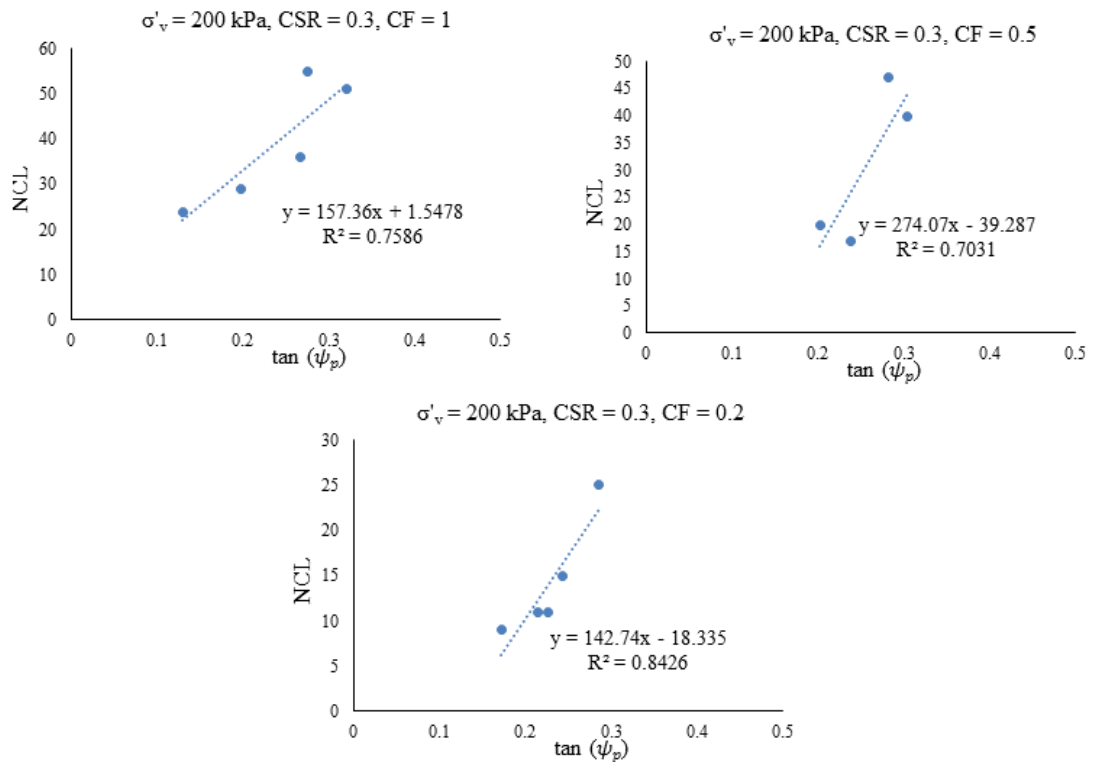


Figure 4.22. Sile CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.3$.

Table 4.19. Sil Sand $\tan\psi_p$ - NCL Linear Relationships.

Combination	Test Characteristics	Relationship
1	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 1	NCL = $647.57 \tan\psi_p - 72.92$
2	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 0.5	NCL = $208.99 \tan\psi_p - 20.407$
3	$\sigma'_v = 50$ kPa, CSR = 0.2, CF = 0.2	NCL = $192.83 \tan\psi_p - 23.172$
4	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 1	NCL = $37.377 \tan\psi_p + 1.0272$
5	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.5	NCL = $29.346 \tan\psi_p + 0.3331$
6	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.2	NCL = $27.454 \tan\psi_p - 1.984$
7	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 1	NCL = $1421.1 \tan\psi_p - 136.26$
8	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 0.5	NCL = $625.84 \tan\psi_p - 95.098$
9	$\sigma'_v = 100$ kPa, CSR = 0.2, CF = 0.2	NCL = $367.87 \tan\psi_p - 42.602$
10	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 1	NCL = $89.696 \tan\psi_p - 6.7385$
11	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 0.5	NCL = $81.169 \tan\psi_p - 6.6325$
12	$\sigma'_v = 100$ kPa, CSR = 0.3, CF = 0.2	NCL = $32.792 \tan\psi_p - 2.5782$
13	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 1	NCL = $2789 \tan\psi_p - 247.44$
14	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 0.5	NCL = $645.2 \tan\psi_p - 54.324$
15	$\sigma'_v = 200$ kPa, CSR = 0.2, CF = 0.2	NCL = $528.08 \tan\psi_p - 52.312$
16	$\sigma'_v = 200$ kPa, CSR = 0.3, CF = 1	NCL = $157.36 \tan\psi_p + 1.5478$
17	$\sigma'_v = 200$ kPa, CSR = 0.3, CF = 0.5	NCL = $274.07 \tan\psi_p - 39.287$
18	$\sigma'_v = 50$ kPa, CSR = 0.3, CF = 0.2	NCL = $142.74 \tan\psi_p - 18.335$

4.3.3. Kilyos Sand Cyclic Simple Shear Test Results

CSS tests were conducted on dry Kilyos sand samples with CSR (0.2 and 0.3) and CF (1, 0.5 and 0.2 Hz) combinations, under 50 kPa, 100 kPa and 200 kPa consolidation pressure. The test results are given as follows:

Table 4.20. Kilyos CSS Results $\sigma'_v = 50$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	50	0.2	1	0.43	15.2	154
2	50	0.2	1	0.44	15.5	236
3	50	0.2	1	0.47	16.4	454
4	50	0.2	1	0.47	16.4	365
5	50	0.2	1	0.68	23.0	809
6	50	0.2	1	0.71	23.9	1222
7	50	0.2	0.5	0.44	15.2	149
8	50	0.2	0.5	0.45	15.7	113
9	50	0.2	0.5	0.52	18.1	213
10	50	0.2	0.5	0.7	23.8	452
11	50	0.2	0.5	0.74	24.9	391
12	50	0.2	0.2	0.39	13.8	10
13	50	0.2	0.2	0.50	17.5	34
14	50	0.2	0.2	0.68	23.1	72
15	50	0.2	0.2	0.65	22.1	53
16	50	0.2	0.2	0.70	23.8	80
17	50	0.2	0.2	0.42	14.7	14
18	50	0.3	1	0.55	19.1	63
19	50	0.3	1	0.49	17.1	45
20	50	0.3	1	0.41	14.3	26
21	50	0.3	1	0.60	20.5	76
22	50	0.3	1	0.63	21.6	66
23	50	0.3	0.5	0.44	15.4	16
24	50	0.3	0.5	0.42	14.9	18
25	50	0.3	0.5	0.69	23.4	41
26	50	0.3	0.5	0.39	13.9	14
27	50	0.3	0.5	0.42	14.7	10
28	50	0.3	0.5	0.63	21.5	38
29	50	0.3	0.2	0.46	15.9	8
30	50	0.3	0.2	0.44	15.4	5
31	50	0.3	0.2	0.42	14.8	3
32	50	0.3	0.2	0.50	17.4	9
33	50	0.3	0.2	0.59	20.4	9
34	50	0.3	0.2	0.33	11.6	4
35	50	0.3	0.2	0.59	20.2	14
36	50	0.3	0.2	0.7	23.6	22

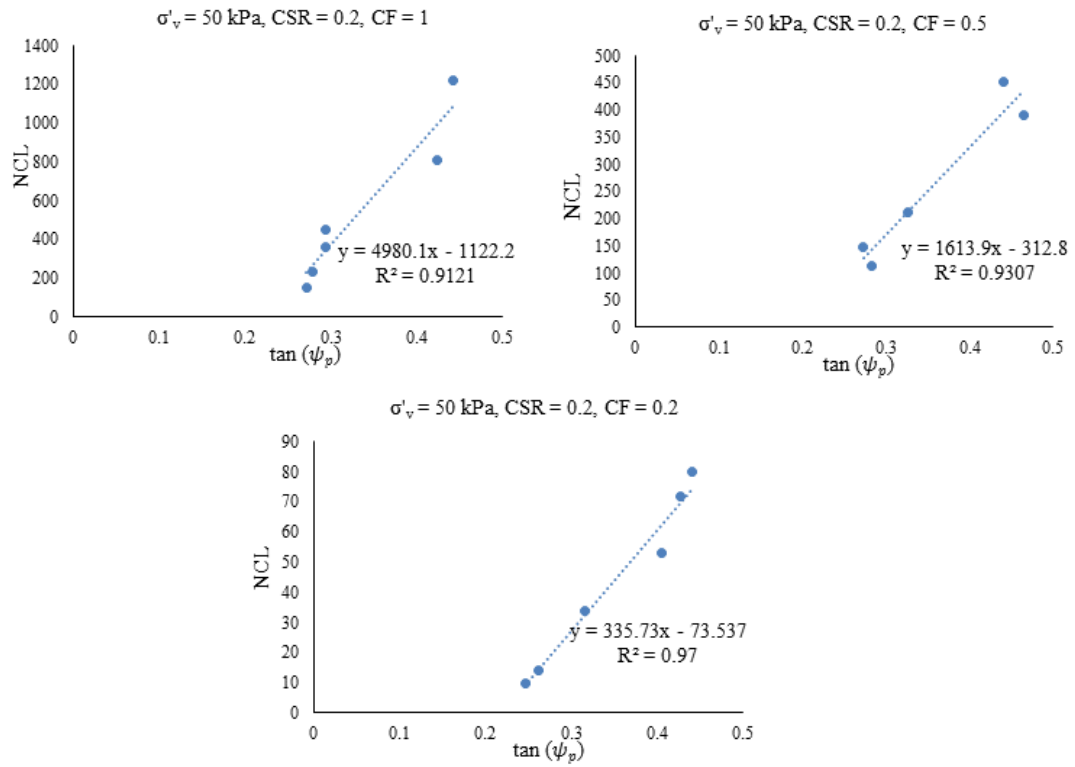


Figure 4.23. Kilyos CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.2$.

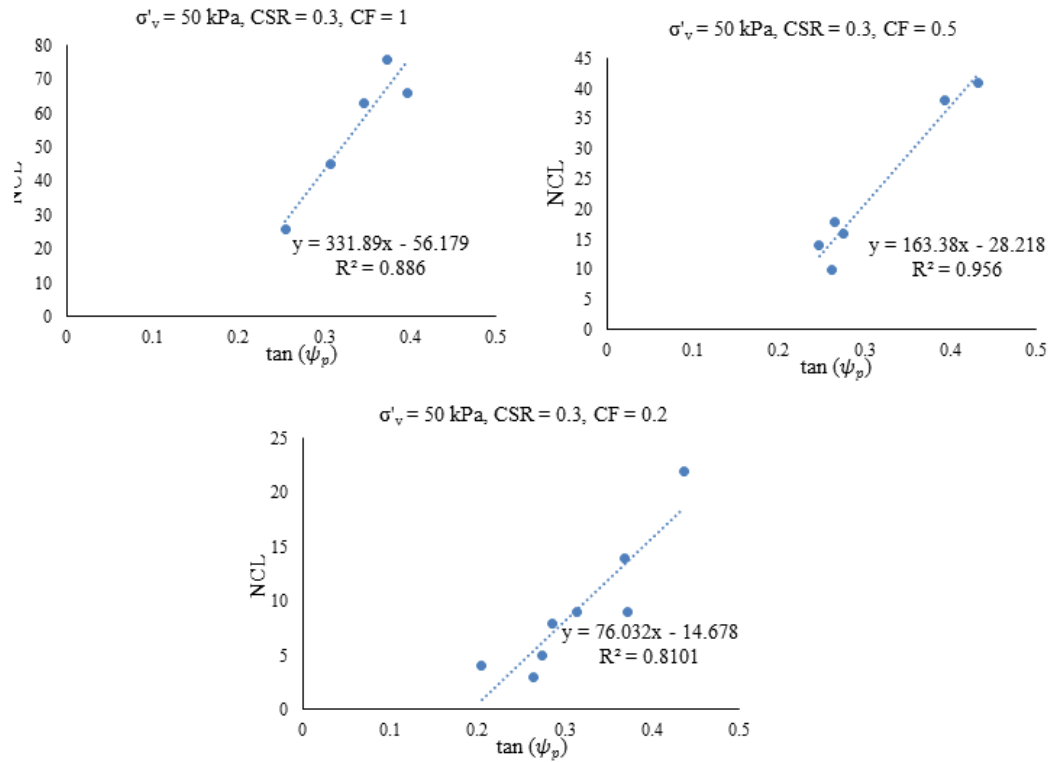


Figure 4.24. Kilyos CSS Results for $\sigma'_v = 50$ kPa and $CSR = 0.3$.

Table 4.21. Kilyos CSS Results $\sigma'_v = 100$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	100	0.2	1	0.47	16.5	444
2	100	0.2	1	0.49	16.9	471
3	100	0.2	1	0.54	18.8	770
4	100	0.2	1	0.56	19.3	487
5	100	0.2	1	0.68	23.1	1816
6	100	0.2	1	0.73	24.6	1957
7	100	0.2	0.5	0.42	14.6	247
8	100	0.2	0.5	0.44	15.3	299
9	100	0.2	0.5	0.53	18.2	389
10	100	0.2	0.5	0.58	20.1	437
11	100	0.2	0.5	0.71	23.9	827
12	100	0.2	0.2	0.56	19.2	81
13	100	0.2	0.2	0.55	18.9	88
14	100	0.2	0.2	0.49	17.1	68
15	100	0.2	0.2	0.38	13.2	48
16	100	0.2	0.2	0.63	21.6	108
17	100	0.2	0.2	0.71	24.1	136
18	100	0.3	0.5	0.64	21.9	186
19	100	0.3	0.5	0.77	25.8	212
20	100	0.3	0.5	0.36	12.6	39
21	100	0.3	0.5	0.71	24	181
22	100	0.3	0.5	0.68	23.1	144
23	100	0.3	1	0.64	21.8	50
24	100	0.3	1	0.53	18.4	21
25	100	0.3	1	0.35	12.2	8
26	100	0.3	1	0.62	21.2	30
27	100	0.3	1	0.58	19.9	33
28	100	0.3	0.2	0.49	17.1	16
29	100	0.3	0.2	0.54	18.6	29
30	100	0.3	0.2	0.34	12.0	4
31	100	0.3	0.2	0.74	25.0	62
32	100	0.3	0.2	0.79	26.3	47

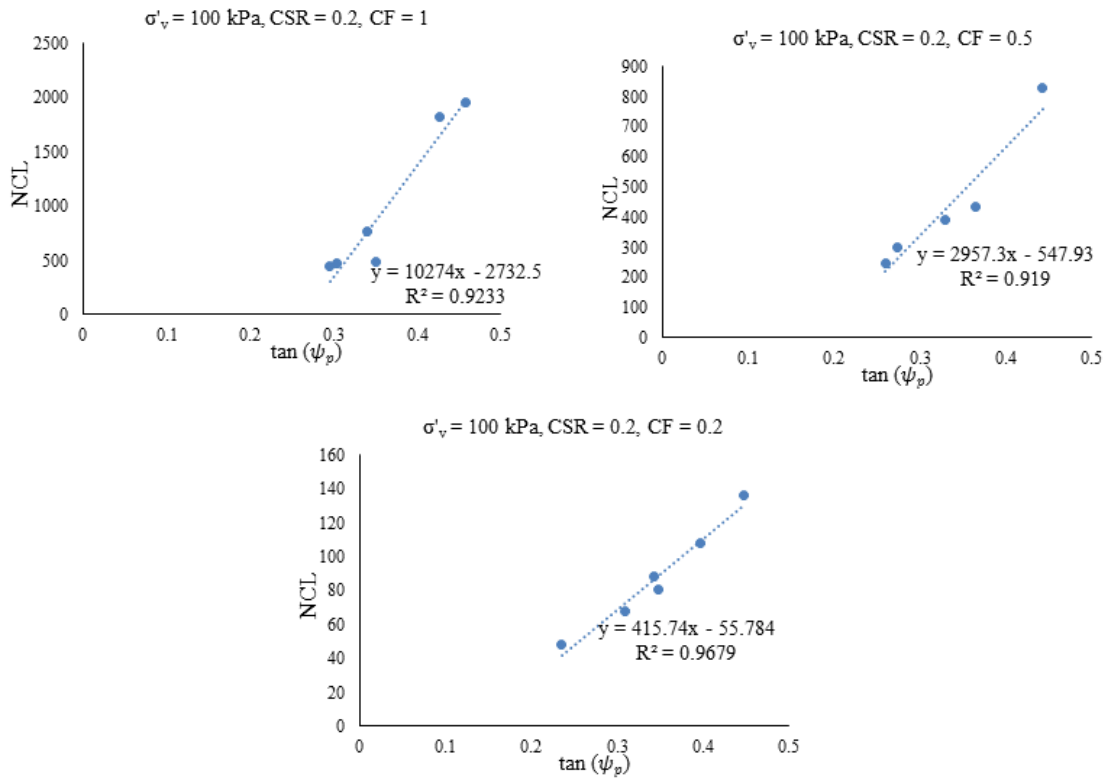


Figure 4.25. Kilyos CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.2$.

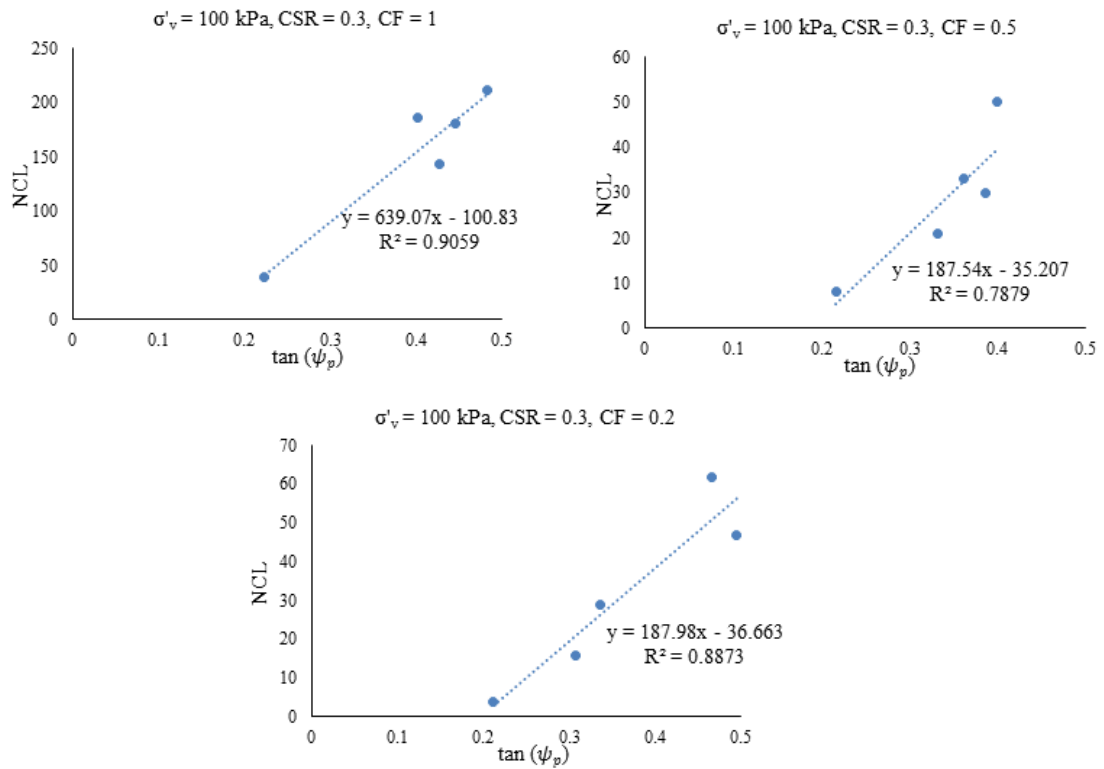


Figure 4.26. Kilyos CSS Results for $\sigma'_v = 100$ kPa and $CSR = 0.3$.

Table 4.22. Kilyos CSS Results $\sigma'_v = 200$ kPa.

Sample Number	σ'_v (kPa)	CSR	CF	I_D	ψ_p	NCL
1	200	0.2	1	0.46	16.1	667
2	200	0.2	1	0.47	16.3	704
3	200	0.2	1	0.50	17.4	1204
4	200	0.2	1	0.61	20.9	3824
5	200	0.2	1	0.63	21.6	4180
6	200	0.2	1	0.78	26.1	4591
7	200	0.2	0.5	0.50	17.5	213
8	200	0.2	0.5	0.54	18.6	410
9	200	0.2	0.5	0.70	23.6	711
10	200	0.2	0.5	0.81	26.8	1550
11	200	0.2	0.5	0.53	18.2	421
12	200	0.2	0.2	0.52	18.1	132
13	200	0.2	0.2	0.54	18.6	156
14	200	0.2	0.2	0.44	15.5	76
15	200	0.3	0.2	0.73	24.4	431
16	200	0.3	0.2	0.58	20.1	205
17	200	0.3	1	0.36	12.5	25
18	200	0.3	1	0.64	21.8	156
19	200	0.3	1	0.68	23.2	272
20	200	0.3	1	0.46	16.1	55
21	200	0.3	1	0.71	24	304
22	200	0.3	0.5	0.41	14.3	66
23	200	0.3	0.5	0.45	15.6	33
24	200	0.3	0.5	0.81	26.9	195
25	200	0.3	0.5	0.6	20.5	145
26	200	0.3	0.5	0.66	22.5	156
27	200	0.3	0.2	0.56	19.3	32
28	200	0.3	0.2	0.61	20.9	52
29	200	0.3	0.2	0.38	13.5	3
30	200	0.3	0.2	0.6	20.6	36
31	200	0.3	0.2	0.74	24.9	103

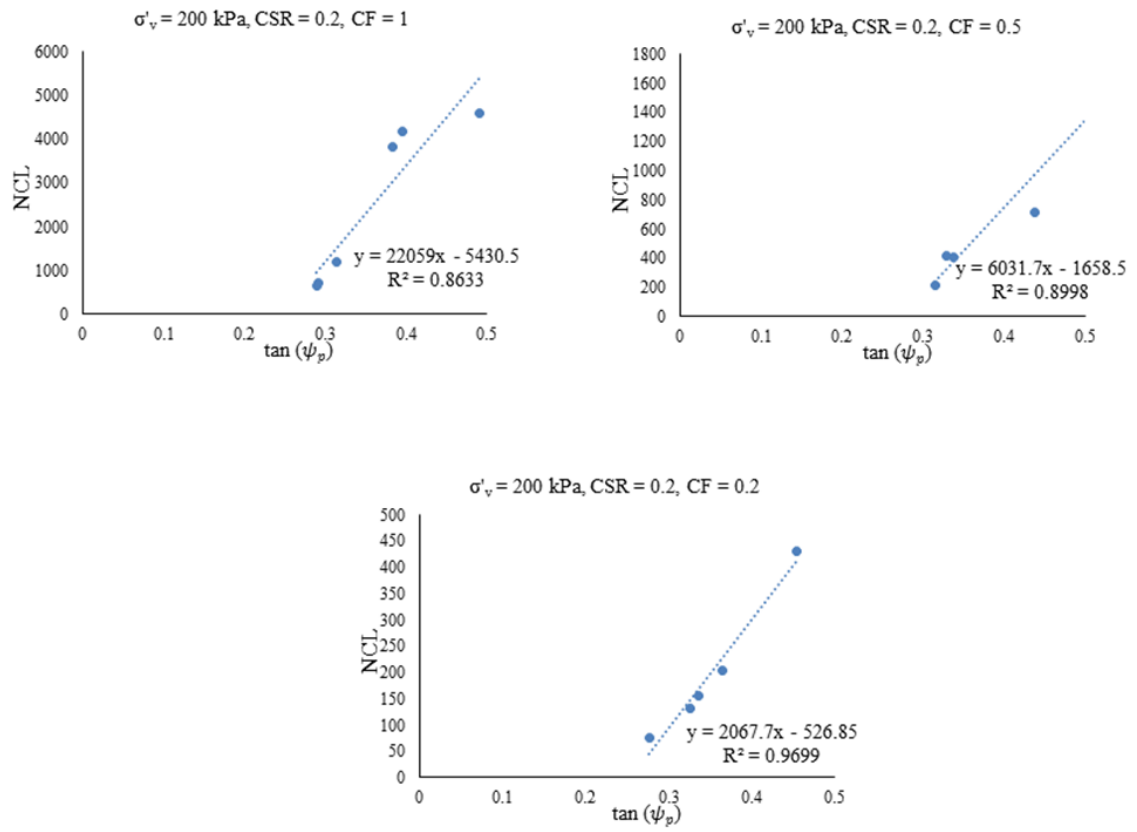


Figure 4.27. Kiyos CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.2$.

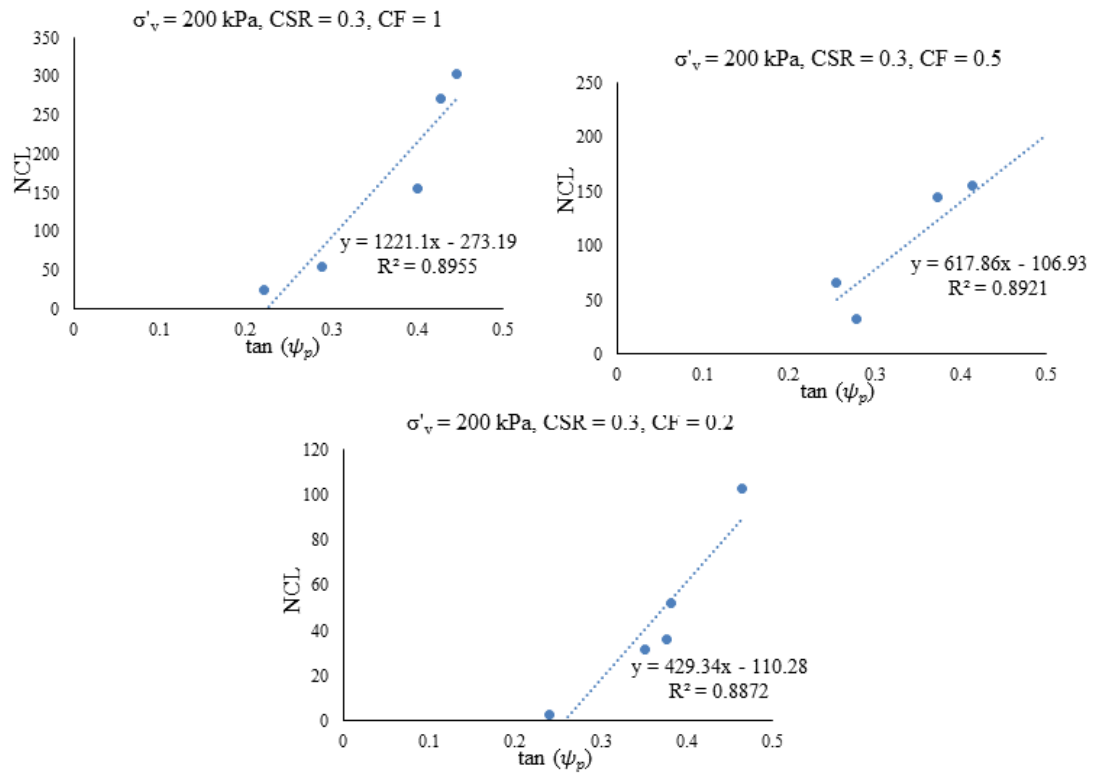


Figure 4.28. Kilyos CSS Results for $\sigma'_v = 200$ kPa and $CSR = 0.3$.

Table 4.23. Kilyos Sand $\tan\psi_p$ - NCL Linear Relationships.

Combination	Test Characteristics	Relationship
1	$\sigma'_v = 50$ kPa, $CSR = 0.2$, $CF = 1$	$NCL = 4980.1 \tan\psi_p - 1122.2$
2	$\sigma'_v = 50$ kPa, $CSR = 0.2$, $CF = 0.5$	$NCL = 1613.9 \tan\psi_p - 312.8$
3	$\sigma'_v = 50$ kPa, $CSR = 0.2$, $CF = 0.2$	$NCL = 335.73 \tan\psi_p - 73.537$
4	$\sigma'_v = 50$ kPa, $CSR = 0.3$, $CF = 1$	$NCL = 331.89 \tan\psi_p - 56.179$
5	$\sigma'_v = 50$ kPa, $CSR = 0.3$, $CF = 0.5$	$NCL = 163.38 \tan\psi_p - 28.218$
6	$\sigma'_v = 50$ kPa, $CSR = 0.3$, $CF = 0.2$	$NCL = 76.032 \tan\psi_p - 14.678$
7	$\sigma'_v = 100$ kPa, $CSR = 0.2$, $CF = 1$	$NCL = 10274 \tan\psi_p - 2732.5$
8	$\sigma'_v = 100$ kPa, $CSR = 0.2$, $CF = 0.5$	$NCL = 2957.3 \tan\psi_p - 547.93$
9	$\sigma'_v = 100$ kPa, $CSR = 0.2$, $CF = 0.2$	$NCL = 415.74 \tan\psi_p - 55.784$
10	$\sigma'_v = 100$ kPa, $CSR = 0.3$, $CF = 1$	$NCL = 639.07 \tan\psi_p - 100.83$
11	$\sigma'_v = 100$ kPa, $CSR = 0.3$, $CF = 0.5$	$NCL = 187.54 \tan\psi_p - 35.207$
12	$\sigma'_v = 100$ kPa, $CSR = 0.3$, $CF = 0.2$	$NCL = 187.98 \tan\psi_p - 36.663$
13	$\sigma'_v = 200$ kPa, $CSR = 0.2$, $CF = 1$	$NCL = 22059 \tan\psi_p - 5430.5$
14	$\sigma'_v = 200$ kPa, $CSR = 0.2$, $CF = 0.5$	$NCL = 6031.7 \tan\psi_p - 1658.5$
15	$\sigma'_v = 200$ kPa, $CSR = 0.2$, $CF = 0.2$	$NCL = 2067.7 \tan\psi_p - 526.85$
16	$\sigma'_v = 200$ kPa, $CSR = 0.3$, $CF = 1$	$NCL = 1221.1 \tan\psi_p - 273.19$
17	$\sigma'_v = 200$ kPa, $CSR = 0.3$, $CF = 0.5$	$NCL = 617.86 \tan\psi_p - 106.93$
18	$\sigma'_v = 50$ kPa, $CSR = 0.3$, $CF = 0.2$	$NCL = 429.34 \tan\psi_p - 110.28$

5. DISCUSSIONS

5.1. Effect of Initial Vertical Effective Stress on Cyclic Mobility

In order to monitor the influence of vertical effective stress on test results, tests for which all the other influences are unchanging, were examined. Accordingly, following figures shows the obtained NCL- $\tan\psi_p$ relationships for $\text{CSR} = 0.2$ & $\text{CF} = 1$ Hz and $\text{CSR} = 0.3$ & $\text{CF} = 1$ Hz for all three sands under investigation in the following figures. As it can be seen from the results, the tests with the higher initial vertical effective stresses have greater liquefaction resistance as evidenced by the higher NCL values. All sand types show the same behavior. Clearly, initial vertical effective stress magnitude influences the liquefaction susceptibility.

Moreover, the changes in NCL- $\tan\psi_p$ relationship is directly proportional to the applied σ'_{vc} value. It is obvious that NCL - $\tan\psi_p$ relationships are practically linear for the same σ'_{vc} levels. However, changing σ'_{vc} , changes the linear relationship. Ranjpour (2015) also observed the same behavior.

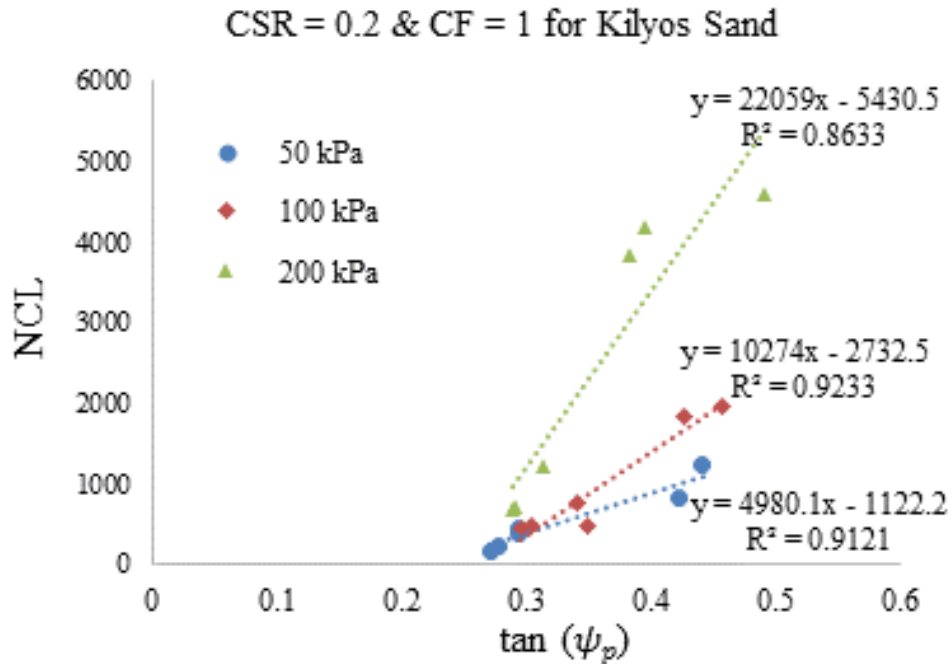


Figure 5.1. $\tan\psi_p$ - NCL relationship ($\text{CF} = 1$ Hz and $\text{CSR} = 0.2$) for Kilyos Sand.

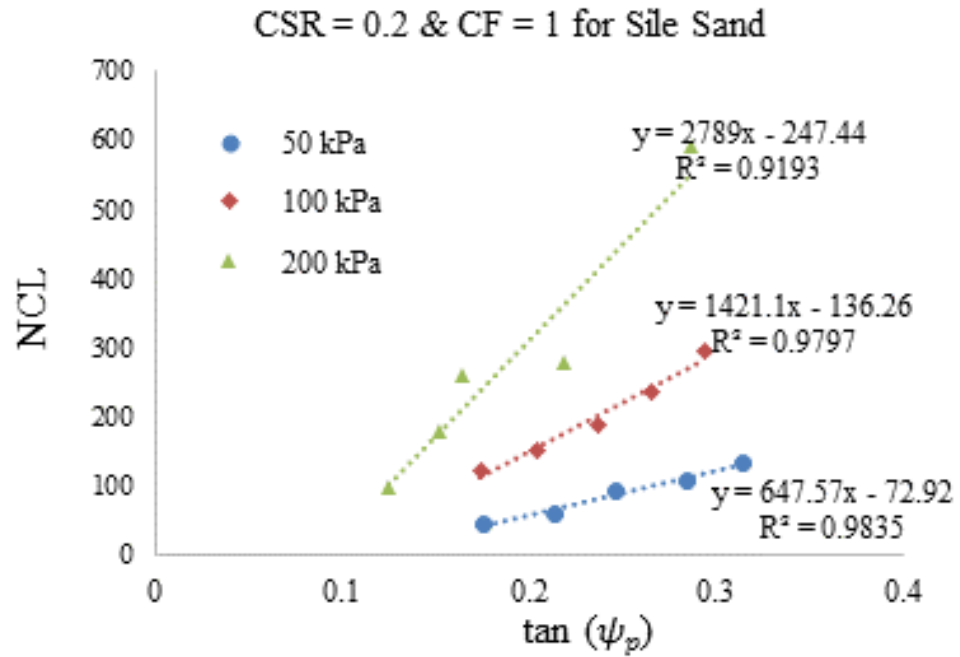


Figure 5.2. $\tan\psi_p$ - NCL relationship (CF = 1 Hz and CSR = 0.2) for Sile Sand.

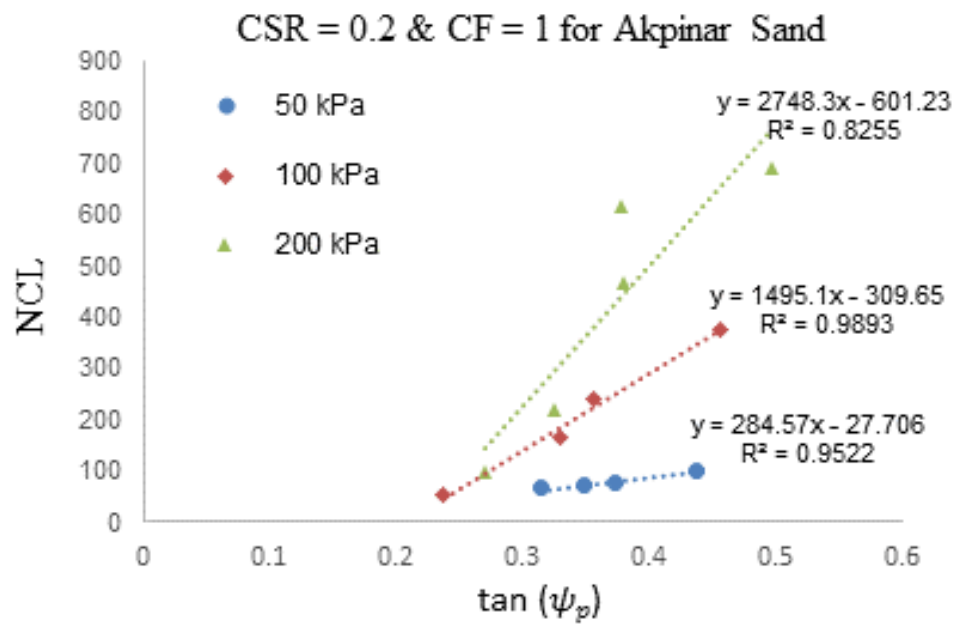


Figure 5.3. $\tan\psi_p$ - NCL relationships (CF = 1 Hz and CSR = 0.2) for Akpinar Sand.

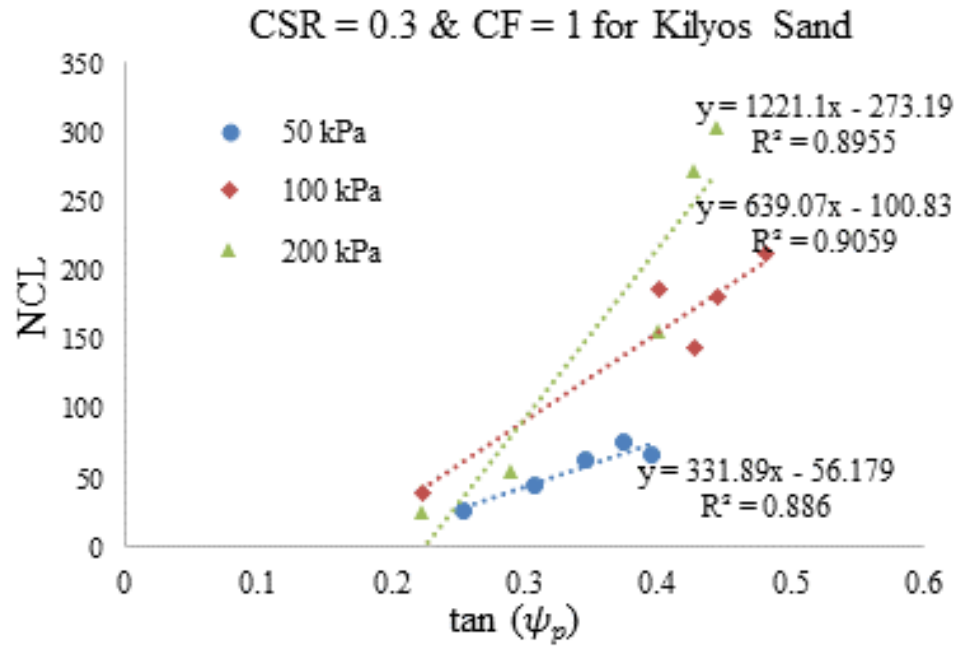


Figure 5.4. $\tan\psi_p$ - NCL relationship (CF = 1 Hz and CSR = 0.3) for Kilyos Sand.

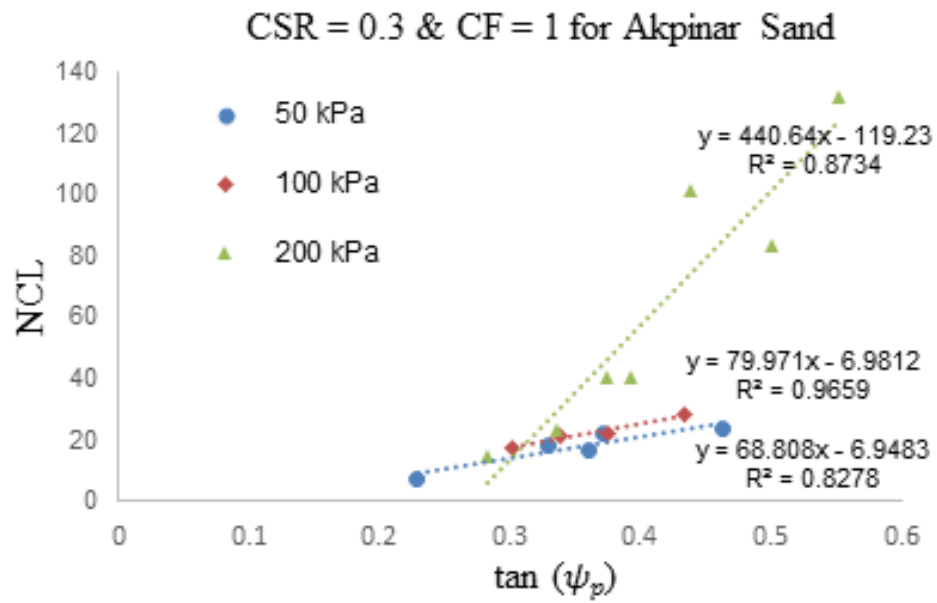


Figure 5.5. $\tan\psi_p$ - NCL relationship (CF = 1 Hz and CSR = 0.3) for Akpınar Sand.

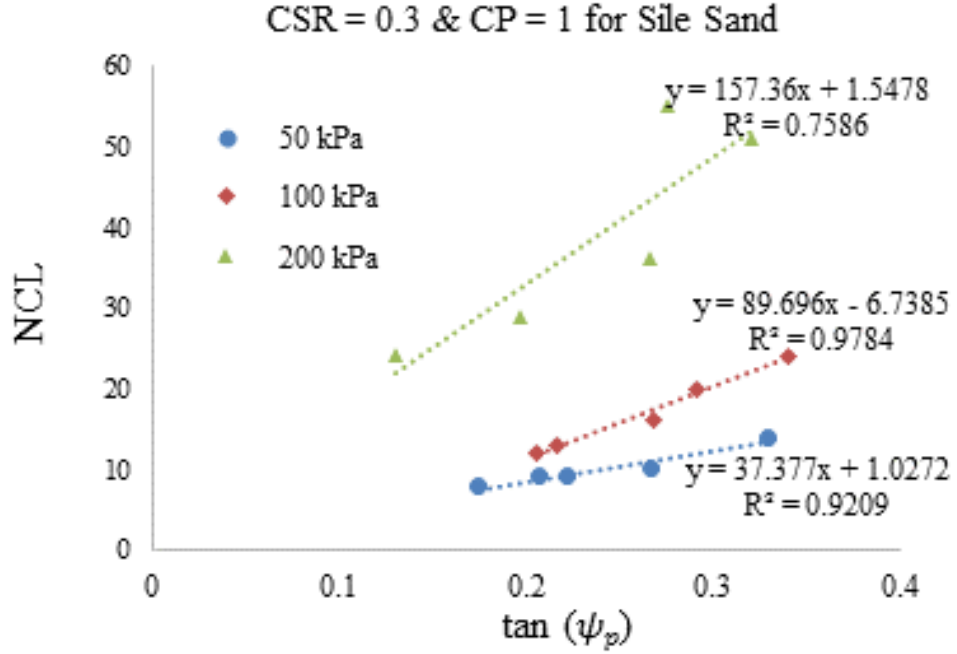


Figure 5.6. $\tan\psi_p$ - NCL relationship (CF = 1 Hz and CSR = 0.3) for Sile Sand.

5.2. Effect of Cyclic Frequency on Cyclic Mobility

Literature review about cyclic simple shear testing shows that; research interest was focused on lower cyclic frequencies, mainly on 0.1 Hz, although this frequency is too low for a typical earthquake. Sriskandakumar *et al.*, (2012) state that the undrained behavior of sand is known to be essentially frequency independent and 0.1 Hz was commonly adopted as reasonable for cyclic loading of granular materials for laboratory work. This study also questions this common approach and uses higher frequencies, which are 1, 0.5 and 0.2 Hz.

To be able to identify the influence of CF on liquefaction susceptibility, $\tan\psi_p$ - NCL relationships of Kilyos sand under 50 kPa, 100 kPa and 200 kPa and same CSR value can be examined. Figure 5.7 compares all the combinations of CSR=0.2 for Kilyos sand and shows that, decrease in cyclic frequency causes decrease in liquefaction resistance.

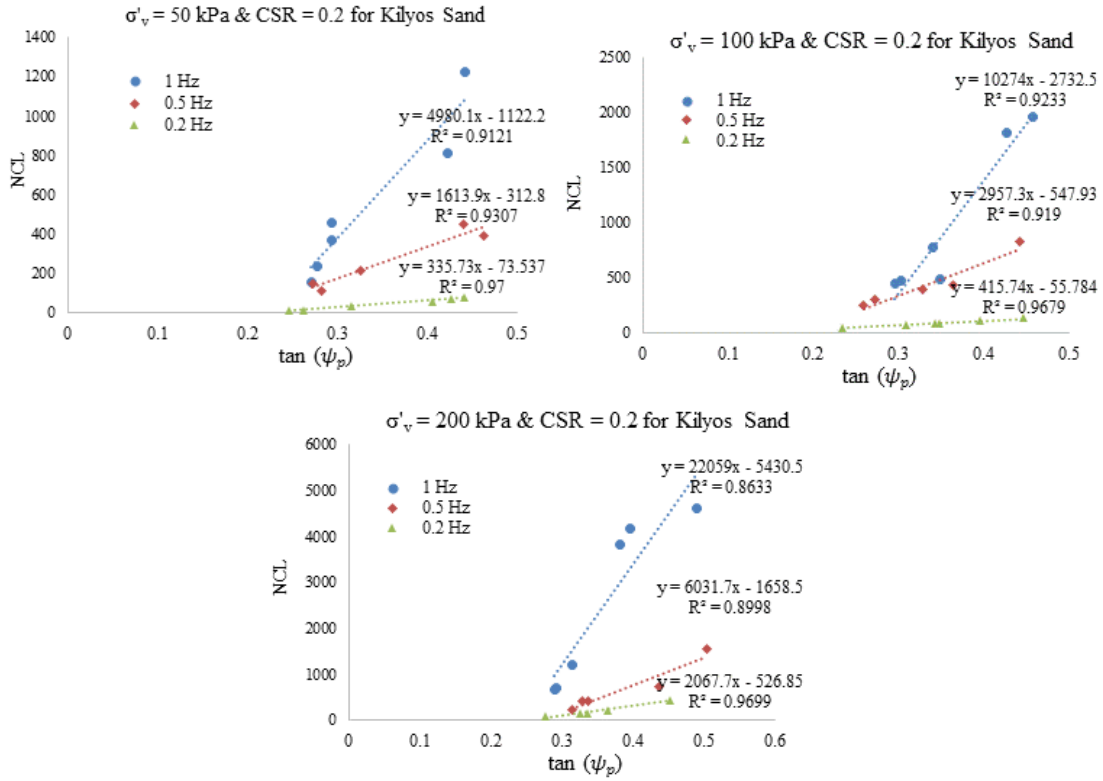


Figure 5.7. $\tan\psi_p$ - NCL relationships ($CF = 1$ Hz and $CSR = 0.2$) for Kilyos Sand under 50 kPa, 100 kPa and 200 kPa initial vertical effective stress.

5.3. Effect of Cyclic Stress Ratio on Cyclic Mobility

Since samples were consolidated with either 50 kPa, 100 kPa or 200 kPa, for tests with $CSR = 0.2$, the cyclic shear stresses applied to the specimens were 10 kPa, 20 kPa or 40 kPa. Additionally, for tests with $CSR = 0.3$, applied cyclic shear stresses were 15 kPa, 30 kPa and 60 kPa.

In order to see the CSR effect clearly, it is important to examine the test results by keeping the cyclic frequency constant. The best indicator for comparing the influences of different factors on liquefaction resistance is to compare their effects on the gradients of the best-fit line equations.

With the aim of observing the behavior of all three sands under investigation, their responses under 50 kPa initial vertical effective stress are plotted. All sands tend

to follow the same trend. As it can be seen in the following figures, NCL values are significantly higher for tests conducted with lower cyclic stress ratios. This outcome is expected, as loading with lower cyclic shear stresses requires greater number of loading cycles to reach liquefaction.

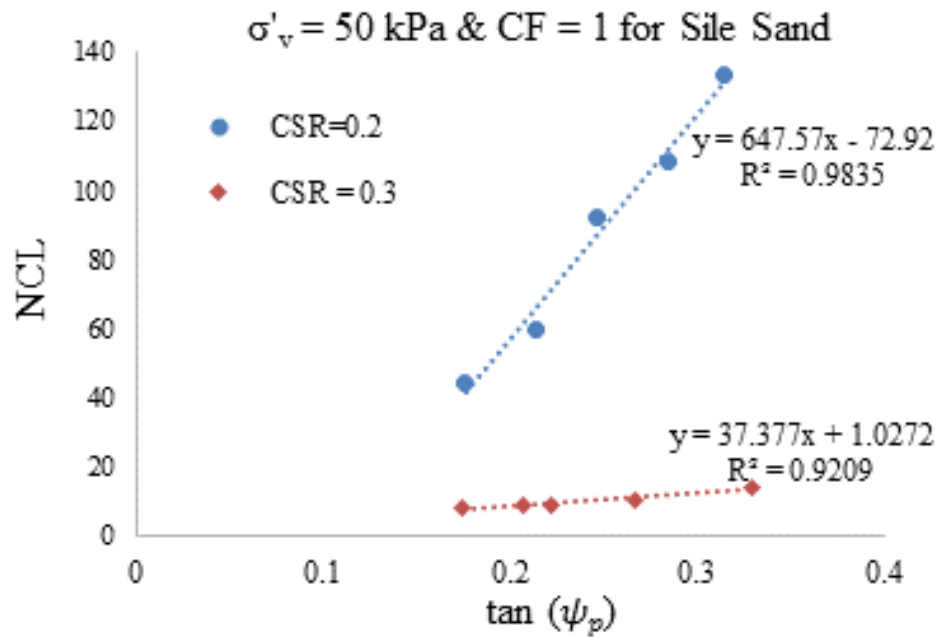


Figure 5.8. $\tan\psi_p$ - NCL relationship of Sile Sand under $\sigma'_v = 50 \text{ kPa}$ and $CF = 1 \text{ Hz}$.

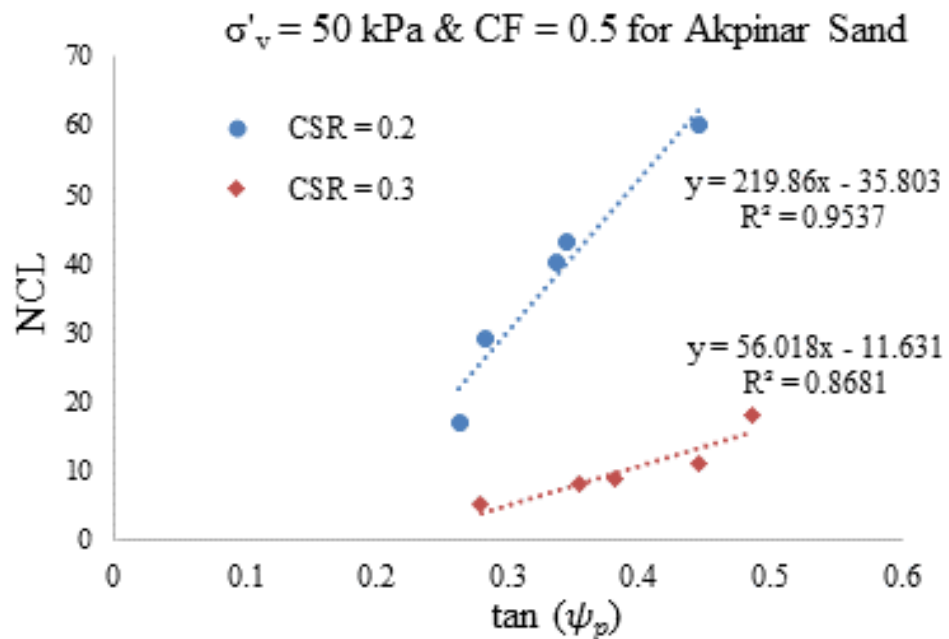


Figure 5.9. $\tan\psi_p$ - NCL relationship of Akpinar Sand under $\sigma'_v = 50 \text{ kPa}$ and $CF = 0.5 \text{ Hz}$.

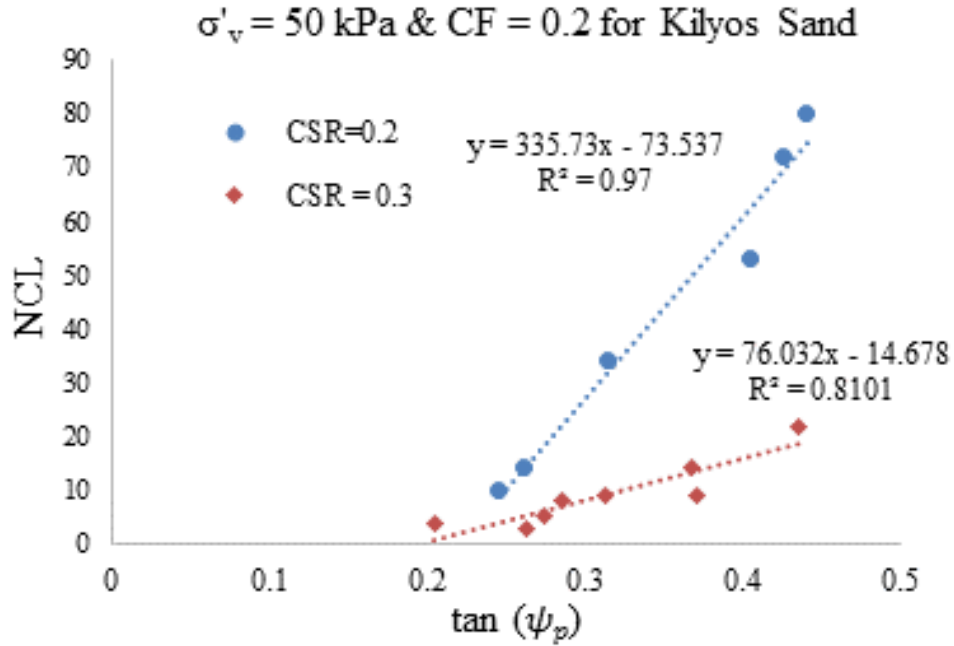


Figure 5.10. $\tan\psi_p$ - NCL relationship of Kilyos Sand under $\sigma'_v = 50 \text{ kPa}$ and $CF = 0.2 \text{ Hz}$.

5.4. Comparing the Behavior Different Sand Types

Analyzing the line equations of all 18 combinations of cyclic simple shear tests for three sands shows that, in every case, Kilyos sand is most resistant to liquefaction.

For all sands in this study, $\tan \psi_p$ - NCL relationships are more influenced by the changes in CSR rather than changes in CF. However this does not mean that the relationships are frequency independent, although this is generally accepted by the current literature for cyclic behavior of sands in cyclic simple shear testing. For all three sands under investigation, increase in cyclic frequency led to an increase in liquefaction resistance.

It is not possible to come to a definitive conclusion by only comparing the results of dilatancy behaviors of Kilyos and Sile sand, since both shape parameters and gradation curves differ for each sand. However it should be noted that; although Sile sand is relatively well graded and has a less circular nature, it is less resistant to liquefaction

when compared to Kilyos sand.

Akpinar and Kilyos sands have similar gradation curves, uniformity and curvature coefficients, while their particle shape parameters differ. Kilyos sand has higher circularity and roundness values in comparison to Akpinar Sand. The overall results show that Kilyos sand is more resistant to liquefaction than Akpinar in all loading combinations. Generally, with increasing angularity, liquefaction resistance is also expected to increase for the samples under same loading combinations. However, for this study, results can suggest that increasing circularity and/or roundness can result in an increase in liquefaction resistance. The reason behind this unexpected outcome can be mineral and/or surface roughness characteristics of Kilyos sand, which were not part of this study.

6. CONCLUSIONS

With the aim of investigating the relationship between dilatancy, cyclic mobility and average particle shape, 19 K_o CD triaxial tests and over 275 cyclic simple shear tests with three sand types were conducted. Dilatancy constants for predicting the peak dilatancy angles and K_o - relative density relationships were obtained from the results of the triaxial tests. 50 particles of each sand type was processed and average circularity and roundness values were found using an image processing software. Since 18 different combinations of variables were used in cyclic simple shear tests, the individual impacts of consolidation pressure, cyclic stress ratio and cyclic frequency on relationships of peak dilatancy versus liquefaction susceptibility for each sand type were presented.

According to analysis of the test results, critical friction angle and liquefaction resistance is the highest for the Kilyos sand, which has the highest circularity and roundness values. Sile sand, which is relatively well graded with less circular nature and Akpınar sand, which has a similar gradation curve with Kilyos sand with a lower circularity and roundness value, are more susceptible to liquefaction in comparison to Kilyos sand.

Results show that, the effect of change in CSR has a more dominant influence on the $\tan\psi_p$ - NCL relationships than the effect of change in CF. Considering the same CF and initial vertical stress values, for the same ψ_p , higher CSR values leads to a decrease in NCL values.

Additionally, it is also noted that; although commonly accepted in the literature, liquefaction resistance is not frequency independent. In this study, three frequencies (0.2, 0.5 and 1 Hz) were used, with the aim of a better simulation of the earthquake field conditions. In every case, change in frequency led to a change in the $\tan\psi_p$ - NCL relationship. Under the same initial vertical stress and cyclic stress ratio combinations, decrease in cyclic frequency causes decrease in liquefaction resistance for the same ψ_p value.

Recommendations for future studies

- Effect of silt content on peak dilatancy angle and liquefaction susceptibility can be investigated.
- Investigating the effect of particle roughness and mineralogy on peak dilatancy angle and liquefaction susceptibility
- Testing with different sand types. Investigating the behaviour of more angular, circular and well graded sands should be a priority.
- Different CSR values can be taken for cyclic simple shear testing.
- Effect of overconsolidation pressure can be investigated.

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