# INFLUENCE OF NON-PLASTIC SILT CONTENT ON SEISMIC LIQUEFACTION OF SANDS

by

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INFLUENCE OF NON-PLASTIC SILT CONTENT ON SEISMIC LIQUEFACTION OF SANDS

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# ACKNOWLEDGEMENTS

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# ABSTRACT


## INFLUENCE OF NON-PLASTIC SILT CONTENT ON SEISMIC LIQUEFACTION OF SANDS

Earthquakes are one of the important problems of Turkey because of being in a seismically active region. During earthquakes, loose, saturated, cohesionless soils could significantly lose their shear strength because of great excess pore water pressures generated and act like fluid, which is called soil liquefaction. Liquefaction is one of the most significant and worth researching phenomena in geotechnical engineering and it has been studied by researchers for decades. Liquefaction occurs mostly on sandy soils, but the soils in situ include a certain amount of silt. In this study, the effects of non-plastic silt contents on the liquefaction potential of sands were investigated experimentally at the Soil Mechanics Laboratory of Yeditepe University. The clean sand was mixed with silt at three different fines contents (i.e. FC = 5 percent, 15 percent, and 35 percent). Cyclic Direct Simple Shear (CDSS) tests were performed on each soil under 100 kPa vertical effective stress at a wide range of different relative densities ($D_r$) and under three different cyclic stress ratios (i.e. CSR = 0.1, 0.12 and 0.14). The results were compared with that of the clean sand (i.e. FC = 0 percent), and the change in the seismic liquefaction potential was investigated. In conclusion, it was observed that liquefaction resistance of silty sands with 5 percent fines content is greater than the resistance of clean sand and liquefaction resistance of silty sands with 35 percent fines content is smaller than the resistance of clean sands. In addition, it has been observed that liquefaction resistance of silty sands with 15 percent fines content decreases on soils where the relative density is less than 45 percent compared to the resistance of clean sands and liquefaction resistance of silty sands with 15 percent fines content increases on soils where the relative density is higher than 45 percent compared to the resistance of clean sands.

# ÖZET

## PLASTİK OLMAYAN SİLT İÇERİĞİNİN KUMLARIN SİSMİK SIVILAŞMASINA ETKİSİ

Depremler, sismik olarak aktif bir bölgede bulunan Türkiye'nin en önemli problemlerinden birisidir. Deprem esnasında, aşırı boşluk suyu basıncının meydana çıkmasından dolayı, gevşek, doygun ve kohezyonsuz zeminler kayma dayanımlarını önemli bir oranda kaybedebilir ve adeta sıvı gibi davranabilirler. Bu durum zemin sıvılaşması olarak adlandırılır. Sıvılaşma, geoteknik mühendisliğinde araştırmaya değer ve en önemli olaylardan biridir ve araştırmacılar tarafından on yıllardır çalışılmaktadır. Sıvılaşma en çok kumlu zeminlerde meydana gelir ancak arazide bulunan zeminler belirli miktarlarda silt içerirler. Bu çalışmada, plastik olmayan silt içeriklerinin temiz kumların sıvılaşma potansiyeline etkileri, deneysel olarak Yeditepe Üniversitesi Zemin Mekaniği Laboratuvarında araştırılmıştır. Temiz kum, 3 farklı ince dane oranında (FC = yüzde 5, yüzde 15 ve yüzde 35) siltlerle karıştırılmıştır. Çevrimsel Direkt Basit Kesme Deneyleri (CDSS), her bir zemin için 100 kPa düşey efektif gerilme altında, geniş aralıkta ve farklı göreceli sıkılıklarda ($D_r$) ve 3 farklı çevrimsel gerilme oranında (CSR = 0.1, 0.12 ve 0.14) gerçekleştirilmiştir. Sonuçlar temiz kum (FC = yüzde 0) ile karşılaştırılmış ve sismik sıvılaşma potansiyelindeki değişiklik araştırılmıştır. Neticede, yüzde 5 ince daneli siltli kumların sıvılaşma direncinin temiz kumların direncinden yüksek olduğu ve yüzde 35 ince daneli siltli kumların sıvılaşma direncinin temiz kumların direncinden düşük olduğu gözlemlenmiştir. Ayrıca, yüzde 15 ince dane içeriğine sahip siltli kumların sıvılaşma direncinin, göreceli sıkılığın yüzde 45'ten az olduğu zeminlerde, temiz kumların direncine kıyasla azaldığı gözlemlenmiş ve yüzde 15 ince dane içeriğine sahip siltli kumların sıvılaşma direncinin göreceli sıkılığın yüzde 45'ten fazla olduğu zeminlerde ise temiz kumların direncine kıyasla arttığı gözlemlenmiştir.

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| | |
|---|---|
| $c'$ | Cohesion |
| $C_c$ | Coefficient of curvuture |
| $C_u$ | Coefficient of uniformity |
| $D_{10}$, $D_{30}$, $D_{60}$ | Grain diameters passing 10, 30 and 60 percent passing |
| $D_r$ | Relative density |
| e | Void ratio |
| $e_c$ | Constant void ratio |
| $e_{min}$ | Minimum void ratio |
| $e_{max}$ | Maximum void ratio |
| $F_S$ | Factor of safety |
| FC | Fines content |
| $g$ | Acceleration of gravity |
| $G_s$ | Specific gravity |
| LFC | Limiting fines content |
| LL | Liquid limit |
| LSC | Limiting silt content |
| $M_W$ | Moment magnitude |
| $N_L$ | Number of cycles required for liquefaction |
| $r_d$ | Stress reduction factor |
| $r_u$ | Excess pore water pressure ratio |
| $u$ | Pore water pressure |
| | |
| $a_{max}$ | Peak horizontal acceleration at the ground surface generated by the earthquake |
| $\sigma'_c$ | Vertical consolidation stress |
| $\sigma'_v, \sigma'_{vo}$ | Vertical effective stress |
| $\sigma_v, \sigma_{vo}$ | Total effective stress |
| $\gamma$ | Shear strain |
| $\tau, \tau_{av}$ | Shear stress |

| | |
|---|---|
| ASTM | American Society for Testing and Materials |
| CDSS | Cyclic direct simple shear |
| CRR | Cyclic resistance ratio |
| CSR | Cyclic stress ratio |
| CTX | Cyclic triaxial |
| NGI | Norwegian Geotechnical Institute |
| USCS | Unified Soil Classification System |

# 1. INTRODUCTION

## 1.1. THE STRUCTURE OF THIS THESIS

In the first chapter of this study, the history of liquefaction, its effects on structures and the purpose of the thesis are mentioned. The second chapter includes what kinds of studies were done in the literature and background. The third chapter covers the experiments which were done to determine the characteristic properties of the soils used in the thesis study. In the fourth chapter, procedure and working principles of the cyclic simple shear test machine were mentioned. In the fifth chapter, the influence of non-plastic silt contents on seismic liquefaction resistance of sands were discussed in the light of the data obtained as a result of experimental studies. In the last chapter of this study, thesis was summarized and the results obtained by experimental studies were brought into the literature.

## 1.2. THE GENERAL CONCEPT OF LIQUEFACTION

Liquefaction which has been studied for decades is a very significant and complex title of geotechnical earthquake engineering. The first destructive earthquakes leading to the study of liquefaction by geotechnical engineers are the Good Friday earthquake in Alaska and the Niigata earthquake in Japan in a three-month period in 1964 [1]. Good Friday earthquake in Alaska, which is one of the strongest earthquakes ($M_W$ = 9.2) ever measured in North America, affected a wide area [2]. In addition, Iwasaki [3] explained that most of damages during Niigata earthquake were caused by loose and saturated sand spreading along the Shinano River and Agano River in Niigata City. These two earthquakes provide a good example for us to see the effects of liquefaction such as loss of bearing capacity, differential settlements, sand boilings, retaining structure failures, floating of buried structures and lateral spreading.

Figure 1.1. Good Friday earthquake in Alaska in 1964 [4]

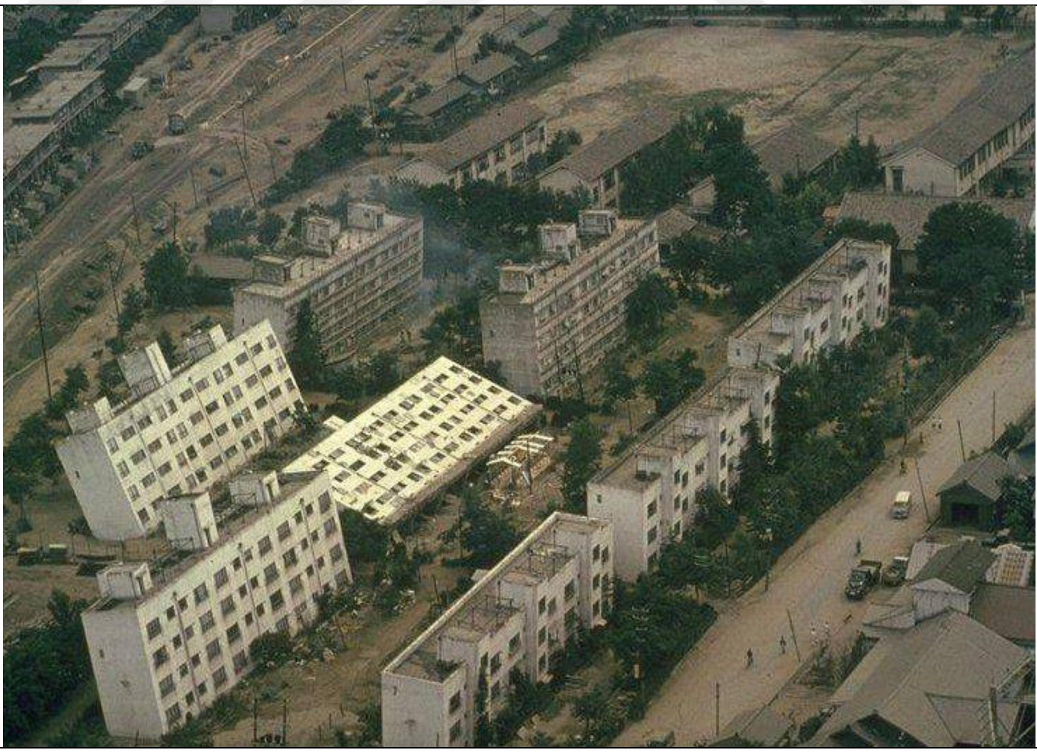

Figure 1.2. Niigata earthquake in Japan in 1964 [5]

In our recent history, there are also many earthquakes where the effects of soil liquefaction have been observed such as Kobe earthquake (1995), Izmit earthquake (1999), Niigata earthquake (2004), Haiti earthquake (2010), Christchurch earthquake (2011), Tohoku earthquake (2011) and Nepal earthquake (2015).

Figure 1.3. Kobe earthquake in Japan in 1995 [6]

Figure 1.4. Izmit earthquake in Turkey in 1999 [7]

Figure 1.5. Christchurch earthquake in New Zealand in 2011 [8]

When all these destructive effects of liquefaction are taken into consideration, it is seen how important this study is, since our country is located in a seismically active region. The liquefaction behavior of clean sands has been studied for more than fifty years. However, the liquefaction behavior of fine grained soils containing silty sands has been continuing for more than twenty years. It has been mentioned that the liquefied soil near the surface contains silts and silty sands in Kocaeli Earthquake in 1999 [9]. It also has been reported that silts and silty sands are liquefied in Tohoku Earthquake in 2011 [10]. In addition, Taylor et al. [11] expressed that sandy soils includes certain amount of silt when liquefaction occurs in Christchurch Earthquake in 2011. The sandy soils in situ already contain various proportions of fine grain which was generally silts. Therefore, studying effects of silty sands on clean sands has been an important research topic for geotechnical engineers.

This study aims to investigate how different contents of non-plastic silt influence the seismic liquefaction of a sand by performing cyclic direct simple shear test.

# 2. LITERATURE REVIEW AND BACKGROUND

## 2.1. EVALUATION OF LIQUEFACTION POTENTIAL

Liquefaction is the loss of shear strength of saturated, loose and cohesionless soils under a sudden static or more typcially a dynamic loading such as an earthquake so that the soils act like a liquid. During liquefaction, due to notably increased pore water pressure, vertical (or mean) effective stress decreases so significantly that soil particles are able to move freely with respect to each other. Formulation of vertical effective stress was shown in equation 2.1.

$$\sigma_v' = \sigma_v - u \tag{2.1}$$

Reducing vertical effective stress causes the shear stress of soil to decrease. Formulation of shear stress in terms of Mohr-Coulomb strength criterion was shown in equation 2.2 [12].

$$\tau = c' + \sigma_v' tan\phi' \tag{2.2}$$

Seed et al. [13] stated that two methods were used to determine the cyclic liquefaction potential. The first one is based on empirical relations of in situ characteristic and observations. The second one is based completely on a controlling stress and strain conditions and the utilization of laboratory experiment procedures. Calculation of cyclic stress ratio (CSR) which is the ratio of the shear stress to the initial vertical effective stress was formulated by Seed and Idriss [14] in equation 2.3.

$$CSR = \frac{\tau_{av}}{\sigma_{vo}'} = 0.65 \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) r_d \tag{2.3}$$

In addition, cyclic resistance ratio (CRR) which is the ratio of resistance of shear stress to liquefaction to the vertical effective stress and corresponds to the moment magnitude of an earthquake was shown in equation 2.4 [15]. In other words, cyclic resistance ratio is the capability of the soil to withstand the liquefaction.

$$CRR = \frac{\tau_{resistance}}{\sigma'_{vo}} \tag{2.4}$$

As a result of studies performed by Seed et al. [13], the number of uniform cycles required for liquefaction corresponding to different magnitude earthquakes were shown in Table 2.1. Irregular earthquake-induced loading is characterized by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles [14]. In addition, graph of earthquake magnintude vs. equivalent number of cycles for liquefaction was shown in Figure 2.1.

Table 2.1. Number of cycles representative of different magnitude earthquakes [13]

| Magnitude | Number of representative cycles at 0.65 $\tau_{max}$ |
|---|---|
| 8.5 | 26 |
| 7.5 | 15 |
| 6.75 | 10 |
| 6 | 5-6 |
| 5.25 | 2-3 |

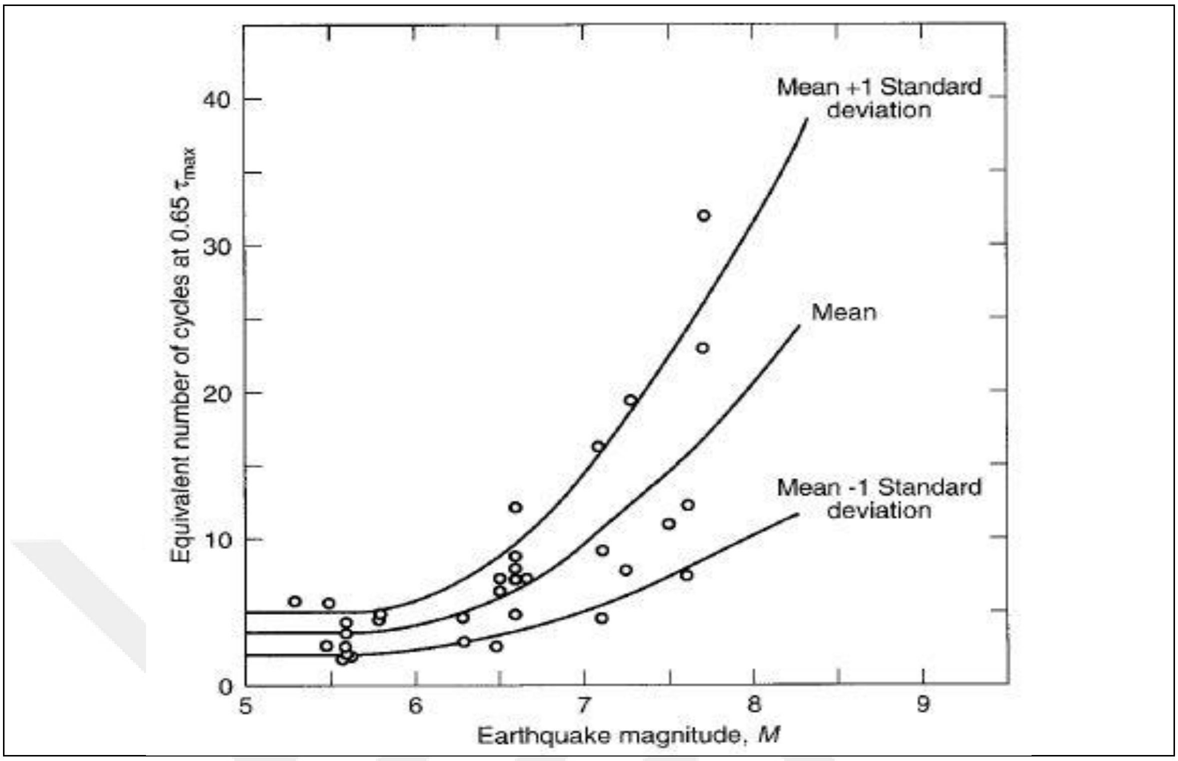


Figure 2.1. Earthquake magnitute vs. equivalent number of cycles at 0.65 $\tau_{max}$, Seed and Idriss (1971) [14]

It has been clear that, if the number of cycles required for liquefaction is selected as 15, this represents to a 7.5 moment magnitude. In this thesis, the number of cycles required for liquefaction was selected as 20 and this corresponds to about 7.9-8.0 moment magnitude.

Factor of safety ($F_S$) which is the ratio of cyclic resistance to the cyclic stress was shown in equation 2.5. There is no risk of liquefaction when the cyclic resistance ratio is greater than the cyclic stress ratio, however, if the cyclic stress ratio exceeds the cyclic resistance ratio liquefaction is supposed to happen.

$$F_S = \frac{CRR}{CSR} \tag{2.5}$$

Seed and Lee [16] reported that there are major factors that affects the risk of liquefaction, and these are void ratio, confining pressure and magnitude of cycle stress or strain. In this study, both the excess pore water pressure ratio ($r_u$) and the peak to peak shear strain ($\gamma$) were considered as liquefaction criteria. Excess pore water pressure ratio can be defined as the excess pore pressure to the vertical consolidation stress as shown in equation 2.6.

$$r_u = \frac{\Delta u}{\sigma_c'} \tag{2.6}$$

It was accepted that liquefaction occurs either when the excess pore water pressure ratio equals to 1.0 (the excess pore pressure equals to the vertical consolidation stress Δu = $\sigma_c'$ = 100 kPa) or when the peak to peak shear strain reaches the 10 percent, whichever occurs first. Therefore, in some cases, experiment were ended by the computer controlled test machine before the excess pore water pressure ratio reach 1.0, because peak to peak strain has reached the 10 percent. In this study, the reason of taking 10 percent of peak to peak shear strain is that excess pore water pressure ratio is close the 1.0. In addition, in some studies in the literature, peak to peak shear strain was chosen as 5 percent [17, 18].

## 2.2. THE EFFECTS OF NON-PLASTIC SILT CONTENT ON LIQUEFACTION POTENTIAL OF SANDS

The effects of silt content on liquefaction response of sands have been investigated with laboratory experiments by researchers for decades. In the literature, liquefaction behavior of silty sands has been investigated based on different density index parameters such as relative density, intergranular void ratio (which is the void ratio of sand matrix) or void ratio. In this study, the effects of silt content on liquefaction resistance of sands were analyzed based on the density index parameter of relative density. This is because; relative density is not only a commonly used density index parameter used in laboratory investigations of granular soils (sands, silty sands etc.), but also a parameter that is commonly linked to various in-situ tests such as SPT [19, 20, 21, 22] and CPT [23].

Some researchers have stated that increasing silt content increase the liquefaction potential of the sands, while others stated vice-versa.

### 2.2.1. Previous Studies Reporting an Increase in Liquefaction Resistance with Silt Content

Salgado et al. [24] reported that the liquefaction resistance of the sands increases with increasing silt content at constant or similar relative density in the range of 0-20 percent FC from cyclic traxial test. Similarly, Hazirbaba and Rathje [25] found that the liquefaction resistance of the silty sands is higher compared to the clean sand at constant relative density (50 percent) between 0-20 percent FC based on cyclic simple shear tests. Amini and Qi [26] performed cyclic traxial tests with Ottawa 20/30 sand and low-plasticity silts at constant relative density (40 percent) between 10-50 percent FC. They found that cyclic resistance increases with increasing silt content. Change of cyclic stress ratio with silt content under 100 kPa confining stress was shown in Figure 2.2. Jradi et al. [27] studied in a very narrow and low fines content range (1, 3 and 5 percent). They performed cyclic traxial tests in two different relative densities (loose and medium dense) and observed that cyclic resistance of sands increases with increasing fines content (≤ 5 percent) both at loose and medium dense states.

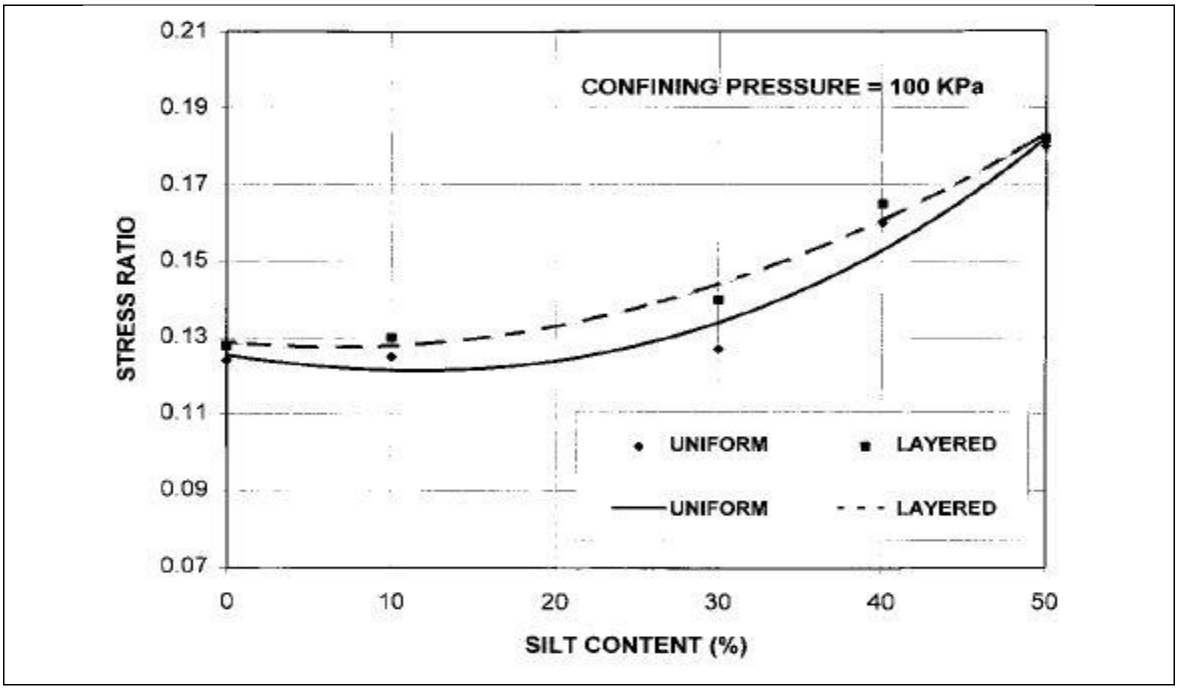


Figure 2.2. Comparison between liquefaction behavior of layered and uniform soils as function of silt content for stress ratios to cause liquefaction at ten cycles, Amini and Qi (2000) [26]

Polito and Martin [28] stated that cyclic resistance ratio of silty sands first increases with increasing silt content up to about 7 percent FC and then it decreases at 2 different constant relative densities (30 and 60 percent).

Similarly, Carraro et al. [29] reported that liquefaction resistance of sands first increased with increasing fines content up to 5 percent FC in range of 40-70 percent $D_r$ and then it decreased from 5 to 15 percent FC. The relationship between relative density and cyclic resistance ratio for between 0-15 percent FC was shown in Figure 2.3.

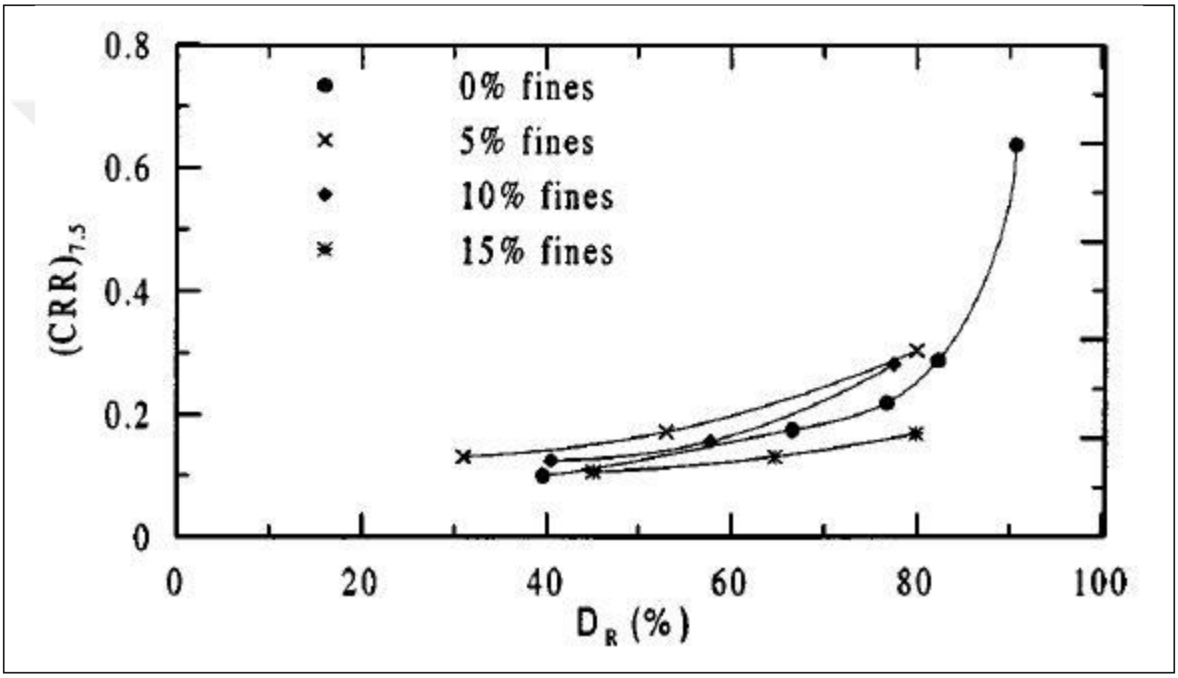


Figure 2.3. Cyclic resistance ratio (CRR)7.5 versus relative density Dr, Carraro et al. (2003) [29]

Monkul and Kendir [30] performed cyclic direct simple shear test with Sile sand 20/30 and non-plastic SI silt for 0 percent, 15 percent and 25 percent FC under different relative densities. They found that cyclic resistance ratio of silty sand with 15 percent FC is higher than cyclic resistance ratio of clean sand for $D_r$<53 percent and after this $D_r$ value, cyclic resistance of clean sand becomes greater than silty sand with 15 percent FC with increasing relative density. For silty sand with 25 percent FC, cyclic resistance of clean sand is greater than the silty sand at all relative densities.

The change of cyclic resistance ratio with relative density at different fines contents was shown in Figure 2.4 [30]. As seen in Figure 2.4, CRR of silty sand decreases as FC increases from 15 percent to 25 percent.

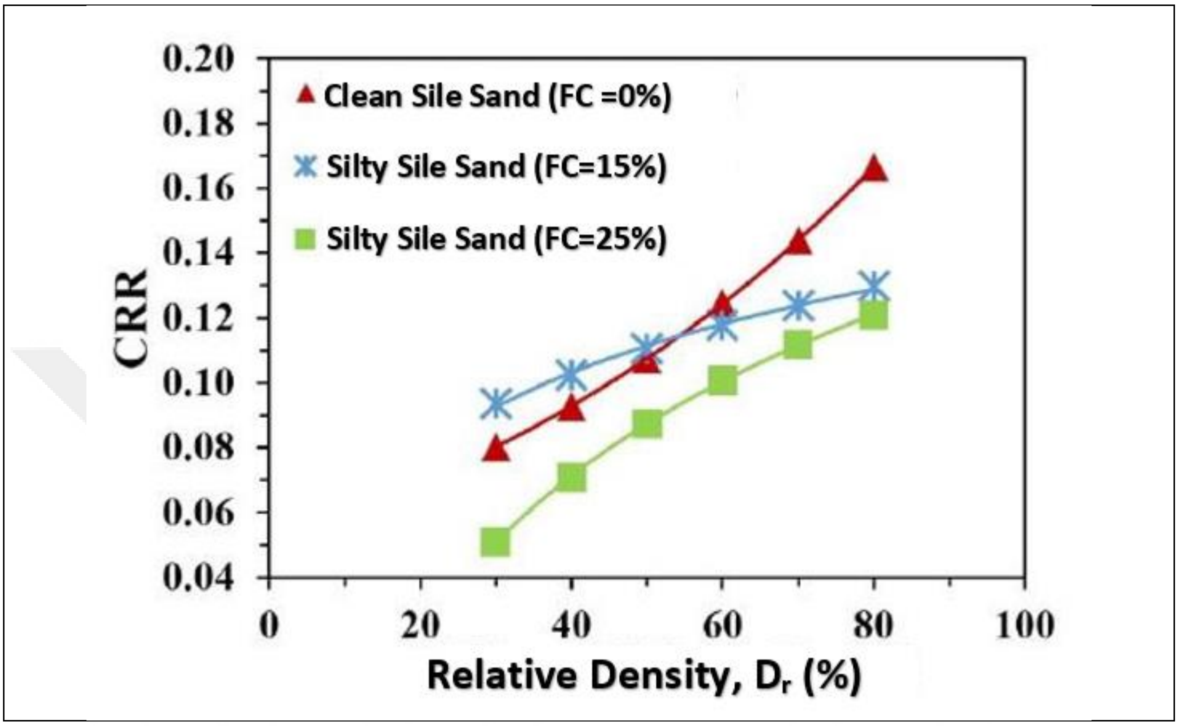


Figure 2.4. The change of cyclic resistance ratio (CRR) of Sile sand 20/30 with relative density ($D_r$) at different fines contents (FC), Monkul and Kendir (2019) [30]

### 2.2.2. Previous Studies Reporting a Decrease in Liquefaction Resistance with Silt Content

Cubrinovski et al. [18], Monkul and Yamamuro [31], Belkhatir et al. [32], Bayat and Bayat [33], Monkul et al. [34] and Oka et al. [35] reported that liquefaction resistance of sands decreases with increasing fines content at constant or similar relative density from monotonic and cyclic traxial compression tests. Wei et al. [36] performed cyclic traxial tests for 4 different sands and crushed silica silt in the range of 0-20 percent FC and they stated that cyclic resistance ratio of sands decreases with increasing silt content at the same void ratio.

Eseller-Bayat et al.[37] reported that clean sand has more resistant than silty sands with 5 and 10 percent FC at the same relative density from cyclic direct simple shear test. Similarly, Sönmezer [38] performed dynamic simple shear tests for silty sands 5 and 30 percent FC at the 2 different relative densities (30 and 50 percent) and reported that liquefaction resistance of sands decreases with increasing fines content for both relative densities.

Porcino et al. [39] performed cyclic direct simple shear tests between a void ratio range of 0.59-0.78 for silty sand with 10 percent FC and reported that clean sand has more resistance than silty sands at the same void ratio. In addition, Doygun et al. [40] found that shear strength of sands decreases with increasing fines content from 1 to 50 percent at constant void ratio (0.68).

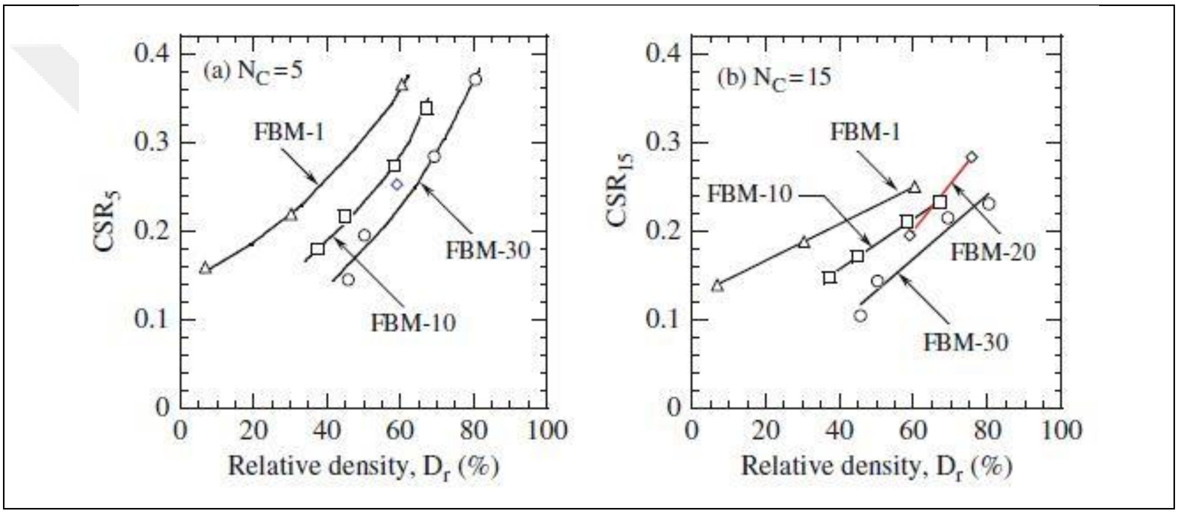


Figure 2.5. Cyclic stress ratios for FBM-soils required to cause 5% DA strain in 5 cycles (a) and 15 cycles (b) as a function of relative density, Cubrinovski et al. (2010) [18]

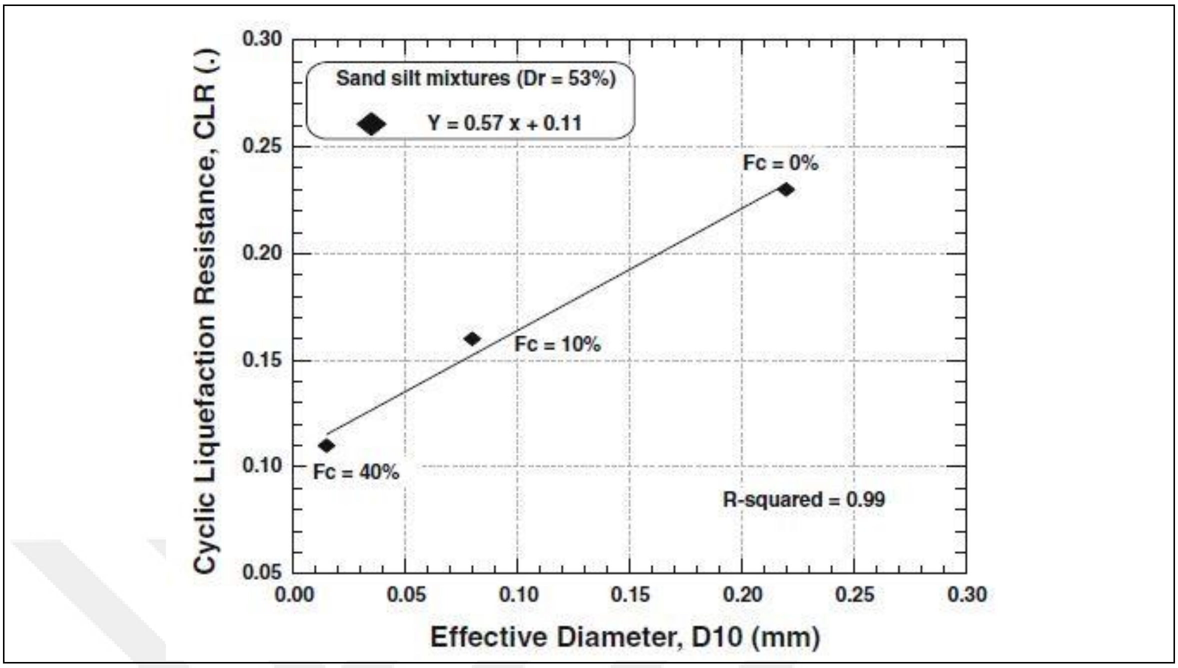


Figure 2.6. Cyclic liquefaction resistance versus effective diameter and fines content. ($\sigma_3' = 100\ kPa, D_r = 53\%$), Belkhatir et al. (2011) [32]

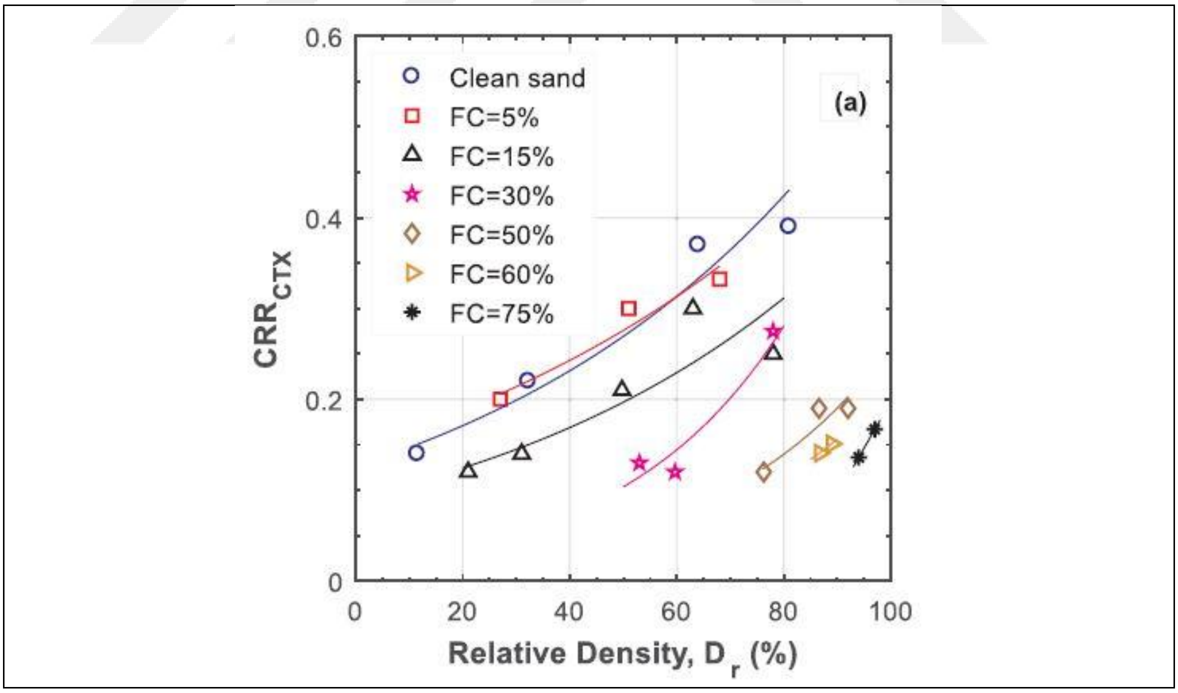


Figure 2.7. Effect of $D_r$ on $CRR_{CTX}$, Oka et al. (2018) [35]

Several researchers reported that cyclic resistance first decreased and then it increased with increasing silt contents. Singh [41], Sitharam and Dash [17] and Bahadori et al. [42] stated that liquefaction resistance of sands first decreases and then increases with increasing fines content at a constant relative density. The change of CRR with increasing fines content of silty sand specimens prepared at a constant relative density (50 percent) was shown in Figure 2.8 for mixtures of Flint Shot No. 4 sand and Alyeska Valdez silt [41]. In additon, the relationship between cyclic resistance and silt content both at constant gross void ratio (0.54) and relative density (55 percent) was shown in Figure 2.9 [17].

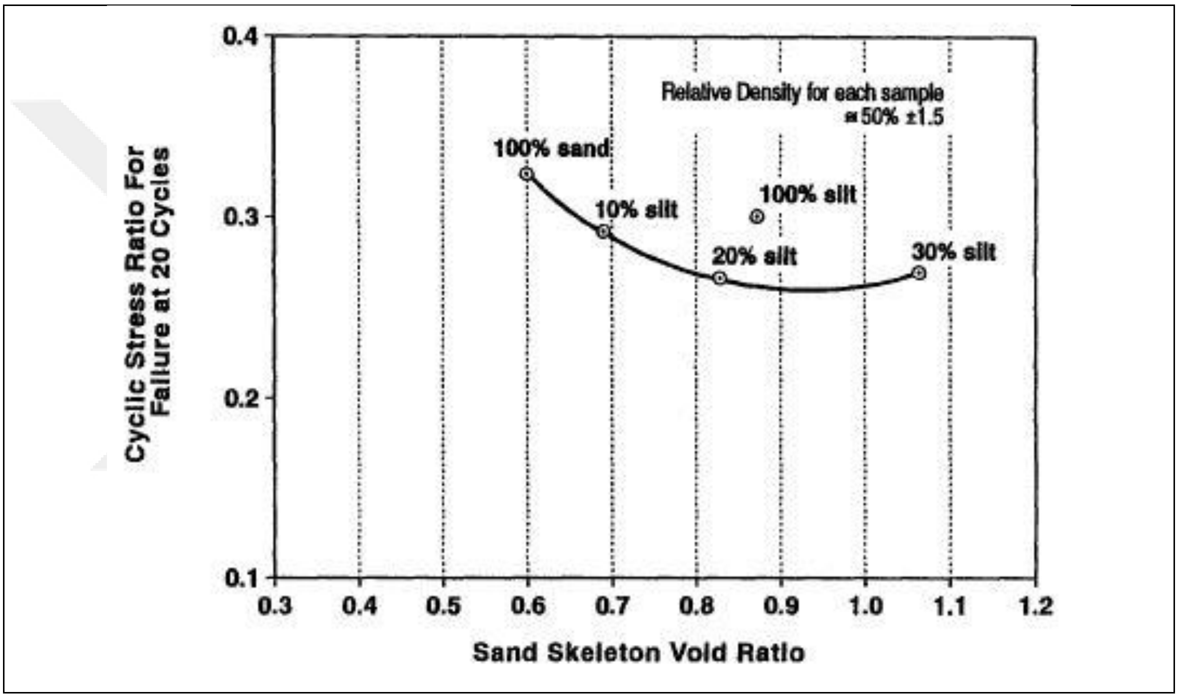


Figure 2.8. Relationship between CRR, FC and sand skeleton void ratio, Singh (1996) [41]

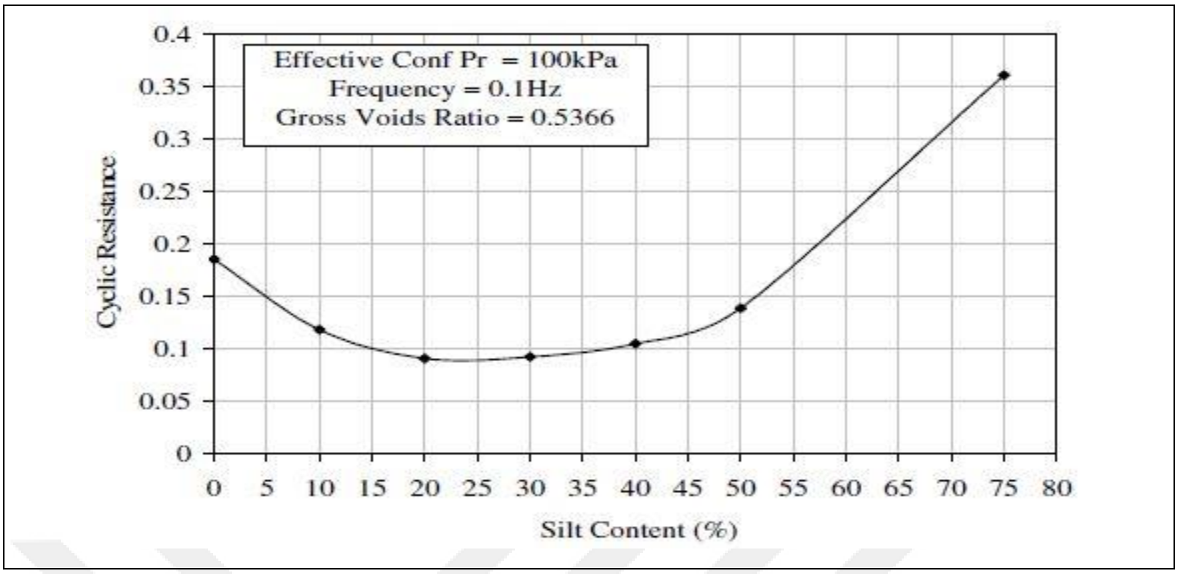


Figure 2.9. Cyclic resistance of Ahmedabad sand at constant gross void ratio vs. % silt content, Sitharam and Dash (2008) [17]

Polito and Martin [28], Thevanayagam et al. [43], Polito and Martin [44], Xenaki and Athanasopoulos [45], Papadopoulou and Tika [46] performed cyclic traxial test based on void ratio and they found that liquefaction resistance first decreased and then increased with increasing fines content at constant or similar void ratio.

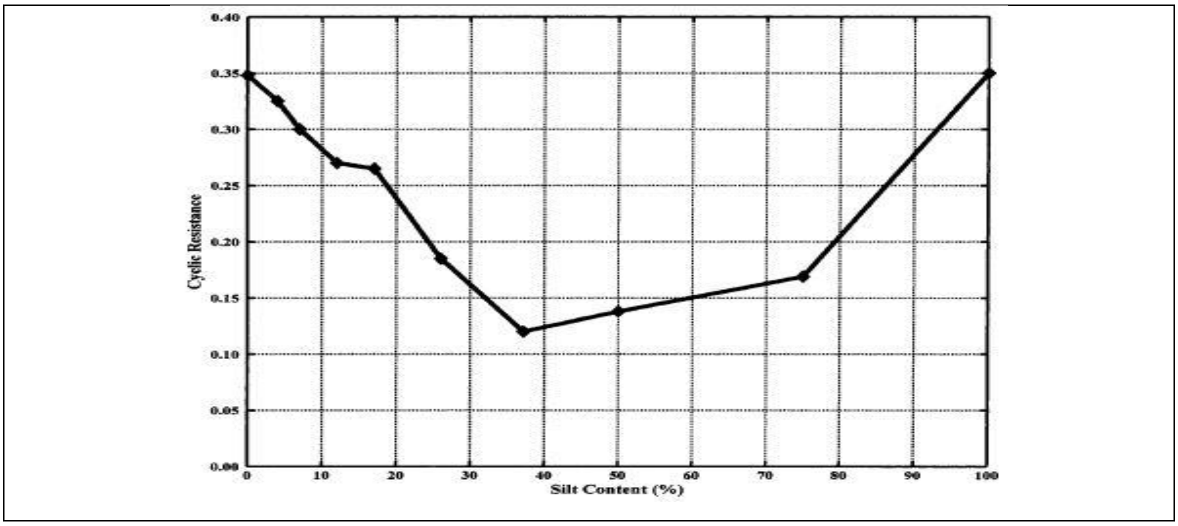


Figure 2.10. Variation in cyclic resistance for Yatesville sand and silt specimens prepared to a constant void ratio of 0.76, Polito and Martin (2003) [44]

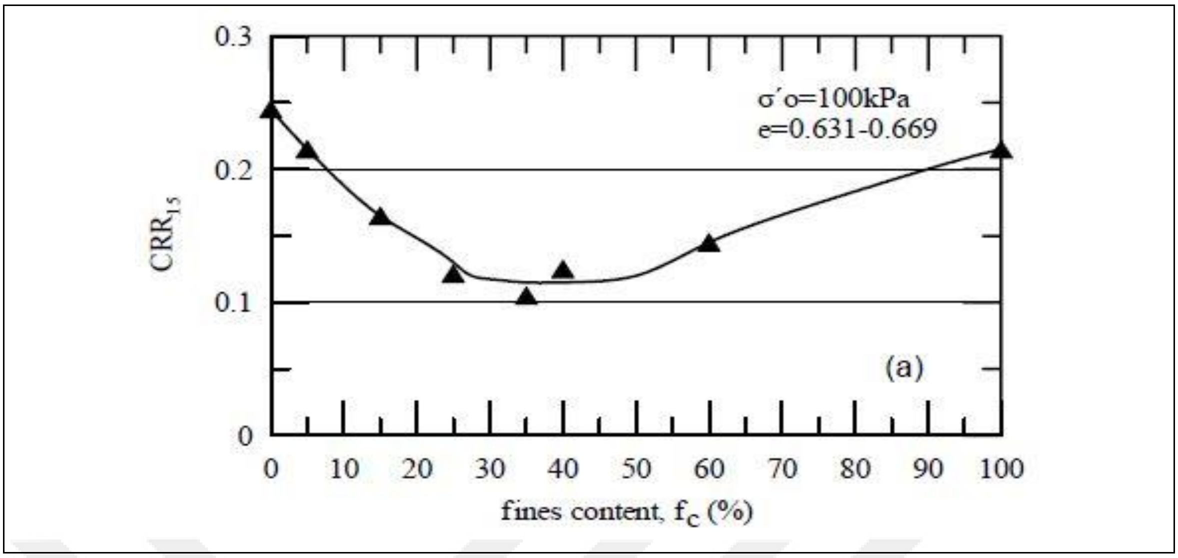


Figure 2.11. Effect of fines content, $f_c$ on liquefaction resistance, $CRR_{15}$, at constant void ratio, $e_c$, at $\sigma_o' = 100kPa$, Papadopoulou and Tika (2007) [46]

Porcino and Diano [47] performed cyclic simple shear test at the constant void ratio (0.68) and they reported that cyclic resistance ratio decreases with increasing fines content and then it increases after limiting fines content (LFC).

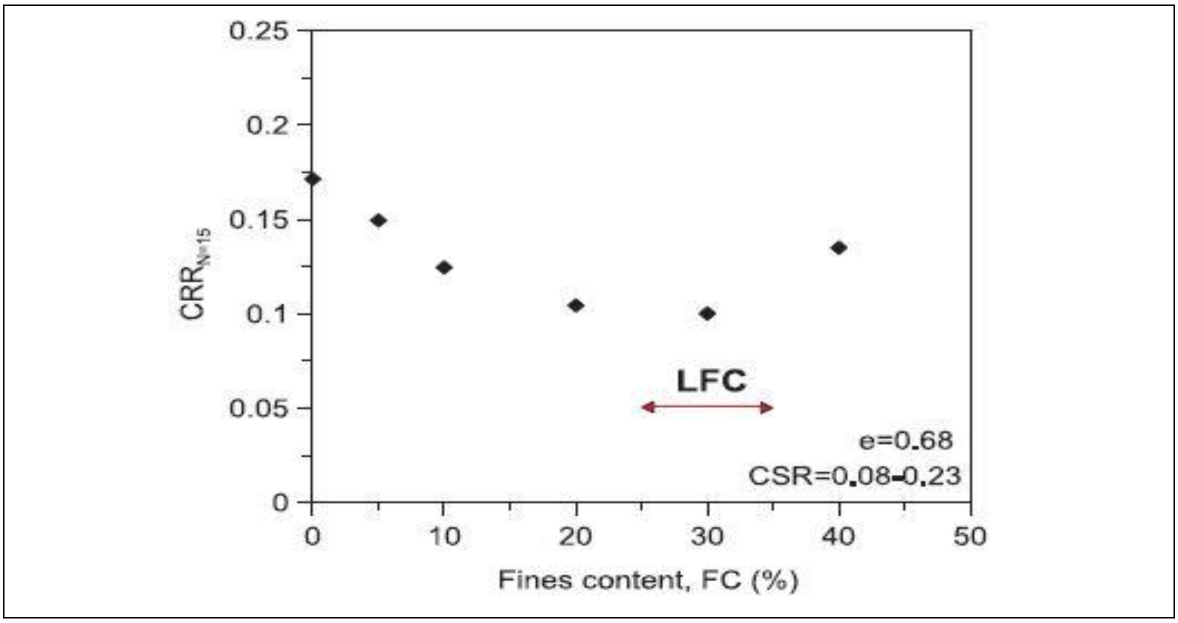


Figure 2.12. Relationship between cyclic resistance ratio at N = 15 loading cycles and fines content for LFC identification, Porcino and Diano (2017) [47]

Wang and Wang [48] investigated liquefaction resistance of silty sands between 5-45 percent FC from cyclic triaxial test and they found that cyclic resistance first decreased and then increase with increasing fines content at a constant dry density. Variation in liquefaction resistance with fines content was shown in Figure 2.13 [48].

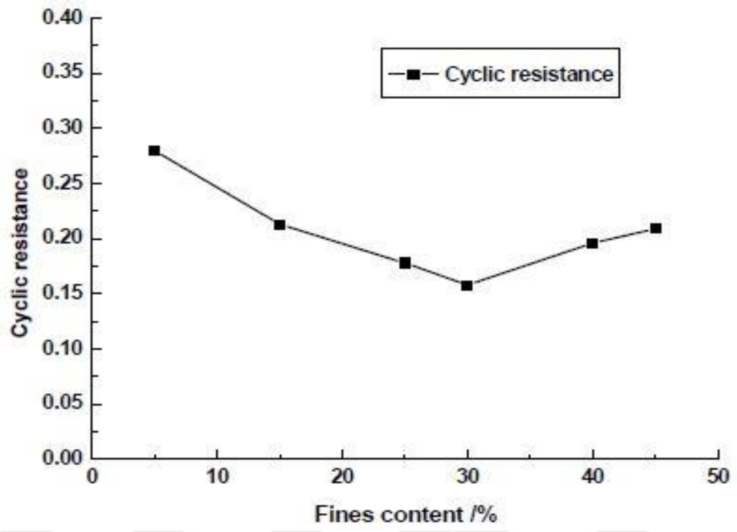


Figure 2.13. Variation in liqeufaction resistance with fines content for $N_L = 20$, Wang and Wang (2010), [48]

Kokusho [49] performed cyclic triaxial tests with specimens prepared at 3 different relative densities (30, 50 and 70 percent) and found that cyclic strength is nearly constant at 30 percent $D_r$ and liquefaction resistance decreases at 50 and 70 percent $D_r$ with incrasing fines content.

Karim and Alam [50, 51] reported that cyclic resistance ratio first decreases until limiting fines content (LFC) and then it becomes constant with increasing fines content at constant relative density (60 percent).

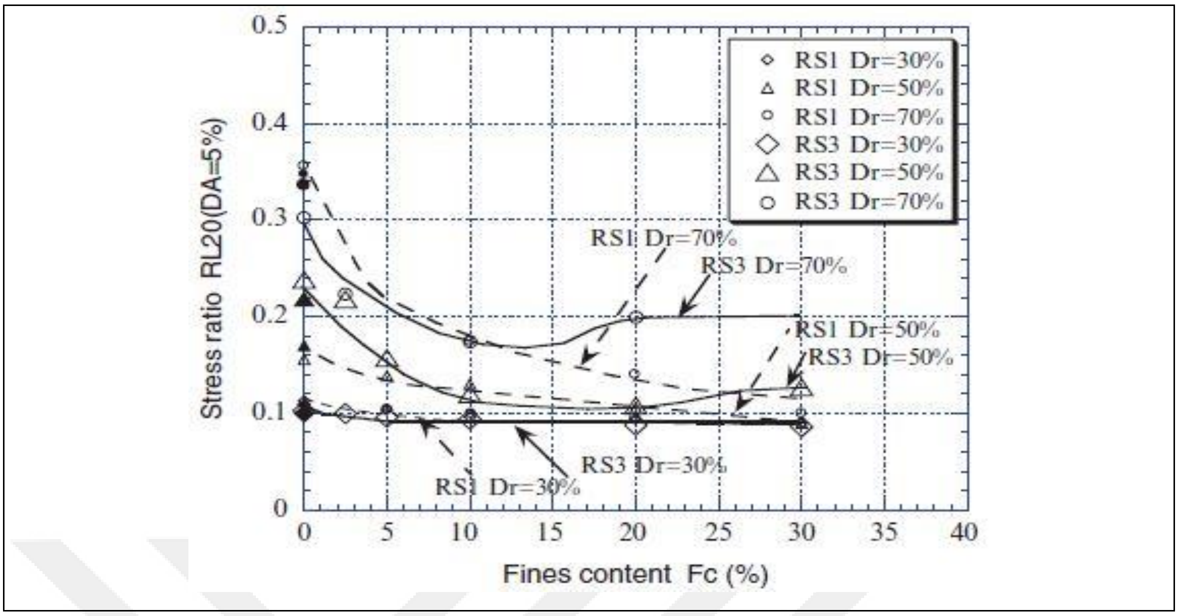


Figure 2.14. Liquefaction strength (stress ratio) versus fines content $F_c$ for RS1 and RS3 with different relative density $D_r$, Kokusho (2007) [49]

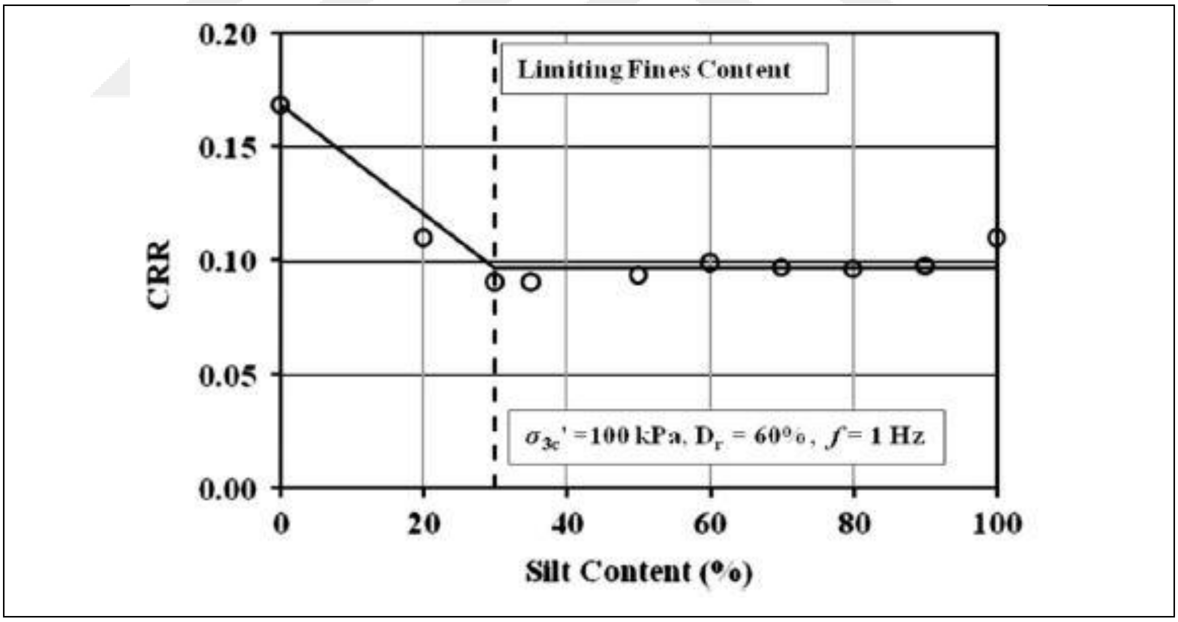


Figure 2.15. Variation of cyclic resistant ratio (CRR) with silt content, Karim and Alam (2014) [50]

The effects of silt content on liquefaction resistance of sands have been revealed as a result of studies performed by many researchers. When the studies are investigated, it has been clear that silt content could affect the liquefaction resistance of sands either positively or negatively. Moreover, there is no consensus about the silt effect on liquefaction resistance of sands (it could increase or decrease the cyclic liquefaction potential), which requires further research.

Table 2.2 includes summary of the studies which contain effects of fines content on liquefaction resistance of sands in the literature.

Table 2.2. Literature review for effects of FC on liquefaction resistance of sand

| References | Test Type | Sand Type | Silt Type | FC Range (%) | Relative Density (%) | Liquefaction Resistance (with increasing FC) |
|---|---|---|---|---|---|---|
| Singh (1996) [41] | Cyclic Traxial Test | Flint Shot No.4 Sand & Ottawa Sand | Non-Plastic Silt | 10, 20, 30 | 50 (constant) | decreases until 20% FC then increases |
| Salgado et al. (2000) [24] | Cyclic Traxial Test | Ottawa Sand | Non-Plastic Silt | 5, 10, 15, 20 | Different Dr (constant or similar) | increases |
| Amini & Qi (2000) [26] | Cyclic Traxial Test | Ottawa 20-30 Sand | Low Plasticity Silt | 10 - 50 | 40 (constant) | increases |
| Polito & Martin (2001) [28] | Cyclic Traxial Test | Monterey Sand | Yatesville Silt | 5, 10, 15, 20, 25, 35, 50, 75 | 0.68 (constant void ratio) | decreases until 35% FC then increases |
| | | Yatesville Sand | | 4, 7, 12, 17, 26, 37, 50, 75 | 0 - 100 | increases FC up to about 7% then decreases (Dr = 30 & 60%) |
| Thevanayagam et al. (2002) [43] | Cyclic Traxial Test | F55 Foundry Sand | Non-Plastic Silica Silt | 7, 15, 25, 40, 60, 100 | *At the same global void ratio | decreases until LFC = 25% then increases |
| Polito & Martin (2003) [44] | Cyclic Traxial Test | Monterey Sand | Yatesville Silt | 0 - 100 | 0.75 (constant sand skeleton void ratio) | constant until LFC = 26% |
| | | Yatesville Sand | | | 0.76 (constant void ratio) | decreases until 36% FC then increases |
| Carraro et al. (2003) [29] | Cyclic Traxial Test | Ottawa Sand | Non-Plastic Silt | 5, 10, 15 | 40 - 80 | increases until 5% FC then decreases |

Table 2.2 (continue). Literature review for effects of FC on liquefaction resistance of sand

| References | Test Type | Sand Type | Silt Type | FC Range (%) | Relative Density (%) | Liquefaction Resistance (with increasing FC) |
|---|---|---|---|---|---|---|
| Xenaki & Athanasopoulos (2003) [45] | Cyclic Traxial Test | Medium Sand | Non-Plastic Silt | 10, 30, 42 55, 100 | *At the same void ratio | decreases until LFC = 44% then increases |
| Kokusho (2007) [49] | Cyclic Traxial Test | RS1, RS2 and RS3 Sand | Non-Plastic Silt | 5, 10, 20, 30 | 30, 50 and 70 | nearly constant at Dr 30% decreases at Dr 50 & 70% |
| Papadopoulou & Tika (2007) [46] | Cyclic Traxial Test | Clean Quartz Sand | Non-Plastic Silt (Natural Quartz Deposits) | 5, 15, 25, 35, 40, 60, 100 | *At the same void ratio<br>*At constant intergranular void ratio | decreases until LFC = 30-35% then increases<br>increases |
| Sitharam & Dash (2008) [17] | Cyclic Traxial Test | Ahmedabad Sand | Non-Plastic Silt | 10, 20, 30, 40,50, 75 | 55 (constant) | decreases until LSC = 26% then increases |
| Bahadori et al. (2008) [42] | Hollow Cylindrical Torsional Shear | Firoozkuh Sand | Non-Plastic Firoozkuh Silt | 15, 30, 70 | 42 - 78 | decreases at FC = 15 & 30% increases at FC = 70% |
| Cubrinovski et al. (2010) [18] | Cyclic Undrained Triaxial Test | Fitzgerald Bridge Mixes (FBM) | Non-Plastic Silt | 1, 10, 20, 30 | 3 different Dr | decreases (at a given Dr) |
| Wang & Wang (2010) [48] | Cyclic Traxial Compression Test | Skeleton Sand (Pure Sand) | Non-Plastic Silt | 5 - 45 | 30 - 70 *(at a constant dry density) | decreases until 30% FC then increases |

Table 2.2 (continue). Literature review for effects of FC on liquefaction resistance of sand

| References | Test Type | Sand Type | Silt Type | FC Range (%) | Relative Density (%) | Liquefaction Resistance (with increasing FC) |
|---|---|---|---|---|---|---|
| Monkul & Yamamuro (2011) [31] | Cyclic Undrained Triaxial Test | Nevada Sand-B | Loch Raven SilCoSil 125 Postsdam Non-Plastic Silt | 0 - 20 | At the same Dr | decreases |
| Belkhatir et al. (2011) [32] | Cyclic Traxial Test | Chlet Sand | Non-Plastic Silt | 0 - 40 | 20, 53 and 91 (at constant Dr = 53%) | decreases |
| Bayat & Bayat (2013) [33] | Cyclic Traxial Test | Clean Sand (No.10 - No.200) | Non-Plastic Silt | 5, 10, 15, 20 | At the same Dr | decreases |
| Karim & Alam (2014) [50] | Cyclic Traxial Stress-Control Test | Padma River (Fine Sand) | Non-Plastic Silt | 0 - 100 | 60 (at a constant Dr) | decreases until LSC = 30% then becomes constant |
| Takch et al. (2016) [52] | Cyclic-Monotonic Shear Test | Ottawa Sand | Non-Plastic Silt (Quartz Particles) | 50, 75, 100 | 13 - 55 *(at the same void ratio) | increases |
| Monkul et al. (2017) [34] | Undrained Monotonic Traxial Test | Sile Sand 20/30 Sile Sand 50/55 Sile Sand 80/100 | IZ Silt SI Silt TT Silt | 5, 15, 25 | Different Dr | decreases (at similar Dr) |
| Karim & Alam (2017) [51] | Strain-Controlled Monotonic Traxial Test | Padma River (Fine Sand) | Non-Plastic Silt | 0 - 100 | 30, 60 and 78 (at constant Dr) | decreases until LFC = 30% then remains constant |

Table 2.2 (continue). Literature review for effects of FC on liquefaction resistance of sand

| References | Test Type | Sand Type | Silt Type | FC Range (%) | Relative Density (%) | Liquefaction Resistance (with increasing FC) |
|---|---|---|---|---|---|---|
| Oka et al. (2018) [35] | Cyclic Triaxial Test | F-75 Sand (Silica Quartz Grained) | Non-Plastic Silt (Sil-co-sil 125 silica quartz) | 5, 15, 30, 50, 75 | Different Dr (constant) | decreases |
| Jradi et al. (2020) [27] | Cyclic Traxial Test | Fontainebleau Sand | Non-Plastic Silt | 1, 3, 5 | 2 different Dr (Loose and Medium Dense) | increases |
| Wei et al. (2020) [36] | Cyclic Traxial Test | 4 different Sand (Toyoura, Fujian) Ottawa (20-30 / 50-70) | Crushed Silica Silt | 0 - 20 | *At the same void ratio | decreases |
| Hazirbaba & Rathje (2009) [25] | Cyclic Simple Shear Test | Monterey #0/30 Sand | Sil-Co-Sil 52 Non-Plastic Silt | 5, 10, 15, 20 | 50 (constant) | increases |
| Porcino & Diano (2017) [47] | Cyclic Direct Simple Shear Test | Ticino Sand | Non-Plastic Silt | 5, 10, 20 30, 40 | *At the same void ratio | LFC = 25-35% decreases until LFC then increases |
| Porcino et al. (2018) [39] | Cyclic Direct Simple Shear Test | Ticino Sand | Non-Plastic Silt (TS10) | 10 | 0.59 - 0.78 (void ratio range) | decreases (at the same void ratio) |
| Eseller-Bayat et al. (2019) [37] | Cyclic Direct Simple Shear Test | Sile Sand 20/55 | Non-Plastic Silt | 5, 10 | 30, 40 and 50 | decreases until 5% FC then remains constant |

Table 2.2 (continue). Literature review for effects of FC on liquefaction resistance of sand

| **References** | **Test Type** | **Sand Type** | **Silt Type** | **FC Range (%)** | **Relative Density (%)** | **Liquefaction Resistance (with increasing FC)** |
|---|---|---|---|---|---|---|
| Monkul & Kendir (2019) [30] | Cyclic Direct Simple Shear Test | Sile Sand 20/30 | Non-Plastic SI Silt | 15, 25 | 30 - 80 | increases at FC = 15% (For Dr<53%) decreases at FC = 15% (for Dr>53%)<br><br>decreases at FC = 25% |
| Doygun et al. (2019) [40] | Cyclic Direct Simple Shear Test | Silica Sand (Pure quarried) (N-Series and W-Series) | Non-Plastic Silt | 1, 15, 50 | 0.68 (constant void ratio) | decreases |
| Sönmezer (2020) [38] | Dynamic Simple Shear Test | Sand 20/30<br>Sand 30/50 | Non-Plastic Silt | 5, 30 | 30 and 50 | decreases |

# 3. CHARACTERIZATION OF TESTED SOILS

The soils which are clean silica sand and non-plastic silt used in experimental studies were obtained from the city of Izmir in Turkey. Clean silica sand was used as the base sand and silt was used as the fine grain matrix. Silt was named as YET silt which is non-plastic and its liquid limit (LL) is 30 percent, and but only the -No 200 portion ( $< 0.075$mm), obtained by wet sieving, was used in the experimental studies. Experiments to determine the properties of soils were performed at the Yeditepe University Soil Mechanics Laboratory. Specific gravities ($G_s$) for sand and silt were found 2.641 and 2.644 respectively by using ASTM standards [53]. Sieve analysis was performed for base sand and hydrometer test was performed for YET silt according to ASTM standards [54] and their grain size distribution curves were shown in Figure 3.1 and Figure 3.2. Also, silty sands were obtained by mixing the base sand with YET silt at various fines contents. The grain size distribution curves for silty sands with 5, 15 and 35 percent fines contents are plotted in Figure 3.3. Coefficient of uniformities ($C_u$), coefficient of curvatures ($C_c$), grain diameters ($D_{10}$, $D_{30}$ and $D_{60}$) and Unified Soil Classification System (USCS) group symbols [55] of the silica sand and YET silt were shown in Table 3.1.

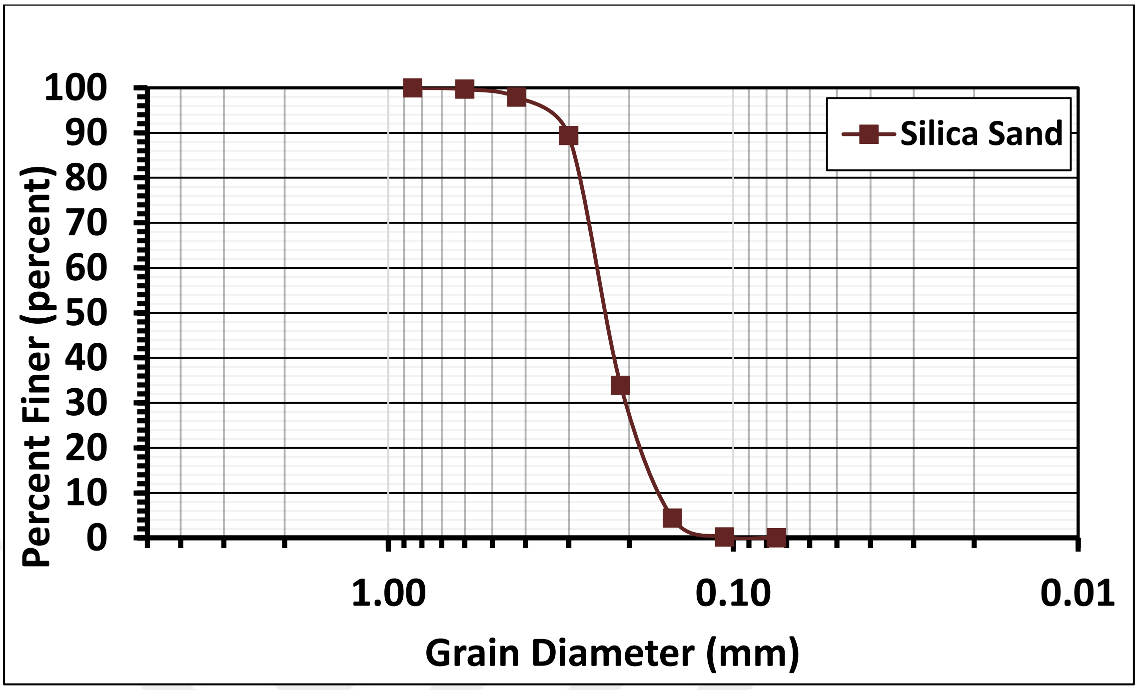


Figure 3.1. Grain size distribution of the silica sand

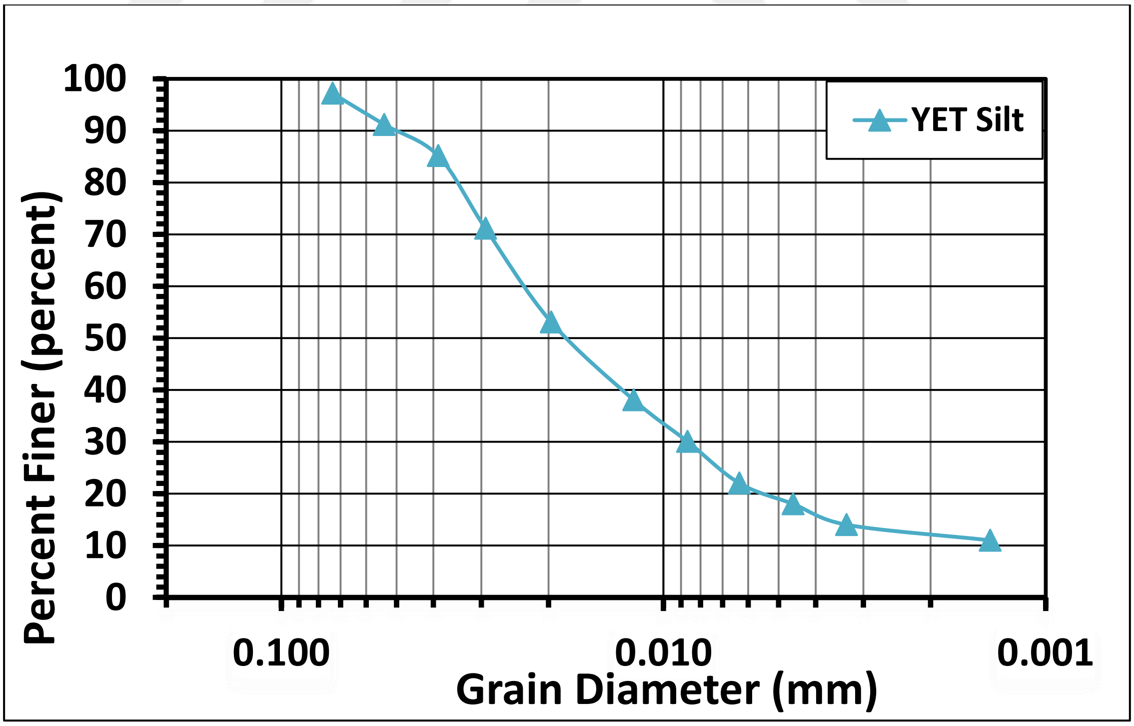


Figure 3.2. Grain size distribution of the YET silt

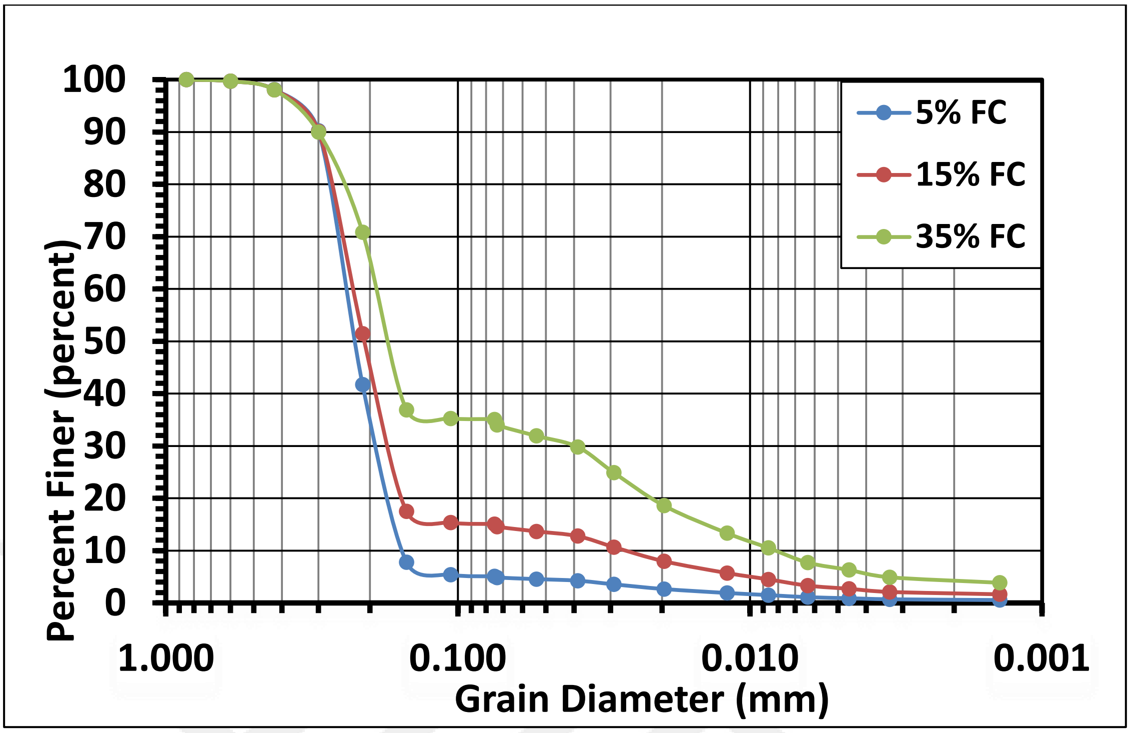


Figure 3.3. Grain size distribution of silty sands with 5, 15 and 35% fine contents

Table 3.1. Some properties of the soils used in the study

| | Silica Sand | 5% FC | 15% FC | 35% FC | YET Silt |
|---|---|---|---|---|---|
| $G_s$ | 2.641 | 2.641 | 2.641 | 2.642 | 2.644 |
| $e_{min}$ | 0.717 | 0.632 | 0.543 | 0.457 | 0.832 |
| $e_{max}$ | 1.039 | 0.990 | 0.902 | 0.833 | 1.471 |
| $D_{10}$ | 0.165 | 0.155 | 0.026 | 0.008 | 0.001 |
| $D_{30}$ | 0.205 | 0.190 | 0.175 | 0.040 | 0.009 |
| $D_{60}$ | 0.247 | 0.240 | 0.227 | 0.190 | 0.024 |
| $C_u$ | 1.50 | 1.55 | 8.73 | 23.75 | 16.79 |
| $C_c$ | 1.03 | 0.97 | 5.20 | 1.05 | 2.30 |
| USCS symbol | SP | SM | SM | SM | ML |

And also, minimum ($e_{min}$) and maximum ($e_{max}$) void ratios for clean sand (FC = 0 percent), YET silt (FC = 100 percent) and three fines contents (FC = 5, 15 and 35 percent) were determined by using method of Lade et al. [56] and shown in Figure 3.4. There are various methods for determination of minimum and maximum void ratios such as ASTM D4253

[57], ASTM D4254 [58] and Japanese standard [59], but these methods contained some restrictions, for instance, they were not feasible for the studied FC values [38, 44, 45, 52, 60].

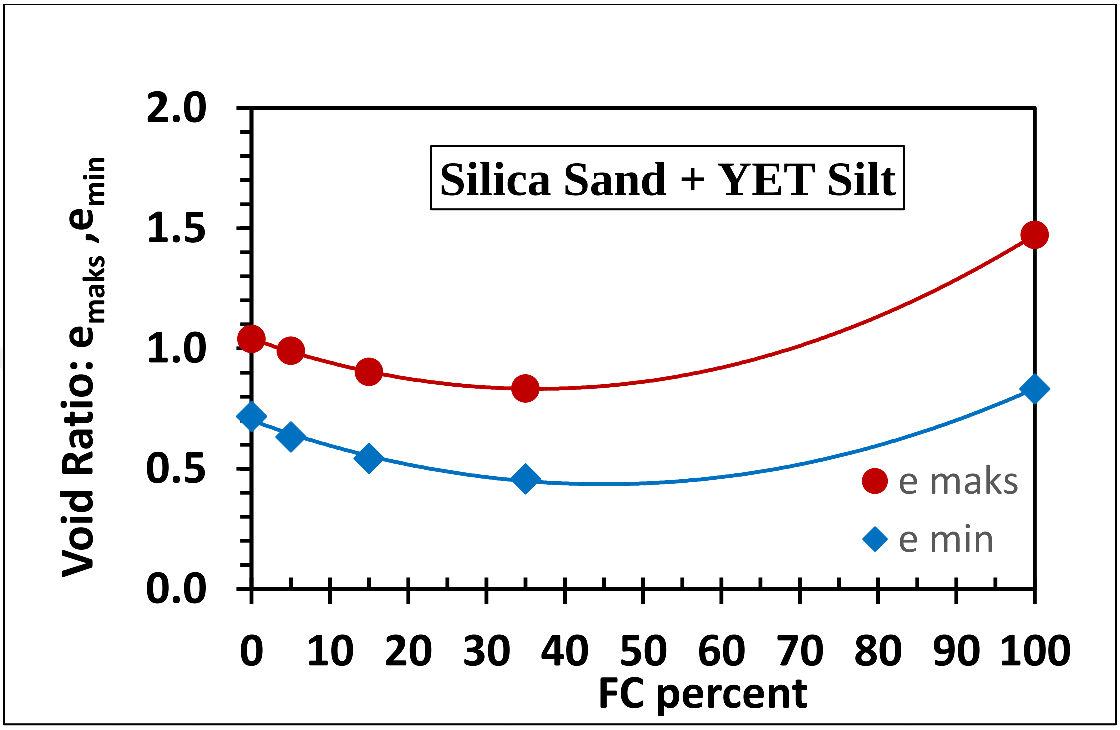


Figure 3.4. The changes of minimum and maximum void ratios with FC obtained by mixed with YET Silt in various ratios

When we analyze the Figure 3.4, as FC increased until about 35 percent, the minimum and maximum void ratios decreased. After this value, minimum and maximum void ratios started to increase till FC equals 100 percent. Monkul [61] reported that this is due to positional tendencies of the silts which have 2 main subject such that they would enter the void space in between adjacent sand grains or they would enter between sand grains by pushing them apart.

In addition, Monkul [61] explained the reason for the observed trend for FC $\leq$ 35 percent, as the silts primarily fill the gaps between the coarse grains formed by the sand matrix. In the increasing fines contents (FC $\geq$35 percent), silt matrix gradually opened the sand grains since it could not fit into the gaps between coarse grains so that the sand matrix became distincly floating in the silt matrix [61].

# 4. CYCLIC DIRECT SIMPLE SHEAR TEST

## 4.1. GENERAL CHARACTERISTICS OF DIRECT SIMPLE SHEAR TESTING

The simple shear testing appeared in the UK and Sweden in the first half of the 1950s and was used to investigate the stress-strain behavior of clayey soils [60, 62]. It has been used for liquefaction studies of sand under dynamic loads since the end of the 1960's [62]. In our country, liquefaction studies based on laboratory experiments have been performed since the 1980s [62].

In today's simple shear devices, undrained and drained constant volume variants are used to represent the undrained situation [60]. Dyvik et al. [63] have shown that both variants provide the same stress-strain behavior in both variants as a result of the static simple shear tests with normally consolidated clay. Bjerrum and Landva [64] stated that the change of vertical stress required for constant volume control was equal to the excessive pore water pressure in undrained test ($\Delta\sigma_v = -\Delta u$) and it was proved by Dyvik et al. [63].

Finn and Vaid [65] have reported that liquefaction can be determined by completely dry samples in their study with sand since the pore water pressure can be calculated using vertical stress. Monkul et al. [60] compared saturated and dry samples for clean, silty and clayey sands with constant volume controlled dynamic simple shear tests, and showed that liquefaction resistance can also be determined with completely dry samples for clean and silty sands, but not for clayey sands. For this reason, liquefaction tests were performed with dry sand and silts in this study.

## 4.2. SPECIMEN PREPARATION WITH AUTOMATIC DRY FUNNEL DEPOSITION TECHNIQUE

Proper preparation of samples for each experiment is extremely important in terms of the accurate and consistent result of the test. In this study, dry funnel method was chosen as a specimen preparation method. In the dry funnel method, the soil was mixed homogeneously in a container. The clean and silty sand which is prepared for the test was poured into a funnel that is placed on a special mold base and the funnel was raised along the vertical axis and test samples were created. Meanwhile, not only the mold provides that the teflon rings fit properly, but also provides that the membrane is tightened to the mold.

## 4.3. CYCLIC DIRECT SIMPLE SHEAR APPARATUS AT YEDITEPE UNIVERSITY

Cyclic direct simple shear (CDSS) tests which were carried out in soil mechanics laboratory of Yeditepe University were performed with the GeoComp ShearTrack II-DSS (NGI type) device shown in Figure 4.1.


Figure 4.1. ShearTrack II-DSS test device

Although the cyclic triaxial testing experiment is apparently more used by different institutions (perhaps because of the availability of the cyclic triaxial equipment), the cyclic loading mechanism in the cyclic direct simple shear test represents earthquake loading conditions better than cyclic triaxial testing [60]. Due to this reason, CDSS was preferred in this study.

In this section, the working principle of CDSS test was explained. CDSS apparatus at Yeditepe University Soil Mechanics Laboratory was explained. CDSS consists of 2 main steps. These are the consolidation step and the cyclic simple shear step. Before these steps begin, testing parameters such as how many grams of sample were used, specific gravity of the sample, target cyclic stress ratio were entered into the interface of "shear.cyclic" programme shown in Figure 4.2.

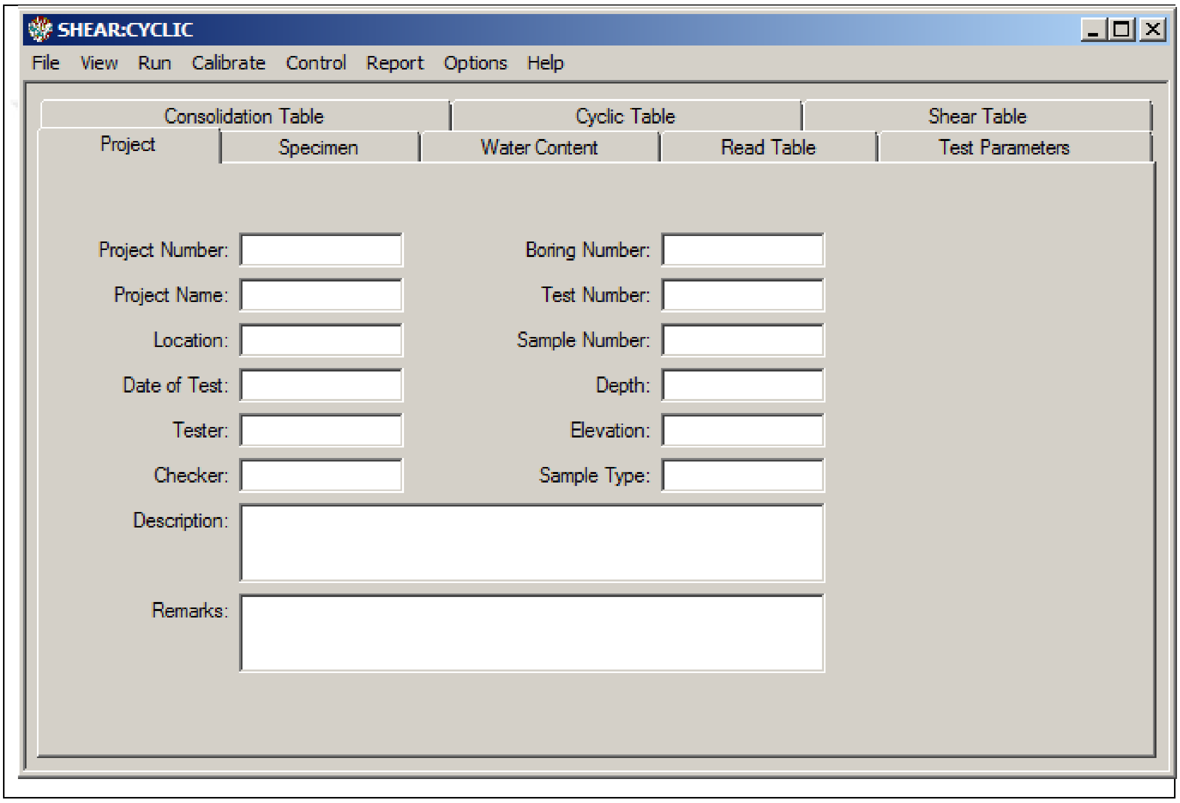


Figure 4.2. Interface of "shear.cyclic" programme

### 4.3.1. Consolidation Step

The consolidation step is the first part of the experiment. Samples prepared throughout all tests which have approximately diameter of 64 mm and height of 20 mm, were consolidated under 100 kPa vertical stress ($\sigma_{vc}$). When the consolidation is completed under that vertical stress in a short time (set to a minimum of 5 minutes), the consolidation step ends. The relationship between vertical displacement and time was shown in Figure 4.3. The second step, which is the cyclic shear step, starts automatically.

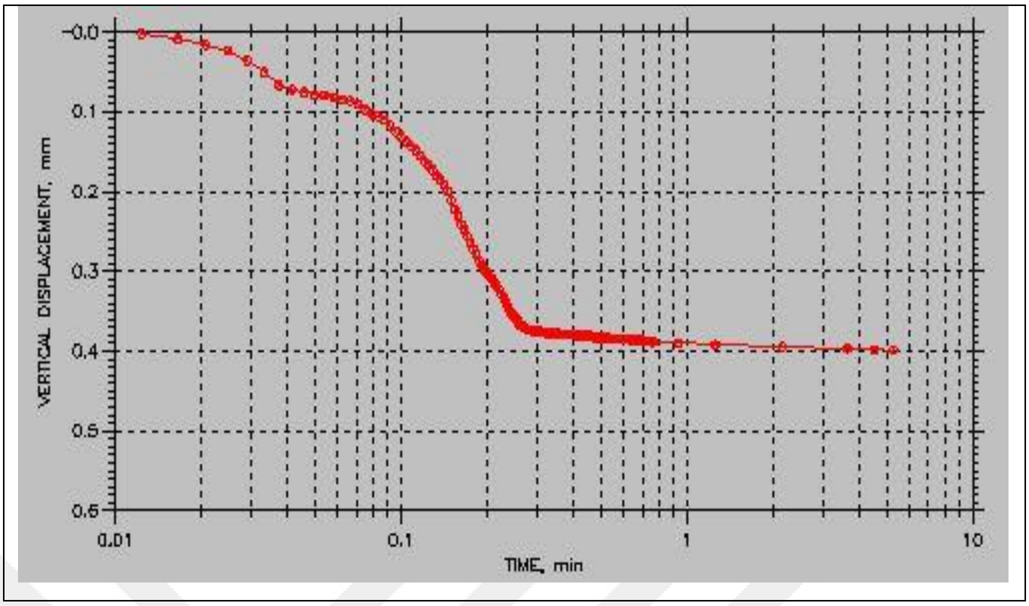


Figure 4.3. Vertical displacement vs. time

### 4.3.2. Cyclic Simple Shear Step

During the cyclic shear step, the samples were exposed to repeated shear stresses at a frequency of 0.1 Hz in the sinuous corresponding to 3 different cyclic stress ratios (CSR = 0.1, 0.12 and 0.14). The criterion which is required for liquefaction to occur and the experiment to end automatically was determined by when the excess pore water pressure ratio ($r_u$) reaches 1 or peak to peak shear strain reaches 10 percent in this study. However, in some cases, even if the excess pore pressure ratio reaches 1, the experiment continues until peak to peak shear strain reaches 10 percent. In this case, after the experiment completed, the value at which the excess pore pressure ratio is equal 1 was used as the required number of cycles to occur liquefaction ($N_L$).

# 5. EXPERIMENTAL RESULTS AND DISCUSSIONS

In this chapter of the study, the effects of silty sands on the liquefaction behavior of sands were analyzed in detail. 123 CDSS tests were performed in total at the Soil Mechanics Laboratory of Yeditepe University and distributions of these tests were shown in Table 5.1.

Table 5.1. Distributions of 123 CDSS tests

| **Soil Type** | **CSR = 0.1** | **CSR = 0.12** | **CSR = 0.14** | **Sum** |
|---|---|---|---|---|
| Clean Sand (FC = 0%) | 16 | 12 | 18 | 46 |
| Silty Sand (FC = 5%) | 12 | 6 | 11 | 29 |
| Silty Sand (FC = 15%) | 10 | 7 | 8 | 25 |
| Silty Sand (FC = 35%) | 7 | 10 | 6 | 23 |
| **Sum** | 45 | 35 | 43 | 123 |

Cyclic tests which were performed for clean sand and 3 different fines contents (FC = 5, 15 and 35 percent) under 3 different cyclis stress ratios (CSR = 0.1, 0.12 and 0.14) were examined in this chapter of the study in terms of the relationship between the number of cycles ($N_L$) required for liquefaction with relative density ($D_r$).

## 5.1. THE RELATIONSHIP BETWEEN RELATIVE DENSITY AND NUMBER OF CYCLE FOR CLEAN SAND

For clean sand, 46 cyclic tests were performed under 3 different CSRs and they were shown in Figure 5.1, Figure 5.2, Figure 5.3 and also all 3 CSRs at the same graph was shown in Figure 5.4.

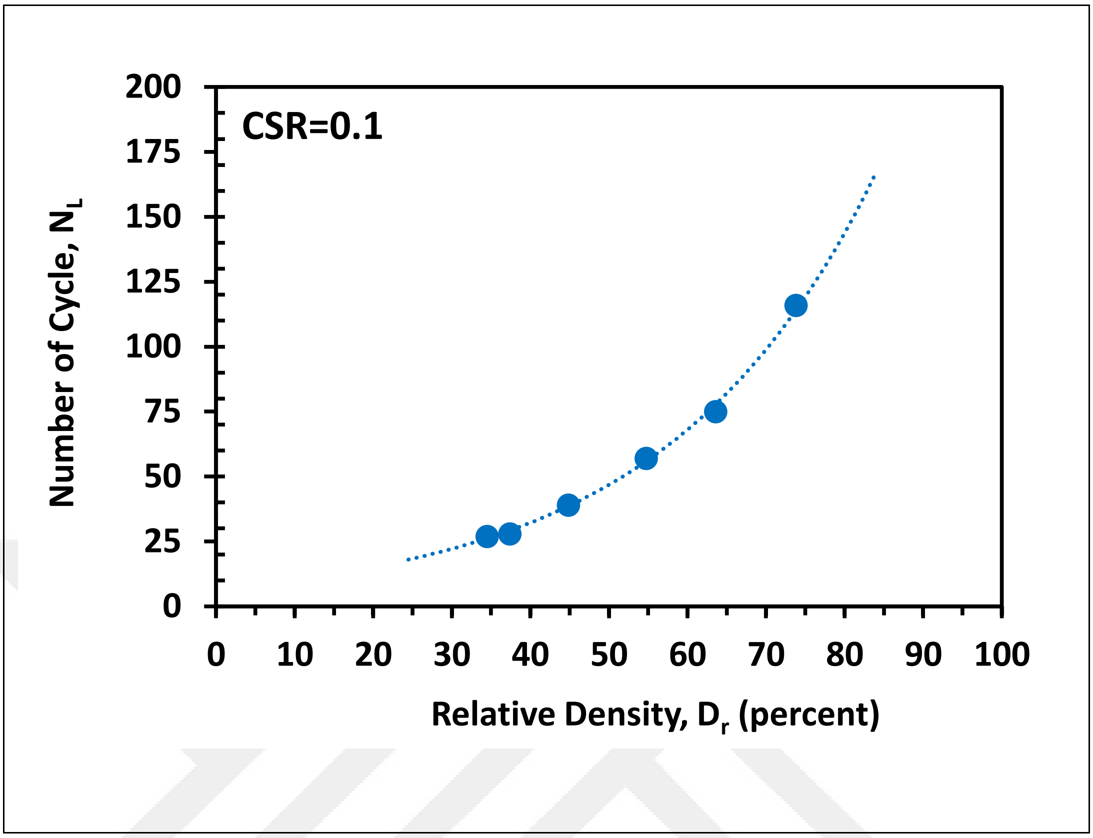


Figure 5.1. $D_r$ vs. $N_L$ for clean sand (FC = 0%) under CSR = 0.1

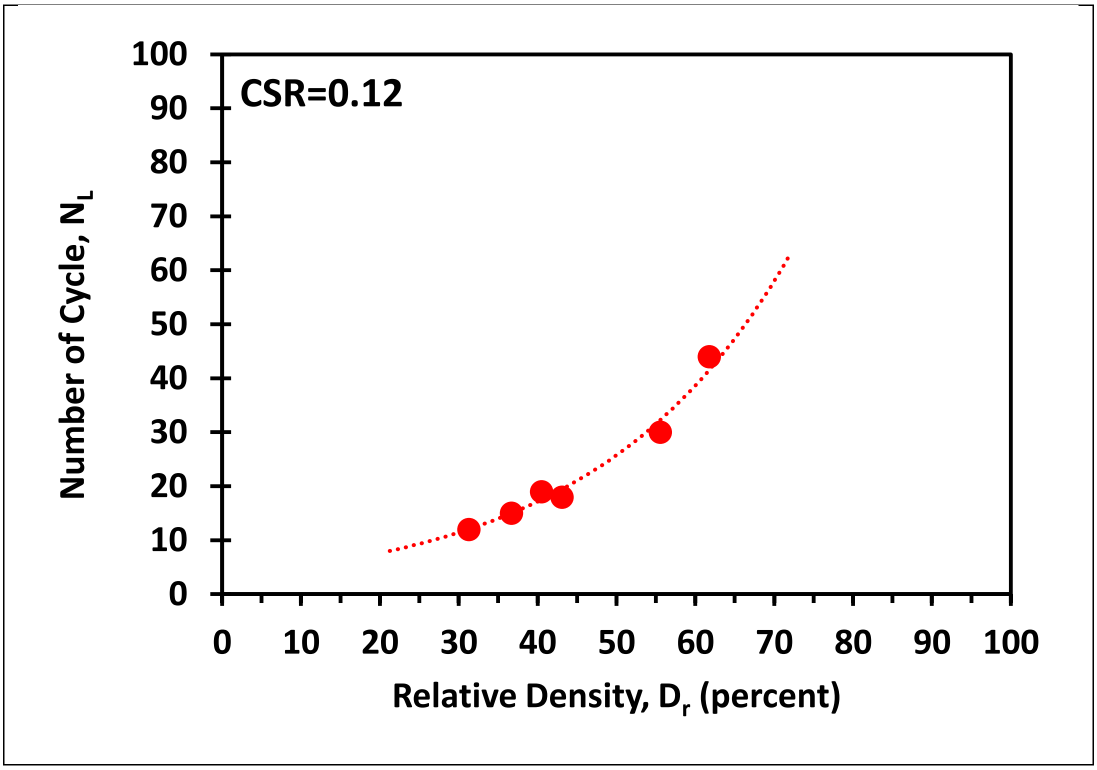


Figure 5.2. $D_r$ vs. $N_L$ for clean sand (FC = 0%) under CSR = 0.12

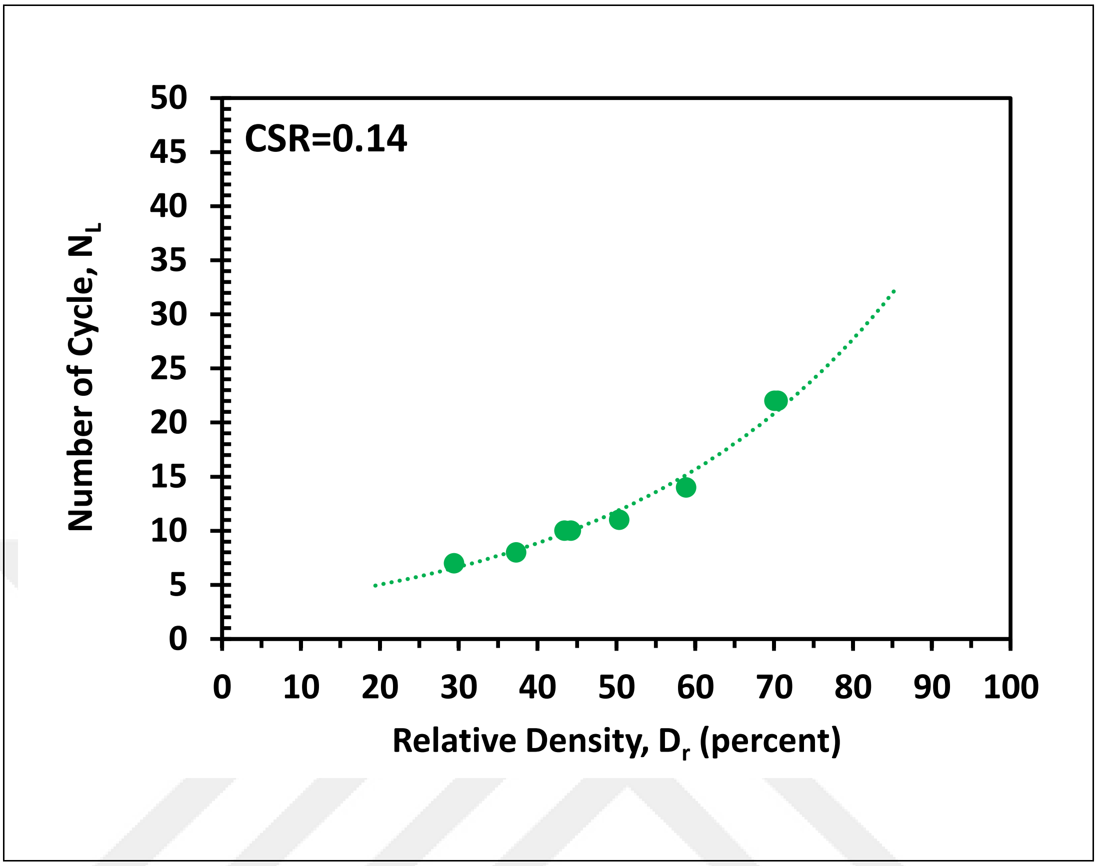


Figure 5.3. Dr vs. NL for clean sand (FC = 0%) under CSR = 0.14

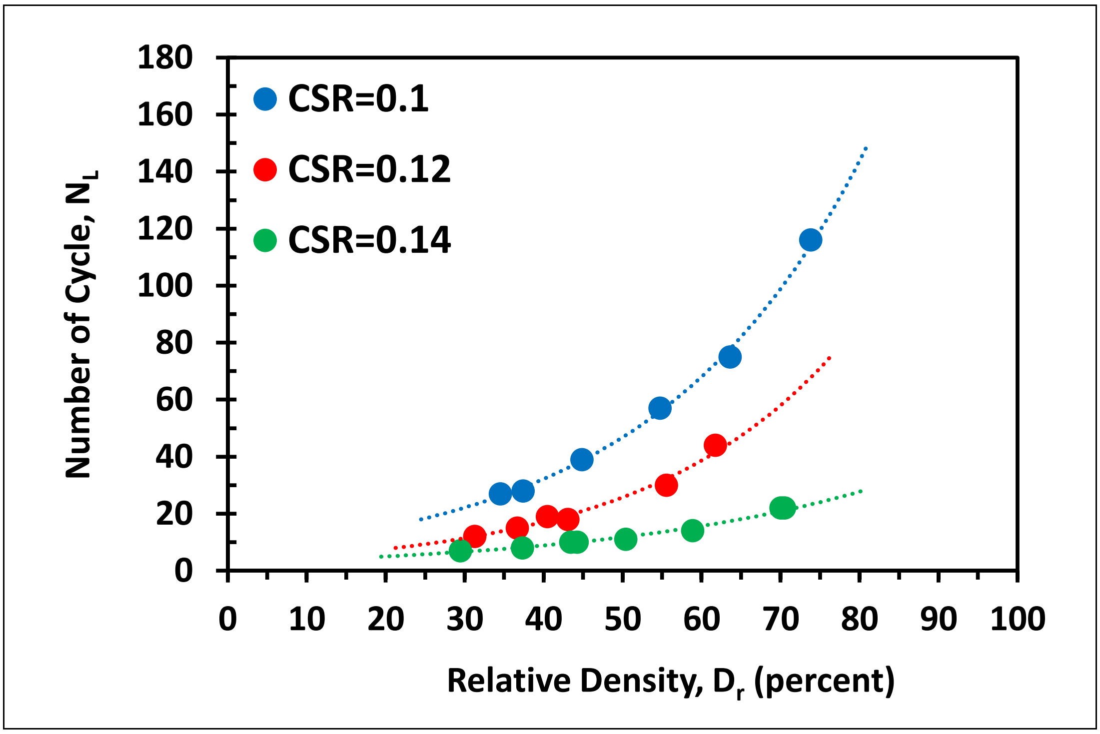


Figure 5.4. Dr vs. $N_L$ for clean sand (FC = 0%) under CSR = 0.1, 0.12 and 0.14

When the Figure 5.4 was analyzed, it was easily seen that as the relative density increases, the number of cycles required for liquefaction increases. At the same time, as the relative density increases, the distances between all 3 trend curves gradually increases.

Monkul and Kendir [30], and Yenigün [66] stated that number of cycles required for liquefaction and relative density are expressed with an exponential function in constant CSRs in their studies. In this study, the exponential relationship between number of cycles required for liquefaction and relative density was shown in equation 5.1.

$$N_L = a.e^{[b.D_r(\%)]} \tag{5.1}$$

The coefficients a and b affect the number of cycles required for liquefaction, while e represents the Euler number which is 2.718. Monkul and Kendir [30] reported that the coefficient a is important for the beginning value of the equation. When the relative density is equal zero, the coefficient a indicates the number of cycle required for liquefaction [30].

## 5.2. THE RELATIONSHIP BETWEEN RELATIVE DENSITY AND NUMBER OF CYCLE FOR SILTY SANDS

For silty sands, 77 cyclic tests were performed under 3 different CSRs for 3 different fines contents. The exponential function which was shown in equation 5.1 was also used for silty sands as well as clean sand. Similar to clean sand, the number of cycles required for liquefaction of silty sands increased as the relative density increased. Graphs plotted for 3 different fines contents under 3 different CSRs were shown under subheading 5.2.1, subheading 5.2.2, and subheading 5.2.3.

### 5.2.1. Silty Sands with FC 5%

For FC 5 percent, 29 cyclic tests were performed under 3 different CSRs and they were shown in Figure 5.5, Figure 5.6, Figure 5.7 and also all 3 CSRs at the same graph was shown in Figure 5.8.

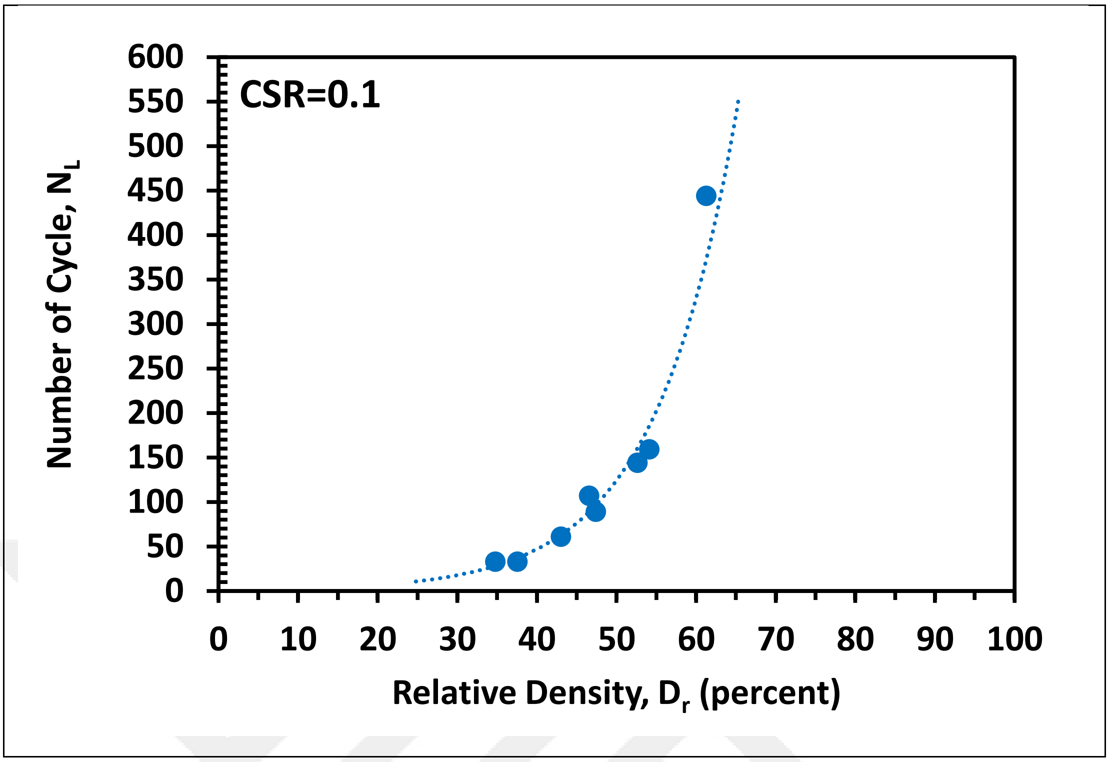


Figure 5.5. Dr vs. $N_L$ for silty sand (FC = 5%) under CSR = 0.1

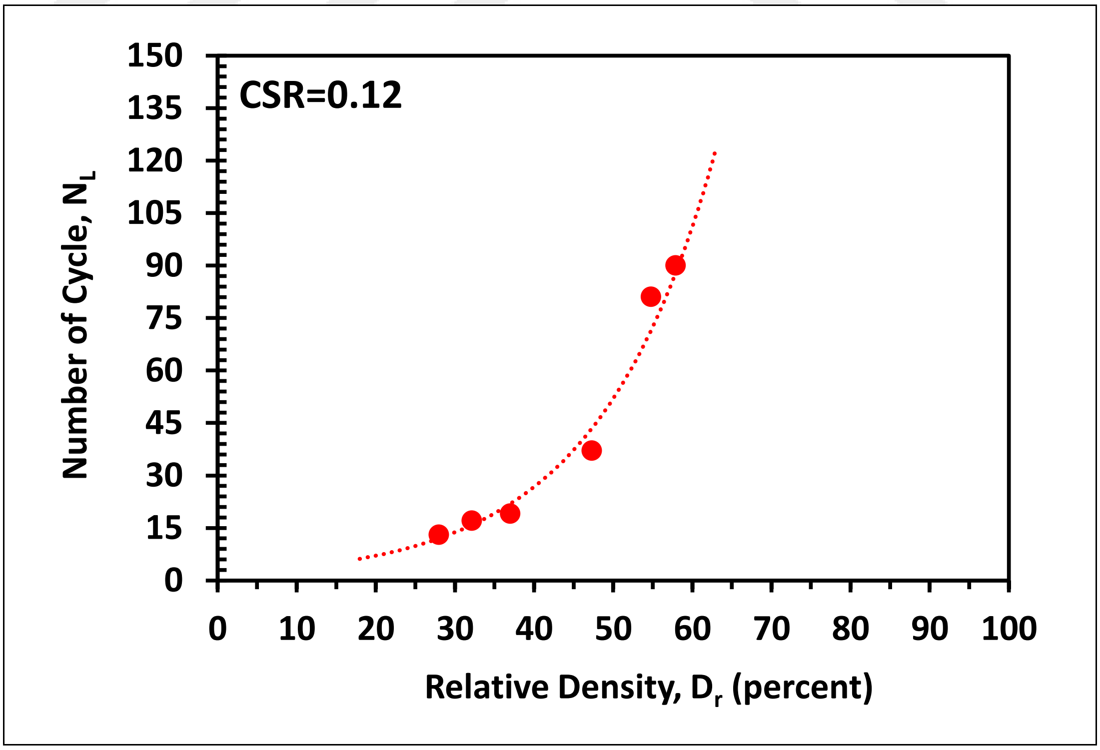


Figure 5.6. Dr vs. $N_L$ for silty sand (FC = 5%) under CSR = 0.12

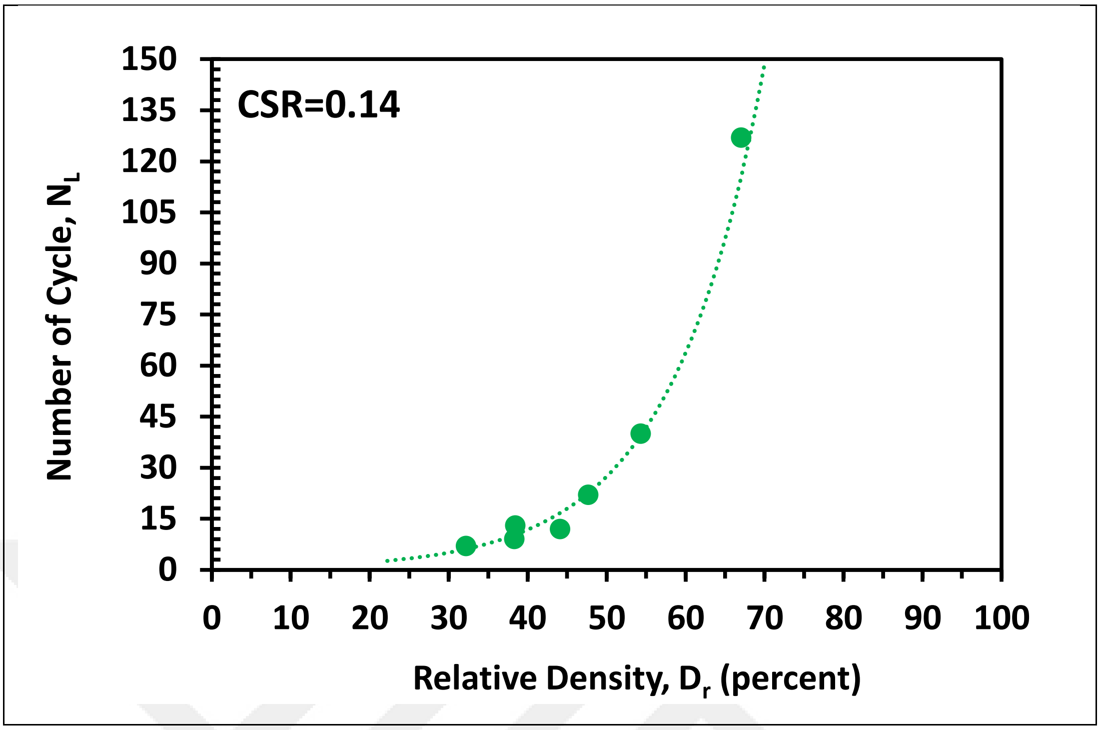


Figure 5.7. Dr vs. $N_L$ for silty sand (FC = 5%) under CSR = 0.14

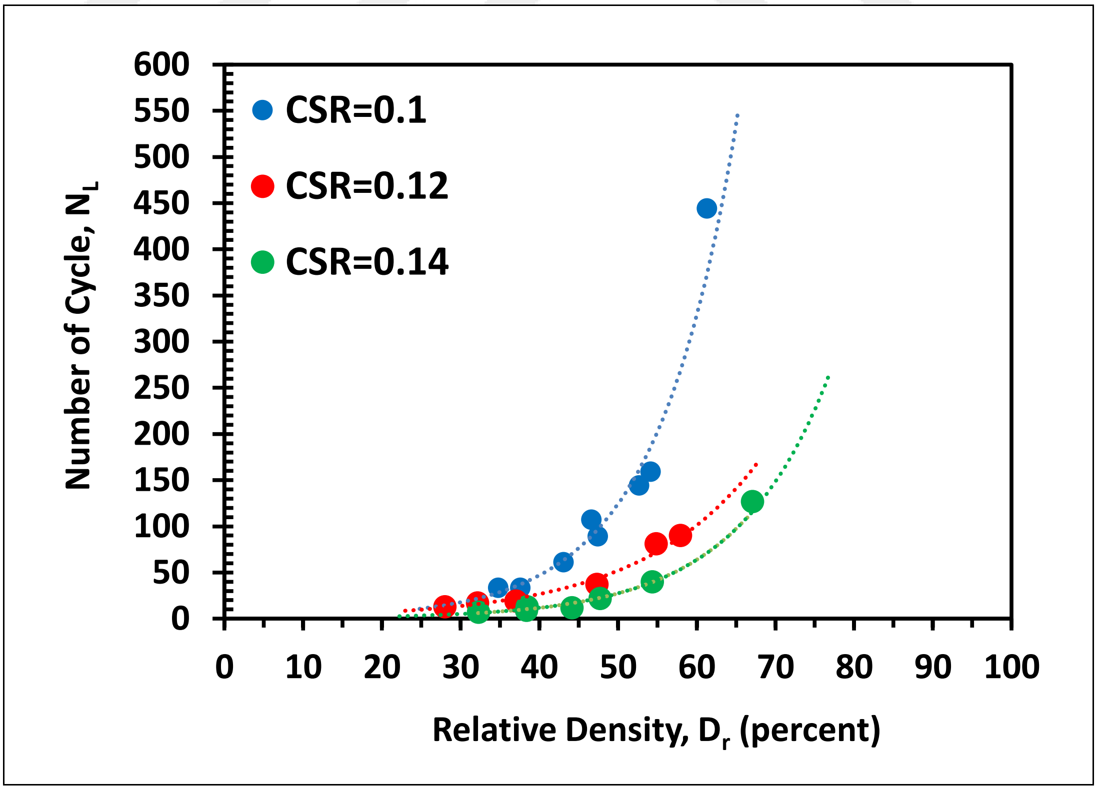


Figure 5.8. Dr vs. $N_L$ for silty sand (FC = 5%) under CSR = 0.1, 0.12 and 0.14

### 5.2.2. Silty Sands with FC 15%

For FC 15 percent, 25 cyclic tests were performed under 3 different CSRs and they were shown in Figure 5.9, Figure 5.10, Figure 5.11 and also all 3 CSRs at the same graph was shown in Figure 5.12.

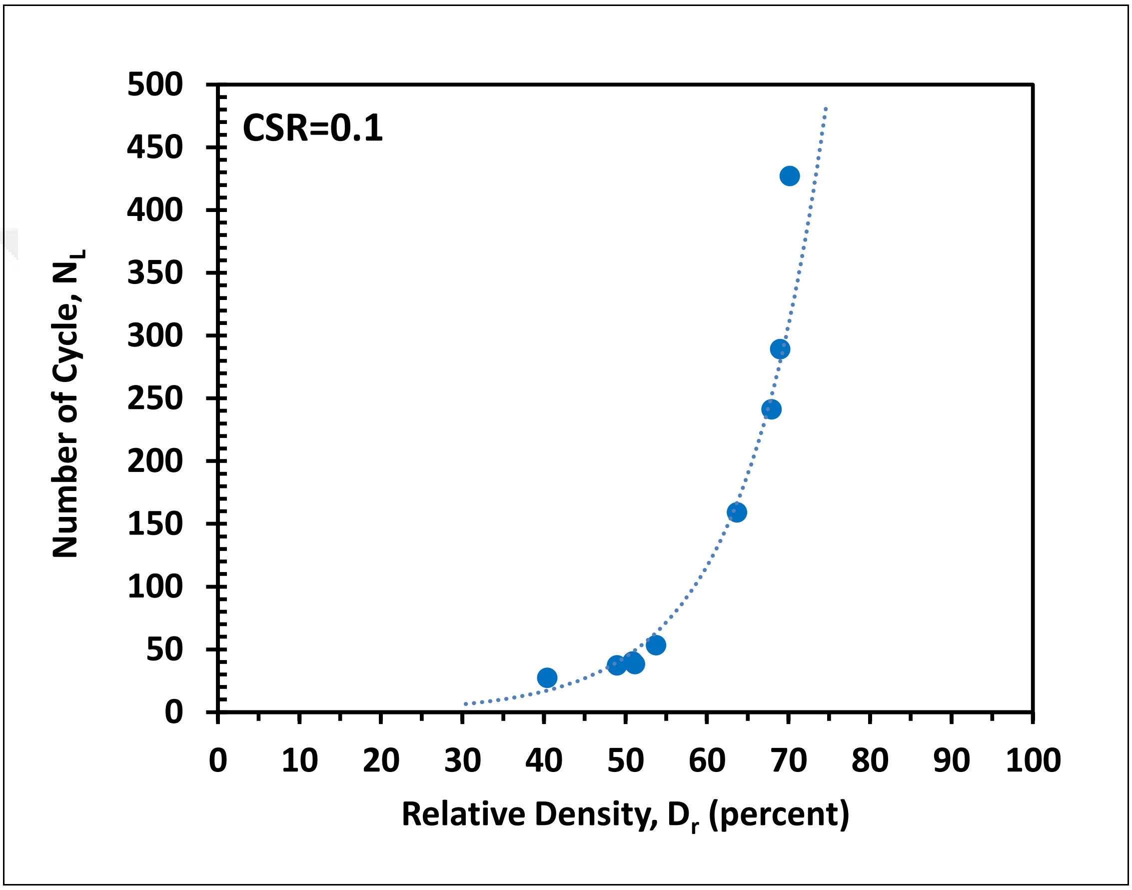


Figure 5.9. Dr vs. $N_L$ for silty sand (FC = 15%) under CSR = 0.1

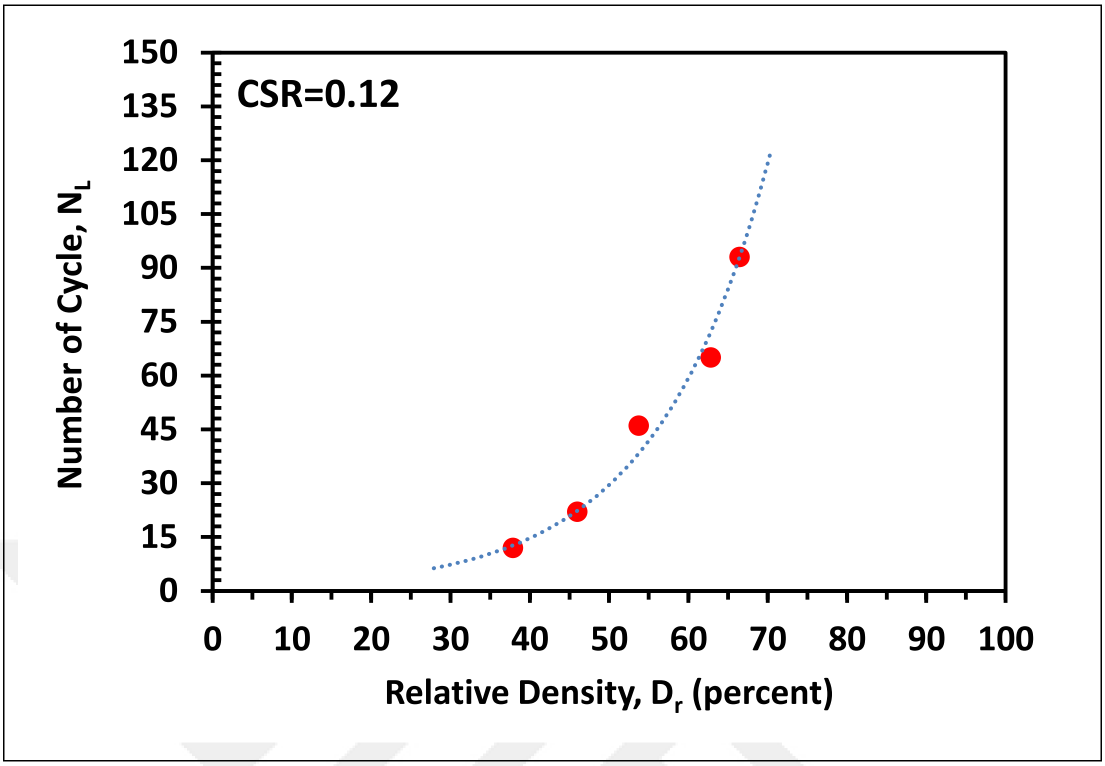


Figure 5.10. Dr vs. $N_L$ for silty sand (FC = 15%) under CSR = 0.12

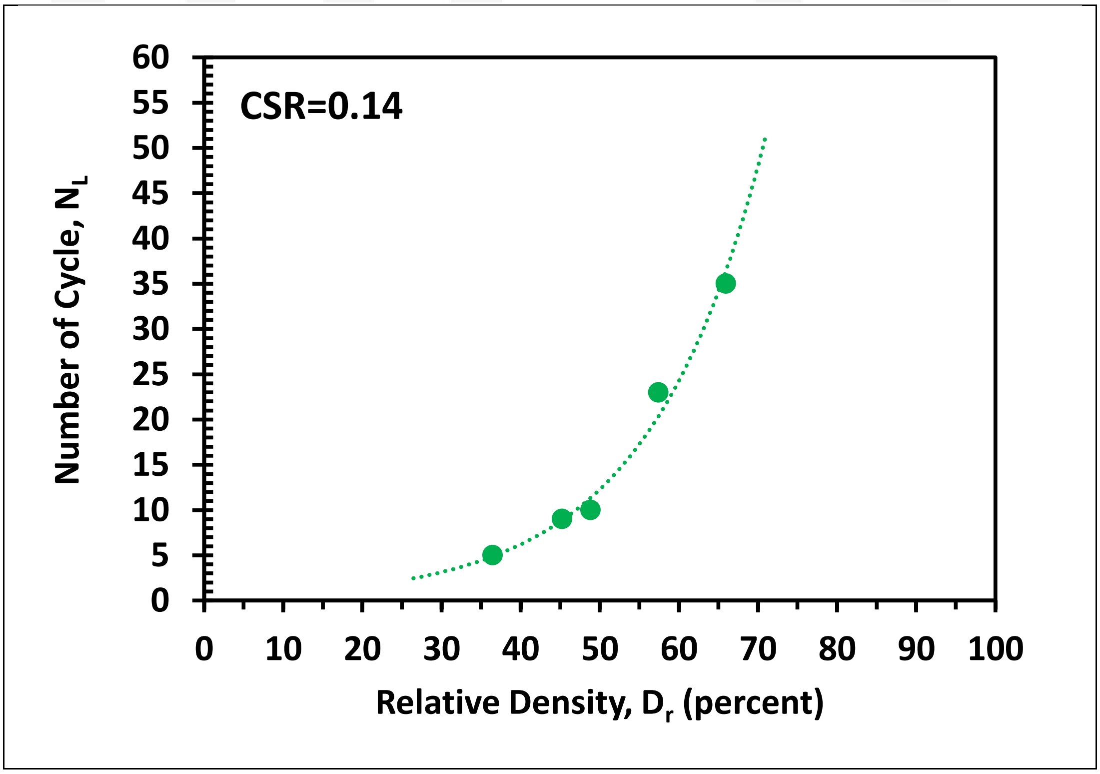


Figure 5.11. Dr vs. $N_L$ for silty sand (FC = 15%) under CSR = 0.14

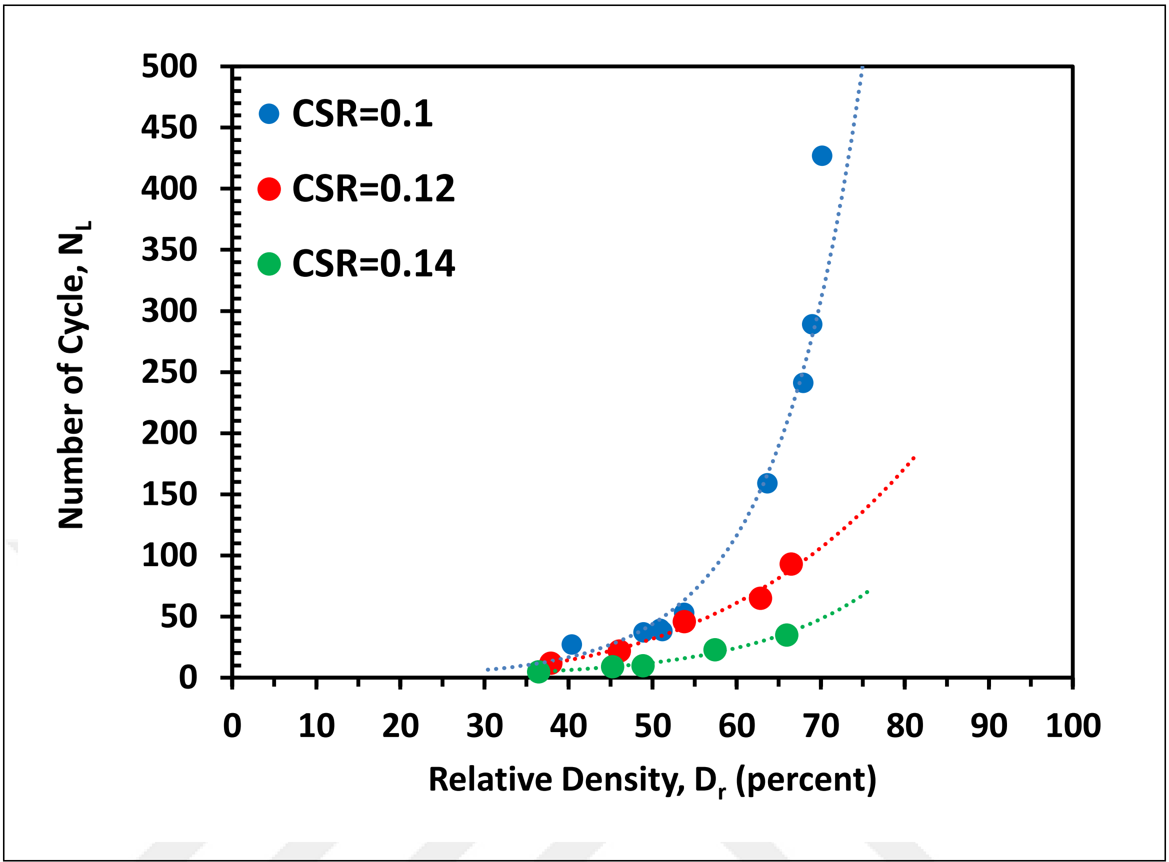


Figure 5.12. Dr vs. $N_L$ for silty sand (FC = 15%) under CSR = 0.1, 0.12 and 0.14

### 5.2.3. Silty Sands with FC 35%

For FC 35 percent, 23 cyclic tests were performed under 3 different CSRs and they were shown in Figure 5.13, Figure 5.14, Figure 5.15 and also all 3 CSRs at the same graph was shown in Figure 5.16. When we analyze the graphs plotted for FC 35 percent, it was seen that the relative density starts from about 60 percent compared to the other specimens (FC = 0, 5 and 15 percent). The reason of this can be easily understood when the minimum and maximum void ratios of the specimens (Figure 3.4) are examined. Silty sand with 35 percent fines content has a lower minimum void ratio compared to other specimens. Lower void ratios would make the sample much denser. Since the minimum and maximum void ratios are a function of relative density, the lowest relative density of the specimen which was prepared for the test was about 60 percent.

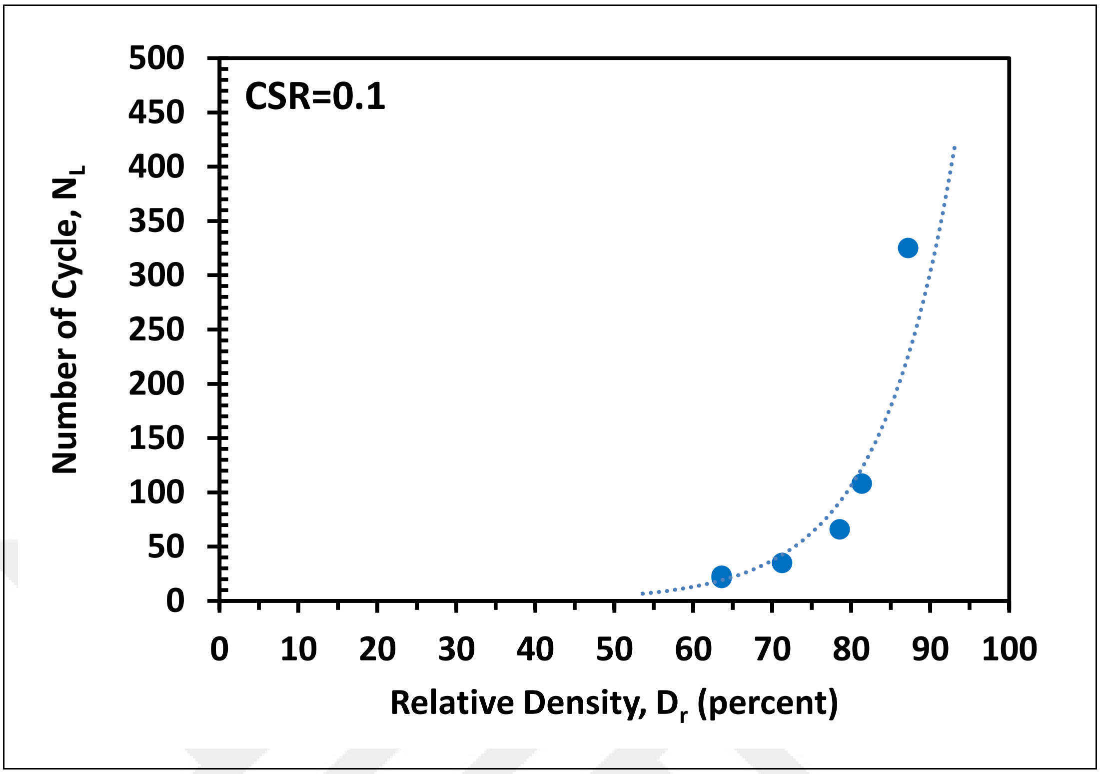


Figure 5.13. Dr vs. $N_L$ for silty sand (FC = 35%) under CSR = 0.1

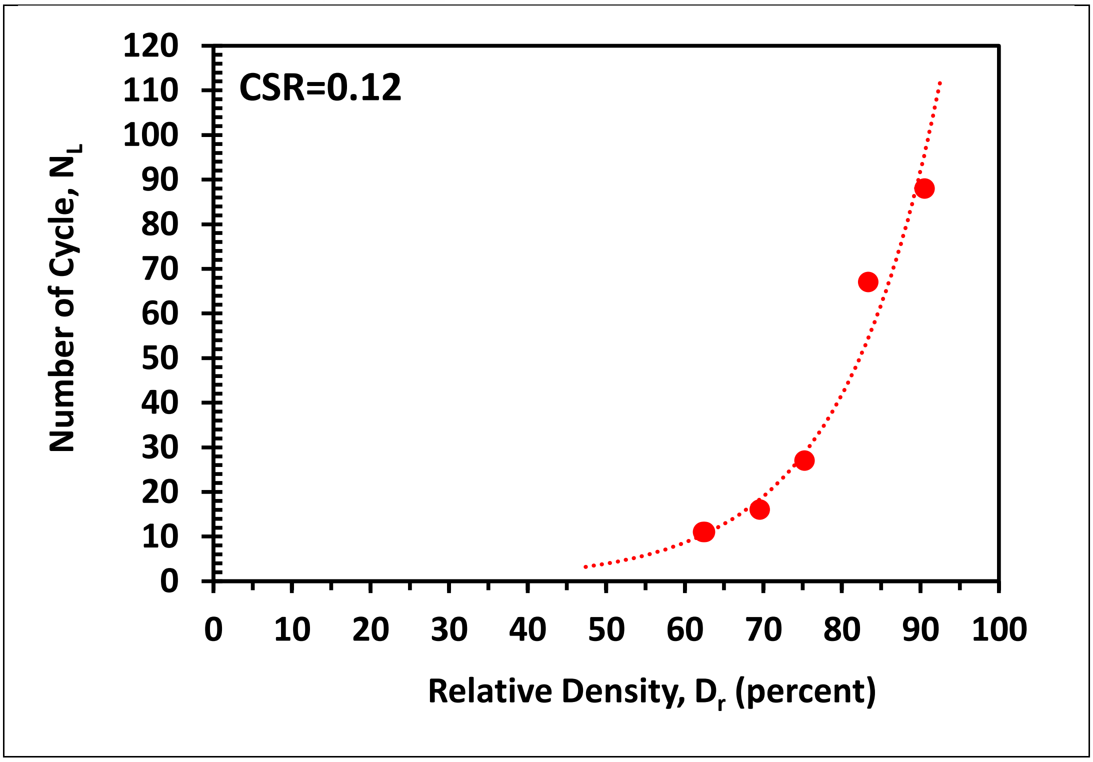


Figure 5.14. Dr vs. $N_L$ for silty sand (FC = 35%) under CSR = 0.12

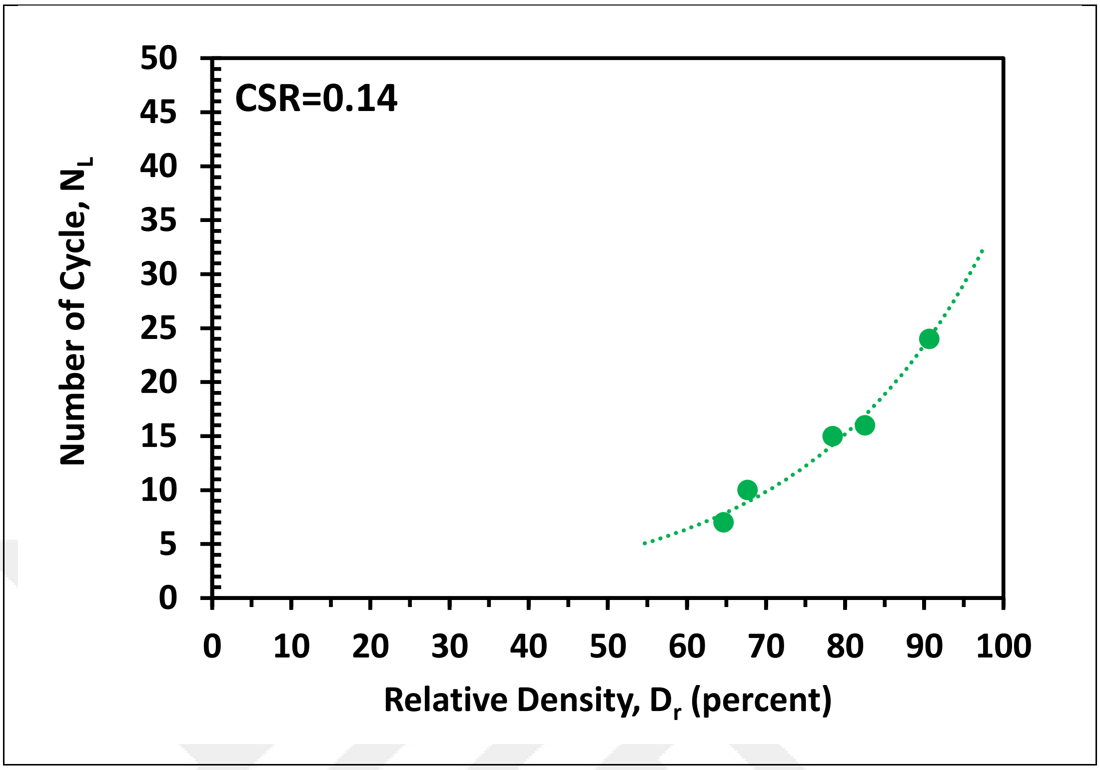


Figure 5.15. Dr vs. $N_L$ for silty sand (FC = 35%) under CSR = 0.14

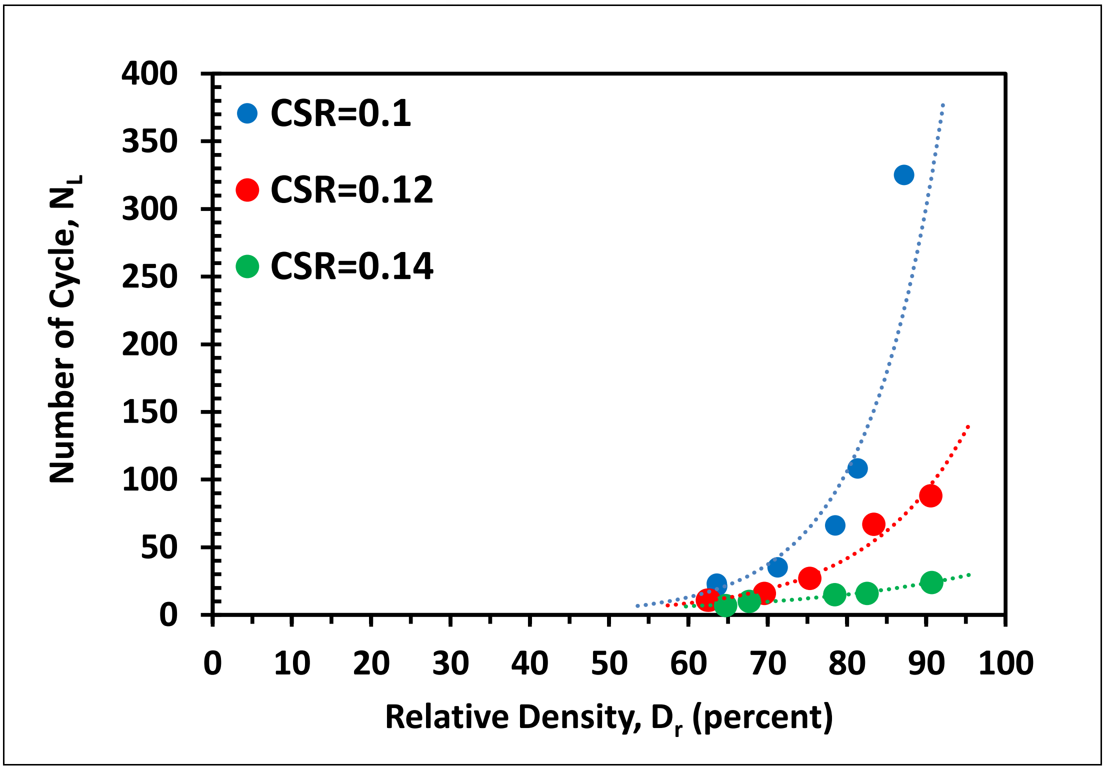


Figure 5.16. Dr vs. $N_L$ for silty sand (FC = 35%) under CSR = 0.1, 0.12 and 0.14

The change of a and b coefficients depending on the percentage of fine grain with CSR in equation 5.1 was shown in table 5.2.

Table 5.2. The change of a and b coefficients for clean and silty sands

| **FC (%)** | **CSR** | **Equation 5.1** | | |
|---|---|---|---|---|
| | | **a** | **b** | **$R^2$** |
| **0** | 0.1 | 7.203 | 0.037 | 1.00 |
| | 0.12 | 3.374 | 0.041 | 0.98 |
| | 0.14 | 2.834 | 0.029 | 0.98 |
| **5** | 0.1 | 0.958 | 0.097 | 0.97 |
| | 0.12 | 1.868 | 0.067 | 0.98 |
| | 0.14 | 0.401 | 0.085 | 0.97 |
| **15** | 0.1 | 0.338 | 0.097 | 0.95 |
| | 0.12 | 0.901 | 0.070 | 0.98 |
| | 0.14 | 0.405 | 0.068 | 0.99 |
| **35** | 0.1 | 0.025 | 0.105 | 0.94 |
| | 0.12 | 0.077 | 0.079 | 0.98 |
| | 0.14 | 0.475 | 0.043 | 0.96 |

When the Table 5.2 is analyzed, the effect of liquefaction behavior of fines content on the sands is obvious. Although coefficient a decreases for clean sand and increases for silty sand with 35 percent FC as CSR increases, no accurate opinion has been reached for silty sand with 5 and 15 percent FC. In addition, coefficient b is significant in terms of affecting the rate of increase [30].

## 5.3. THE RELATIONSHIP BETWEEN RELATIVE DENSITY AND NUMBER OF CYCLE UNDER THREE DIFFERENT CYCLIC STRESS RATIO

The relationship between relative density and the number of cycles required for liquefaction for clean and silty sands under 3 different CSRs were shown in Figure 5.17, Figure 5.18 and Figure 5.19.

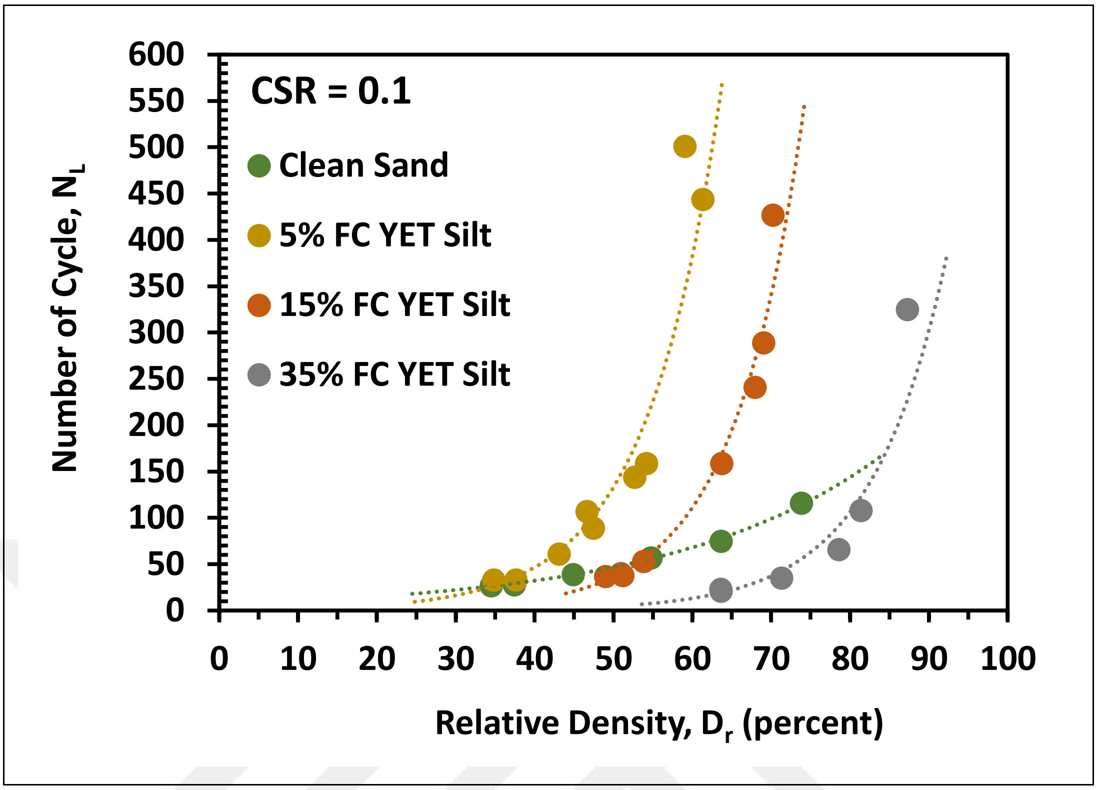


Figure 5.17. $D_r$ vs. $N_L$ for clean and silty sands under CSR = 0.1

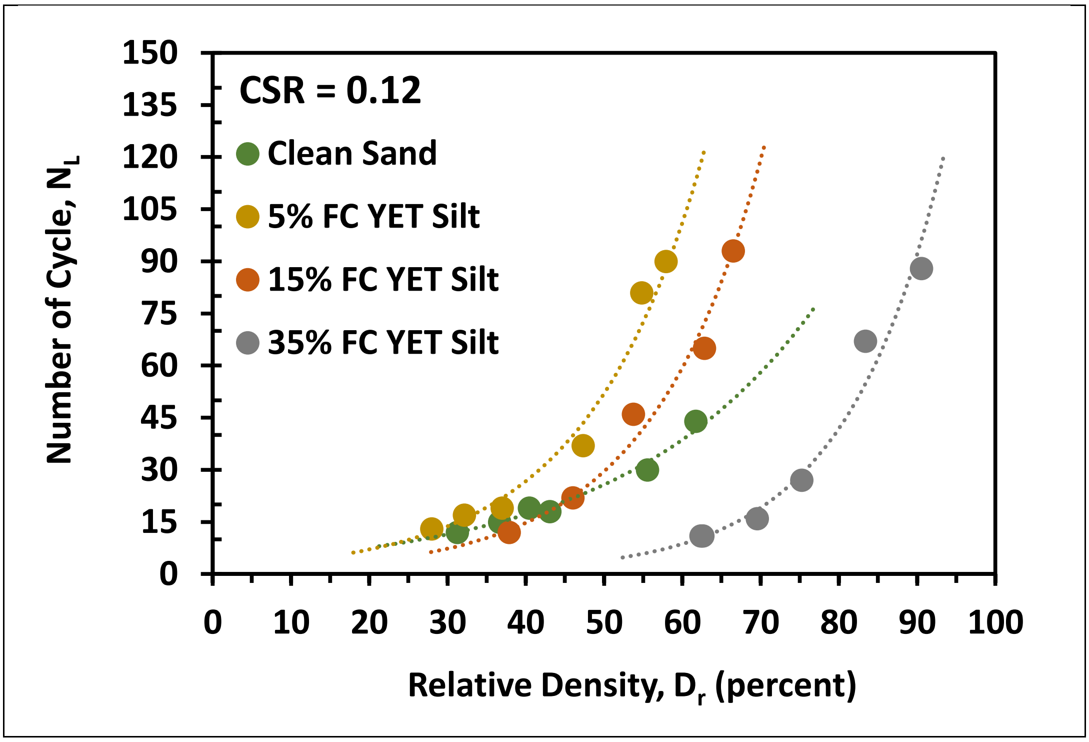


Figure 5.18. $D_r$ vs. $N_L$ for clean and silty sands under CSR = 0.12

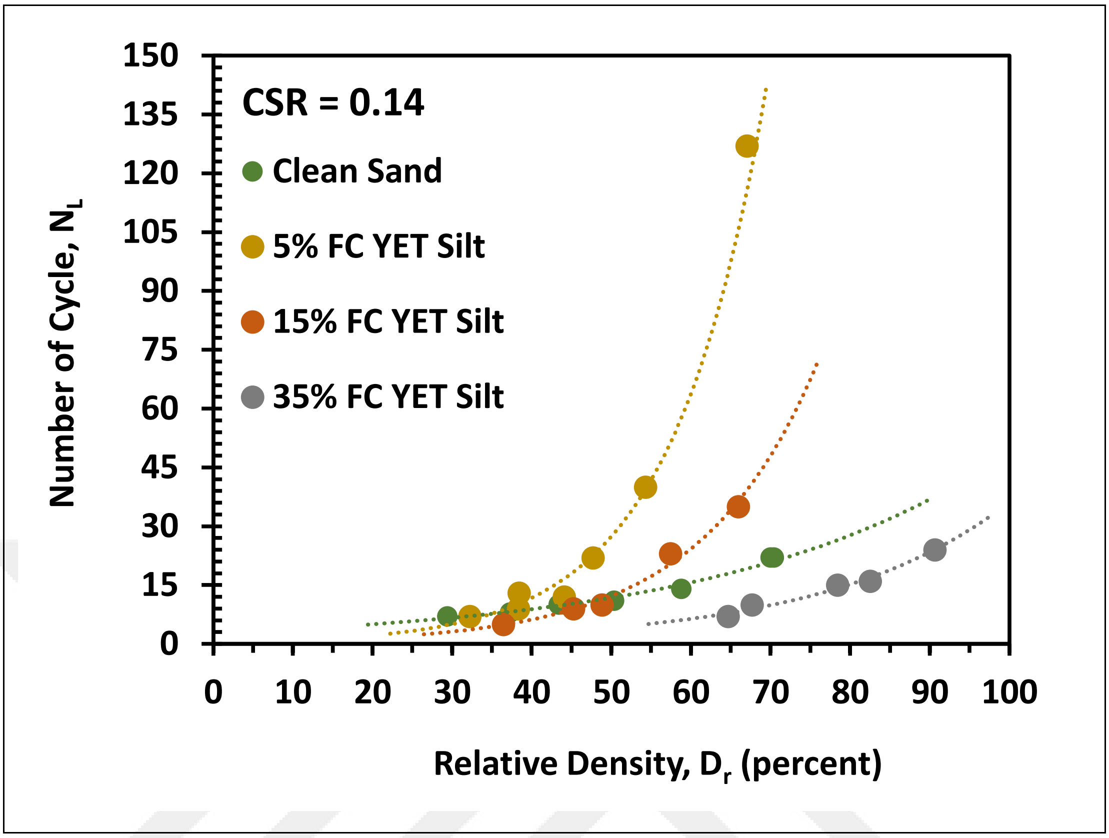


Figure 5.19. $D_r$ vs. $N_L$ for clean and silty sands under CSR = 0.14

When the Figure 5.17, Figure 5.18 and Figure 5.19 are examined, the trend curve shifted to the left when the fines content increased to 5 percent. When the fines content increased to 15 percent, trend curve shifted to the left or right depends on the relative density. When the fines content increased to 35 percent, the trend curve shifted to the right.

## 5.4. THE RELATIONSHIP BETWEEN NUMBER OF CYCLE AND CYCLIC STRESS RATIO

### 5.4.1. For Clean Sand (FC = 0%)

In this study, relative density values varying between 30 percent and 70 percent were selected for clean sand. The change of $N_L$ values calculated by using equation 5.1 in the mentioned constant Dr values with CSR was shown for clean sand in Figure 5.20.

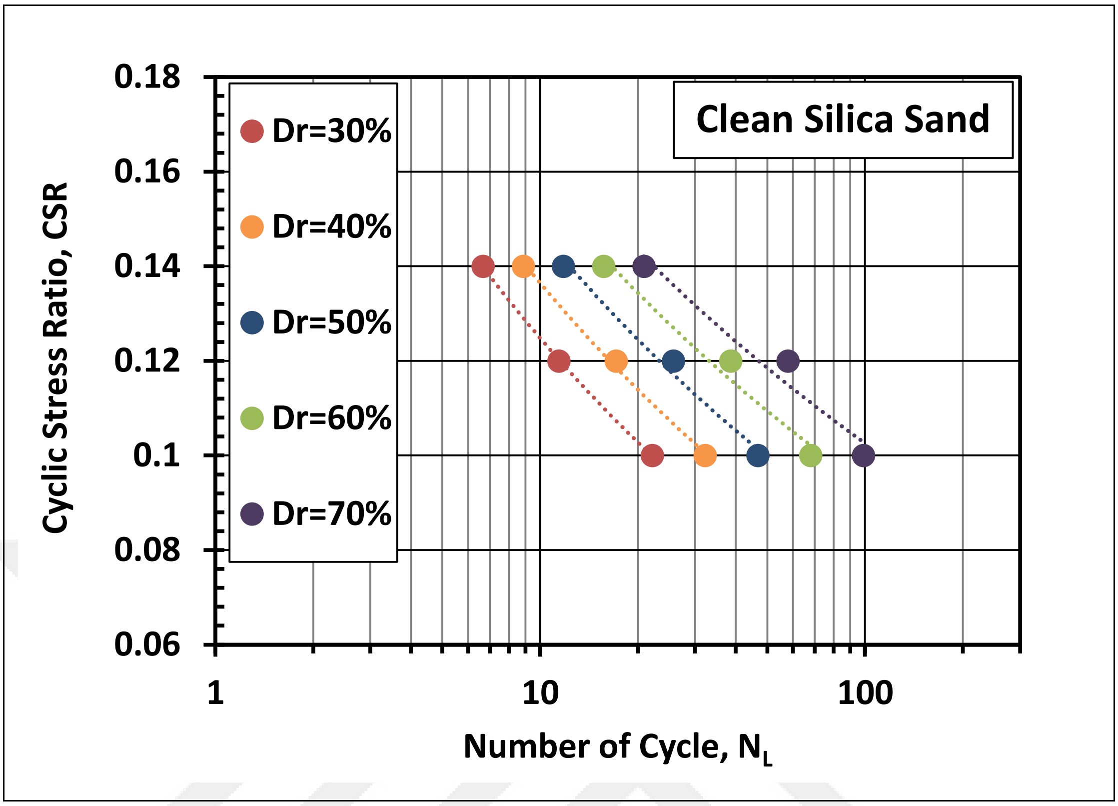


Figure 5.20. $N_L$ vs. CSR for clean sand (FC = 0%) at different relative densities

As the cyclic stress ratio decreased, the number of cycles required for liquefaction increased as expected. This increase is higher as the cyclic stress ratio decreases. Although the curves look like parallel to each other, the increase in the number of cycles required for liquefaction is not the same as the cyclic stress ratio decreases.

### 5.4.2. For Silty Sands (FC = 5, 15 and 35%)

The same $D_r$ (between 30 percent and 70 percent) values were chosen for 5 percent and 15 percent FC in order to compare with clean sand. However, $D_r$ values between 50 percent and 80 percent were used for 35 percent FC because of the reason mentioned in chapter 5.2.3. The change of $N_L$ with CSR was shown for silty sands in Figure 5.21, Figure 5.22 and Figure 5.23.

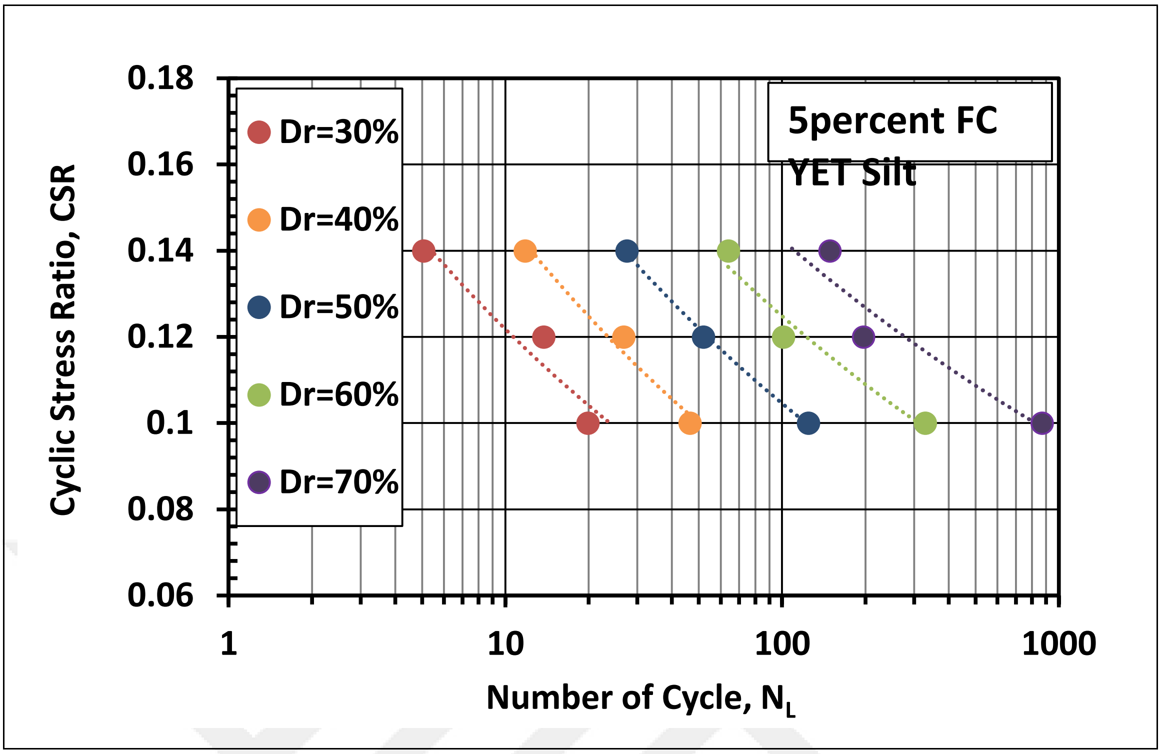


Figure 5.21. $N_L$ vs. CSR for silty sand (FC = 5%) at different relative densities

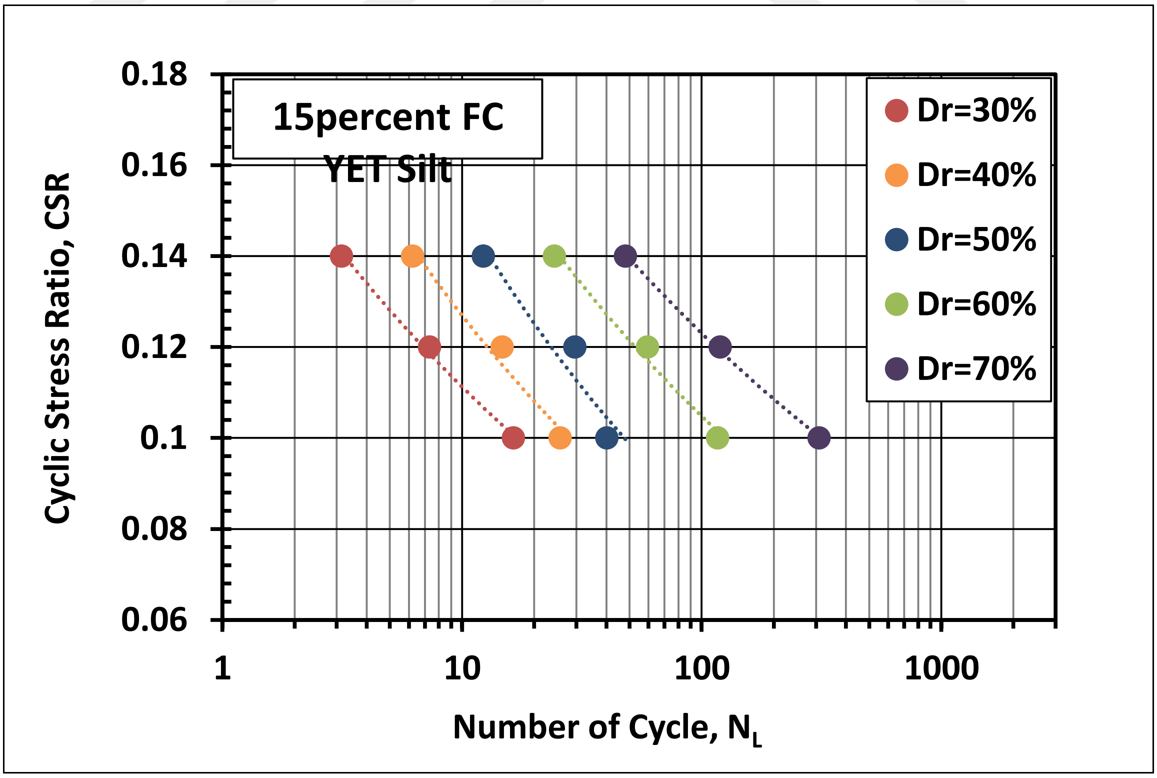


Figure 5.22. $N_L$ vs. CSR for silty sand (FC = 15%) at different relative densities

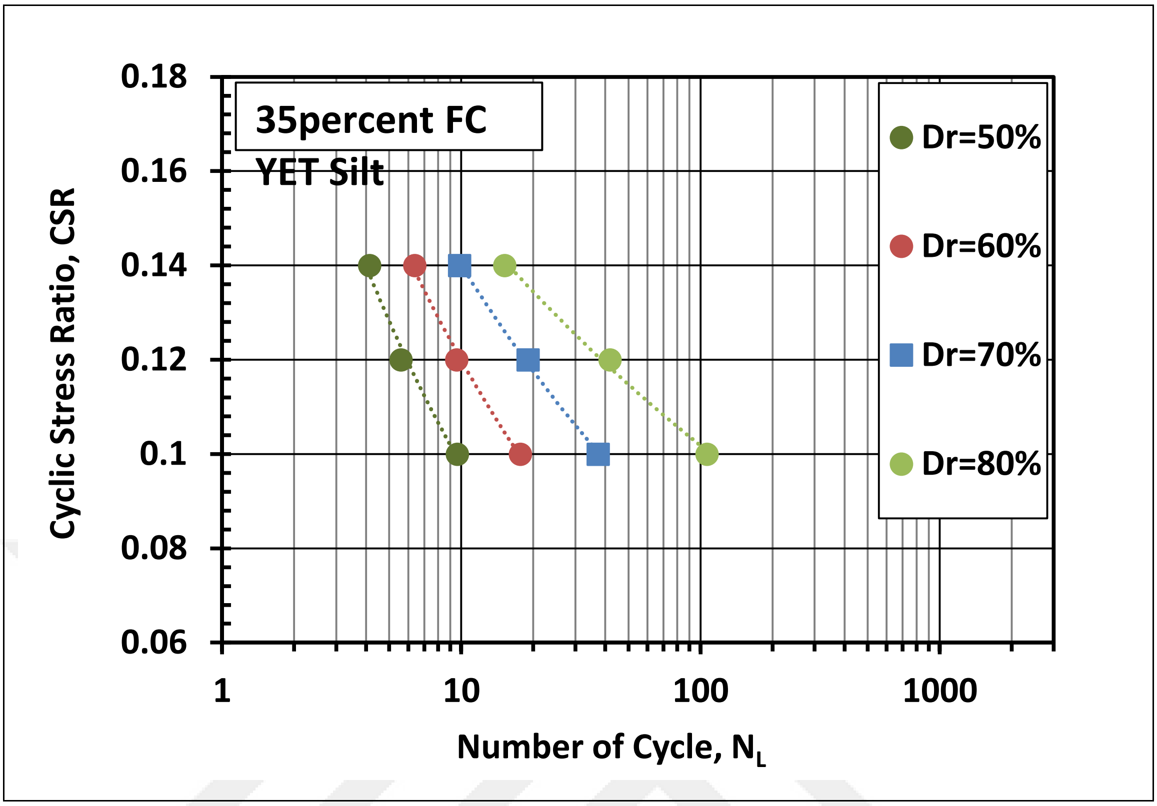


Figure 5.23. $N_L$ vs. CSR for silty sand (FC = 35%) at different relative densities

The number of cycles required for liquefaction of silty sands with 5 percent FC increased faster than clean sands as CSR decreased. This shows that silty sands with 5 percent FC are more resistant than clean sands. The number of cycles required for liquefaction of silty sands with 35 percent FC increased less than the sands as CSR decreased. This means that clean sands are more resistant compared to silty sands with 35 percent FC. This condition is different for silty sands with 15 percent FC. In cases where the relative density is lower than 45 percent, the number of cycles required for liquefaction of clean sands is higher than silty sands with 15 percent FC as CSR decreases. When the relative density is higher than 45 percent, this condition reversed.

Monkul and Kendir [30], Idriss and Boulanger [67] and Wei et al. [33] stated that the change of cyclic stress ratio with the number of cycle required for liquefaction are expressed with an exponential function for clean and silty sands in constant $D_r$. When we analyze the Figure 5.21, Figure 5.22 and Figure 5.23, exponential function for cyclic stress ratio and number of cycle required for liquefaction was shown in equation 5.2.

$$CSR = c.N_L^{(-d)} \tag{5.2}$$

The change of c and d coefficients depending on the percentage of fine grain with $D_r$ in equation 5.2 was shown in Table 5.3.

Table 5.3. The change of c and d coefficients for clean and silty sands

| FC (%) | $D_r$ (%) | Equation 5.2 | | |
|---|---|---|---|---|
| | | c | d | $R^2$ |
| 0 | 30 | 0.238 | 0.280 | 1.00 |
| | 40 | 0.249 | 0.261 | 1.00 |
| | 50 | 0.270 | 0.242 | 0.98 |
| | 60 | 0.263 | 0.224 | 0.97 |
| | 70 | 0.267 | 0.208 | 0.95 |
| 5 | 30 | 0.205 | 0.227 | 0.91 |
| | 40 | 0.257 | 0.241 | 0.97 |
| | 50 | 0.291 | 0.222 | 1.00 |
| | 60 | 0.307 | 0.195 | 0.96 |
| | 70 | 0.309 | 0.168 | 0.90 |
| 15 | 30 | 0.178 | 0.203 | 1.00 |
| | 40 | 0.217 | 0.232 | 0.97 |
| | 50 | 0.273 | 0.260 | 0.90 |
| | 60 | 0.278 | 0.212 | 0.98 |
| | 70 | 0.283 | 0.181 | 1.00 |
| 35 | 50 | 0.240 | 0.391 | 0.99 |
| | 60 | 0.255 | 0.328 | 1.00 |
| | 70 | 0.251 | 0.253 | 1.00 |
| | 80 | 0.225 | 0.173 | 0.99 |

When the Table 5.3 is analyzed, the coefficient c tends to increase as relative density increases for clean and silty sands except silty sands with 35 percent FC. On the contrary, the coefficient d tends to decrease as relative density increases for clean and silty sands.

## 5.5. COMPARISON OF THE LIQUEFACTION RESISTANCE (CRR) OF CLEAN AND SILTY SANDS

### 5.5.1. The Relationship between Relative Density and Cyclic Resistance Ratio

In this study, cyclic resistance ratio (CRR) was chosen as the cyclic stress ratio required for liquefaction in 20 cycles of loading ($CSR_{[@NL=20]}$) [68]. By using equation 5.2, the CRR values can be calculated, or the value corresponding to the 20$^{th}$ number of cycle in Figure 5.20, Figure 5.21, Figure 5.22 and Figure 5.23 can be taken as CRR value. The relatationship between relative denstiy and cyclic resistance ratio was shown in Table 5.4 and Figure 5.24.

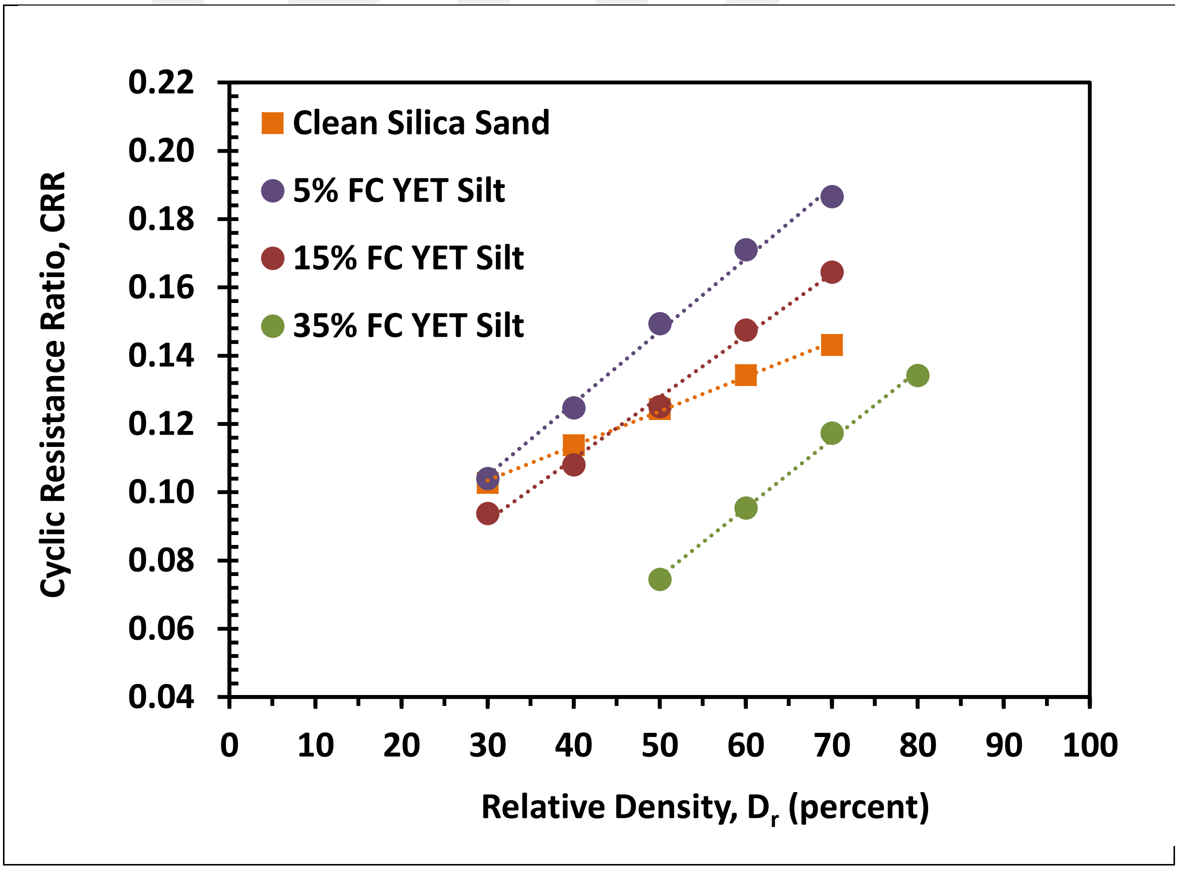


Figure 5.24. The relative density vs. cyclic resistance ratio for clean and silty sands

When we analyze Figure 5.24, we can see the influence of silty sands on seismic liquefaction of clean sands. The cyclic resistance ratio of soils was affected by both fines contents and the relative density. As the relative density increased, the cyclical resistance ratio increased as expected. Firstly, the liquefaction resistance of silty sands with 5 percent FC is equal to the clean sand for 30 percent $D_r$. After this value, as the relative density increased, the silty sands with 5 percent FC became more resistant compared to the clean sand. Secondly, the silty sands with 15 percent FC have lower liquefaction resistance compared to the liquefaction resistance of clean sand for less than about 45 percent $D_r$. Beyond this $D_r$ value, the liquefaction resistance of silty sands with 15 percent FC became higher than the clean sand. Lastly, although the relative density range of silty sands with 35 percent FC is between 50 percent and 80 percent, it is clear seen that the liquefaction resistance of silty sands with 35 percent FC is less than the liquefaction resistance of clean sand.

Table 5.4. CRR values for different relative densities for clean and silty sands

| **Dr** | **CRR** | | | |
|---|---|---|---|---|
| | **Silica Sand** | **5% FC** | **15% FC** | **35% FC** |
| **30** | 0.103 | 0.104 | 0.094 | - |
| **40** | 0.114 | 0.125 | 0.108 | - |
| **50** | 0.124 | 0.149 | 0.125 | 0.075 |
| **60** | 0.134 | 0.171 | 0.148 | 0.095 |
| **70** | 0.143 | 0.187 | 0.165 | 0.117 |
| **80** | - | - | - | 0.134 |

### 5.5.2. The Relationship between Fines Contents and Cyclic Resistance Ratio

The main curiosity point is the behavior of fines contents on the liquefaction resistance of clean sand. In order to clearly see the fines content effect, the relationship between fines content (FC) and the cyclic resistance ratio (CRR) was shown in Figure 5.25.

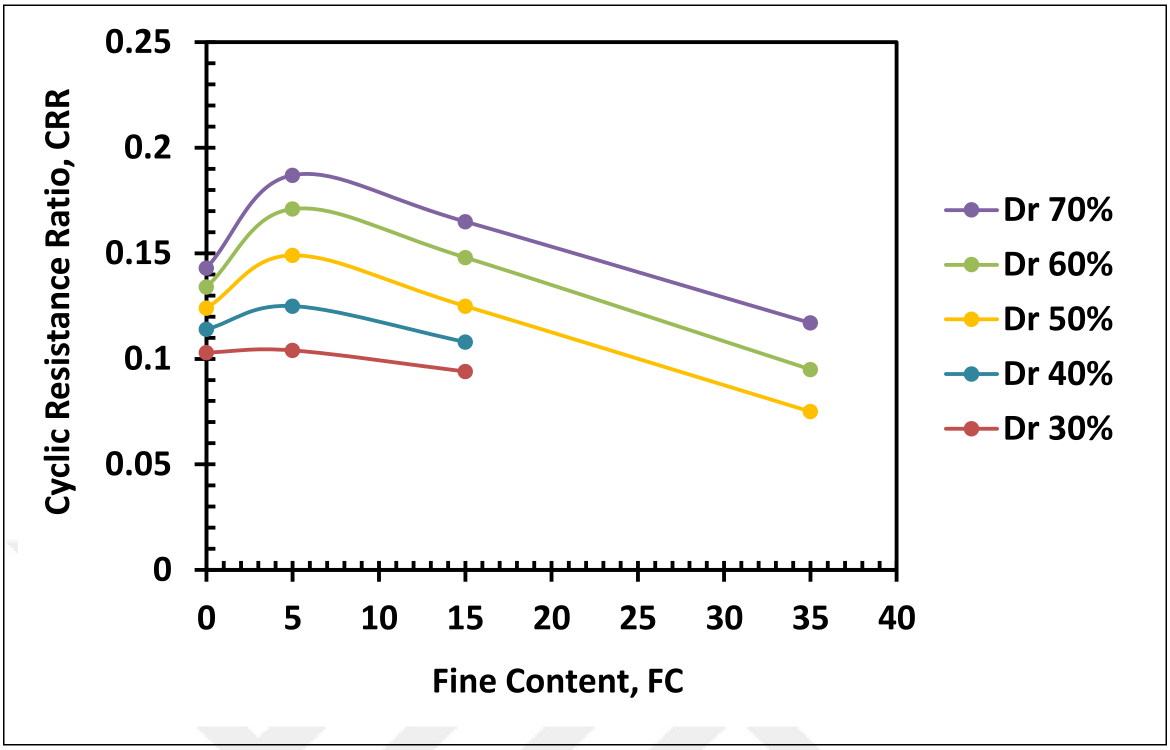


Figure 5.25. The fines content vs. cyclic resistance ratio at different relative densities

When the Figure 5.25 is carefully analyzed, the positive effect of the relative density on liquefaction resistance of the silty sand, especially for FC 5 percent, has became much clear as the relative density increases. Another point is that the cyclic resistance ratio reached the peak in 5 percent FC, and then it started to decrease with increasing FC. These findings were found to be compatible in the literature with Polito's PhD thesis [69] and study of Carraro et al. [29]. Then, this relation was shown in Figure 5.26.

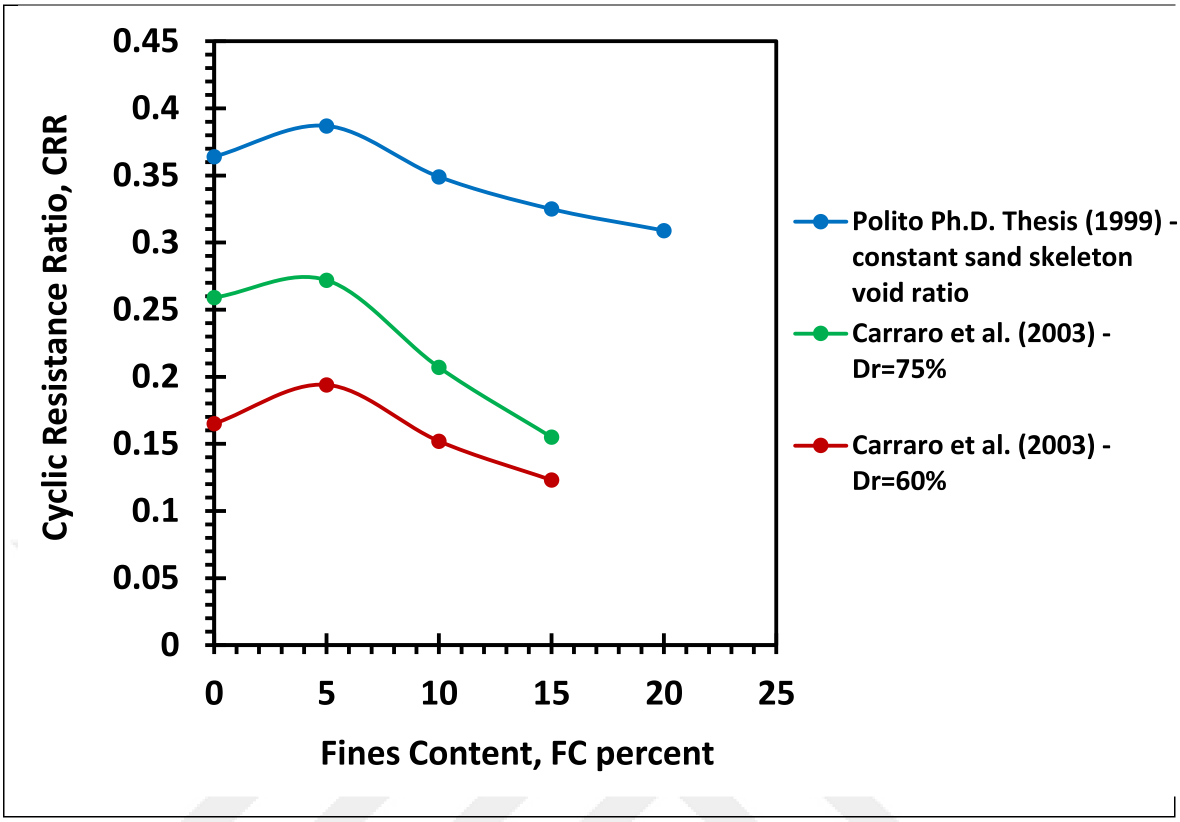


Figure 5.26. The fines content vs. cyclic resistance ratio from PhD thesis of Polito (1999) and Carraro et al. (2003) [15, 65]

### 5.5.3. The Relationship between Coefficient of Uniformity ($C_u$) and Cyclic Resistance Ratio for Silty Sands

The change of coefficient of uniformity ($C_u$) with cyclic resistance ratio was shown in Figure 5.27. Silty sands with 5 percent and 15 percent FC were shown in figure at five different relative densities (30, 40, 50, 60 and 70 percent) but silty sand with 35 percent FC were shown in figure for three different relative densities (50, 60 and 70 percent). If figure is analyzed, it can be clearly seen that the cyclic resistance ratio decreases as coefficient of uniformity increases.

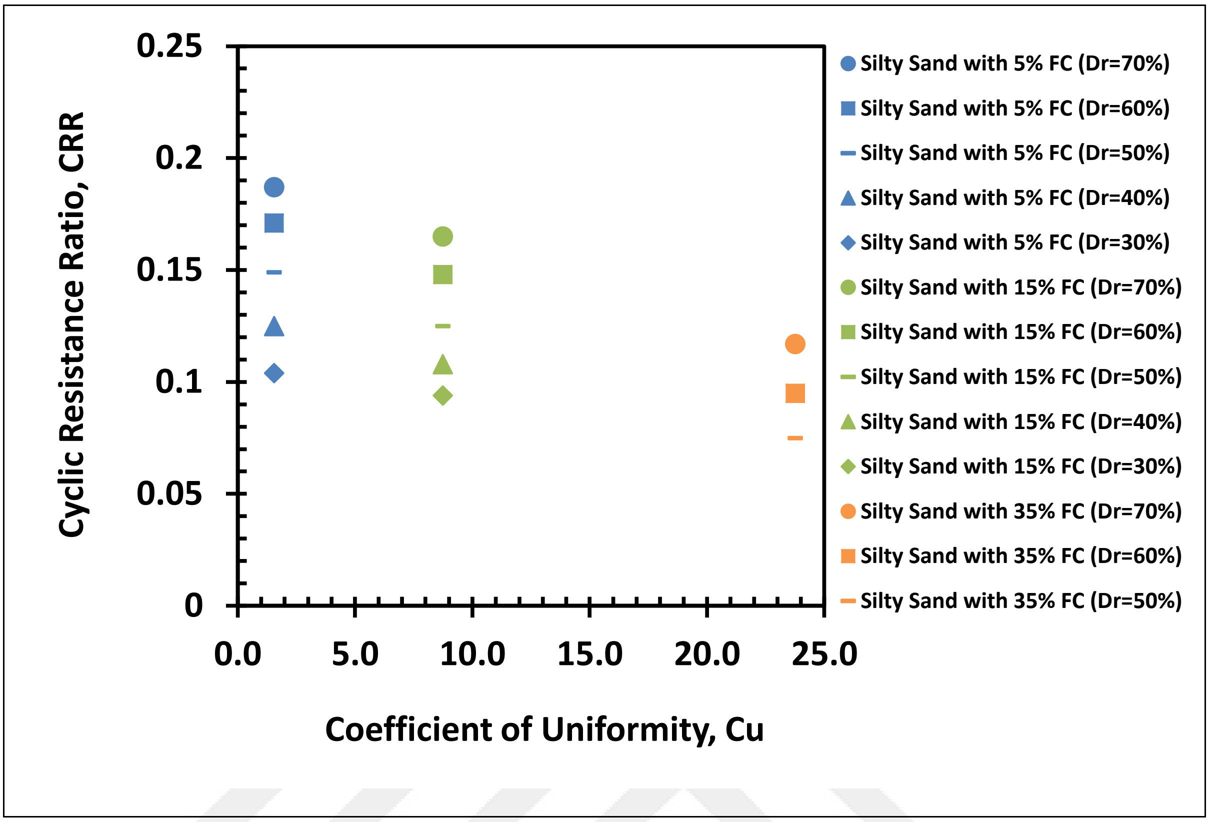


Figure 5.27. The coefficient of uniformity vs. CRR for tested silty sands

When the Figure 5.27 is analyzed, cyclic resistance ratio (CRR) can be expressed as a function of coefficient of uniformity ($C_u$) in equation 5.3. And also, the change of x and y coefficients for silty sands (5, 15 and 35 percent) was shown in Table 5.5.

$$CRR = -x.C_u + y \tag{5.3}$$

Table 5.5. The change of x and y coefficients for silty sands

| $D_r$ | Equation 5.3 | | |
|---|---|---|---|
| | **x** | **y** | $R^2$ |
| 30 | 0.0014 | 0.1062 | 1.00 |
| 40 | 0.0024 | 0.1287 | 1.00 |
| 50 | 0.0033 | 0.1541 | 1.00 |
| 60 | 0.0034 | 0.1770 | 1.00 |
| 70 | 0.0032 | 0.1922 | 1.00 |

The coefficient of y increases as the relative density increases. In addition, the coefficient of x increases up to 60 percent of $D_r$ and then it decreases.

# 6. SUMMARY AND CONCLUSIONS

Liquefaction behavior of silty sands has been an important topic studied by researchers for decades. In this study, the effects of silty sands on liquefaction of clean sands were investigated. For this study, sand and non-plastic silt (YET Silt) specimens, whose properties were determined as a result of laboratory experiments, were used. Sand was mixed in 3 different fines contents (5, 15 and 35 percent FC) and silty sands were obtained.

Cyclic experiments were performed in Soil Mechanics Laboratory of Yeditepe University with ShearTrack II-DSS test device under 3 different cyclic stress ratios (CSR = 0.1, 0.12 and 0.14) and at different relative densities. A total of 123 cyclic tests were performed for clean and silty sands. Two criteria were selected for liquefaction to occur. The first one is that the excess pore water pressure ratio ($r_u$) reaches 1.0, and the second one is that the peak to peak strain ($\gamma$) reaches 10 percent. While the effects of silty sands on liquefaction resistance of sands, the number of cycles required for liquefaction was chosen as the cyclic resistance ratio which is 20 ($CSR_{[@NL=20]}$).

The results obtained in consequence of cyclic tests are listed as follows.

1. As a result of experiments performed under 3 different cyclic stress ratios, it was observed that the number of cycles increased with increasing relative density. The relationship between the relative density and the number of cycles was shown in equation 5.1.
2. As a result of experiments performed at various relative densities, it was observed that the number of cycles increased as the cyclic stress ratio decreased. The relationship between number of cycles and the cyclic stress ratio was shown in equation 5.2.
3. Relationship with relative density and cyclic resistance ratio was shown in Figure 5.24. It was observed that liquefaction resistance of silty sands with 5 percent fines content increases compared to the resistance of clean sands and liquefaction resistance of silty sands with 35 percent fines content decreases compared to the resistance of clean sands.
4. It has been observed that liquefaction resistance of silty sands with 15 percent fines content decreases on soils where the relative density is less than 45 percent compared to the resistance of clean sands and liquefaction resistance of silty sands with 15 percent

fines content increases on soils where the relative density is higher than 45percent compared to the resistance of clean sands.

5. Another finding was observed when Figure 5.25 was analyzed. For 4 different relative densities, the cyclic resistance ratio reached a peak when the silt content increased up to 5 percent, then the cyclic resistance began to decrease as the silt content increased. As the relative density increased, the increase of the cyclic resistance became evident for especially silts with 5 percent FC.
6. As the silt contents increased, the coefficient of uniformity ($C_u$) increased. The cyclic resistance ratio decreased with increasing coefficient of uniformity at a constant relative density.

In the light of all these results, the effects of silty sands on clean sands have been analyzed by cyclic direct simple shear tests and brought into the literature.

## REFERENCES


1. Kramer SL. *Geotechnical Earthquake Engineering*. Upper Saddle River, NJ: Prentice Hall; 1996.

2. Post A. Effects of the March 1964 Alaska earthquake on glaciers. US Government Printing Office; 1967.

3. Iwasaki T. Soil liquefaction studies in Japan: state-of-the-art. *Soil Dynamics and Earthquake Engineering*. 1986 Jan 1;5(1):2-68.

4. Close-up of Government Hill elementary school, which was destroyed by the Government Hill landslide. Anchorage, Alaska. Close-up of Government Hill elementary school, which was destroyed by the Government Hill landslide. Anchorage, Alaska [cited 2020 21 May]. Available from: https://earthquake.usgs.gov/earthquakes/ events/ alaska1964/1964pics.php.

5. Effects of liquefaction caused by the earthquake on apartment buildings in Niigata [cited 2020 07 March]. Available from: https://en.wikipedia.org/wiki/1964_Niigata_earthquake#/media/File:Liquefaction_at_ Niigata.JPG.

6. Lateral displacement of a quay wall on Port Island, Kobe 1995. [cited 2020 10 March]. Available from: https://depts.washington.edu/liquefy/selectpiclique/kobe95/ portgraben.jpg.

7. Foundation bearing failures (Adapazari, Turkey, 17 Agust 1999 Izmit Earthquake). [cited 2020 10 March]. Available from: https://akademik.adu.edu.tr/fakulte/muhendislik/ personel/uploads/ssaglam/liquefaction-presentation-1515414235.pdf.

8. Flooding due to post-earthquake soil liquefaction in Christchurch, New Zealand. [cited 2020 21 May] Available from: https://www.citymetric.com/some-engineers-simulated-earthquake-blowing-new-zealand-suburb.

9. Bray JD, Sancio RB, Durgunoglu T, Onalp A, Youd TL, Stewart JP, Seed RB, Cetin OK, Bol E, Baturay MB, Christensen C. Subsurface characterization at ground failure sites in Adapazari, Turkey. *Journal of Geotechnical and Geoenvironmental Engineering.* 2004 Jul;130(7):673-85.

10. Bhattacharya S, Hyodo M, Goda K, Tazoh T, Taylor CA. Liquefaction of soil in the Tokyo Bay area from the 2011 Tohoku (Japan) earthquake. *Soil Dynamics and Earthquake Engineering.* 2011 Nov 1;31(11):1618-28.

11. Taylor ML, Cubrinovski M, Bradley BA. Characterisation of ground conditions in the Christchurch Central Business District. *Aust Geomech J*. 2012;47(4):43–58.

12. Coduto DP, *Geotechnical Engineering Principles and Practices.* New Jersey: Prentice Hall; 1999.

13. Seed HB, Idriss IM, Arango I. Evaluation of liquefaction potential using field performance data. *Journal of Geotechnical Engineering*. 1983 Mar;109(3):458-82.

14. Seed HB, Idriss IM. Simplified procedure for evaluating soil liquefaction potential. *Journal of the Soil Mechanics and Foundations Division*. 1971 Sep;97(9):1249-73.

15. Carraro JA, Bandini P, Salgado R. Liquefaction resistance of clean and non-plastic silty sands based on cone penetration resistance. *Journal of geotechnical and geoenvironmental engineering*. 2003 Nov;129(11):965-76.

16. Seed B, Lee KL. Liquefaction of saturated sands during cyclic loading. *Journal of Soil Mechanics & Foundations Div*. 1966 Nov;92(ASCE# 4972 Proceeding).

17. Sitharam TG, Dash HK. Effect of non-plastic fines on cyclic behaviour of sandy soils. *InGeoCongress 2008: Geosustainability and Geohazard Mitigation 2008* (pp. 319-326).

18. Cubrinovski M, Rees S, Bowman E. Effects of non-plastic fines on liquefaction resistance of sandy soils. *InEarthquake Engineering in Europe*. 2010 (pp. 125-144). Springer, Dordrecht.

19. Meyerhof, G. G. Discussion on Research on Determining the Density of Sands by Spoon Penetration Testing. Proceedings, *Fourth International Conference on Soil Mechanics and Foundation Engineering*; 1957.

20. Tokimatsu, K. and Yoshimi, Y. Empirical correlation of soil liquefaction based on SPT N-value and Fines Content. *Soils and Foundations*. 1983;23(4):56–74.

21. Skempton, A. W. Standard penetration test procedures and the effect in sands of overburden pressure, relative density, particle size, aging and overconsolidation. *Geotechnique*. 1986;36(3):425–447.

22. Cubrinovski, M. and Ishihara, K. Empirical correlation between SPT-N value and relative density for sandy soils. *Soils and Foundations*. 1999;39(5):61–71.

23. Ecemiş, N. and Karaman, M. Influence of non-/low plastic fines on cone penetration and liquefaction resistance. *Engineering Geology*. 2014;181:48–57.

24. Salgado R, Bandini P, Karim A. Shear strength and stiffness of silty sand. *Journal of Geotechnical and Geoenvironmental Engineering*. 2000 May;126(5):451-62.

25. Hazirbaba K, Rathje EM. Pore pressure generation of silty sands due to induced cyclic shear strains. *Journal of geotechnical and geoenvironmental engineering*. 2009 Dec;135(12):1892-905.

26. Amini F, Qi GZ. Liquefaction testing of stratified silty sands. *Journal of geotechnical and geoenvironmental engineering*. 2000 Mar;126(3):208-17.

27. Jradi L, Dupla JC, Seif El Dine B, Canou J. Effect of fine particles on cyclic liquefaction resistance of sands. *International Journal of Geotechnical Engineering*. 2020 Jan 9:1-6.

28. Polito CP, Martin II JR. Effects of non-plastic fines on the liquefaction resistance of sands. *Journal of Geotechnical and Geoenvironmental Engineering*. 2001 May;127(5):408-15.

29. Carraro JA, Bandini P, Salgado R. Liquefaction resistance of clean and non-plastic silty sands based on cone penetration resistance. *Journal of geotechnical and geoenvironmental engineering*. 2003 Nov;129(11):965-76.

30. Monkul MM, Kendir SB. Kumlarda göreceli sıkılık ile sismik sıvılaşma direnci arasındaki ilişkinin dinamik basit kesme deneyleri ile incelenmesi. *8. Uluslararası Geoteknik Sempozyumu*; 2019: İTÜ Süleyman Demirel Kültür Merkezi, İstanbul.

31. Monkul MM, Yamamuro JA. Influence of silt size and content on liquefaction behavior of sands. *Canadian Geotechnical Journal*. 2011 Jun;48(6):931-42.

32. Belkhatir M, Arab A, Schanz T, Missoum H, Della N. Laboratory study on the liquefaction resistance of sand-silt mixtures: effect of grading characteristics. *Granular Matter*. 2011 Oct 1;13(5):599-609.

33. Bayat M, Bayat E. Liquefaction potential of clean and silty sand. *7$^{th}$ National Congress on Civil Engineering*; 7-8 May 2013: University of Sistan and Baluchestan, Zahedan, Iran.

34. Monkul MM, Etminan E, Şenol A. Coupled influence of content, gradation and shape characteristics of silts on static liquefaction of loose silty sands. *Soil Dynamics and Earthquake Engineering*. 2017 Oct 1;101:12-26.

35. Oka LG, Dewoolkar M, Olson SM. Comparing laboratory-based liquefaction resistance of a sand with non-plastic fines with shear wave velocity-based field case histories. *Soil Dynamics and Earthquake Engineering*. 2018 Oct 1;113:162-73.

36. Wei X, Yang J, Zhou Y, and Chen Y. Influence of particle-size disparity on cyclic liquefaction resistance of silty sands. *Géotechnique Letters*. 2020 10:2, 1-7.

37. Eseller-Bayat EE, Monkul MM, Akin Ö, Yenigun S. The coupled influence of relative density, CSR, plasticity and content of fines on cyclic liquefaction resistance of sands. *Journal of Earthquake Engineering*. 2019 Jul 3;23(6):909-29.

38. Sönmezer, Y. B.., "Comparison of stress control and deformation controlled liquefaction tests in silty sands" *El-Cezerî Journal of Science and Engineering*, 2020, 7 (1); 322-337.

39. Porcino DD, Diano V, Tomasello G. Effect of non-plastic fines on cyclic shear strength of sand under an initial static shear stress. *InProceedings of China-Europe Conference on Geotechnical Engineering*, 2018 (pp. 597-601). Springer, Cham.

40. Doygun O, Brandes HG, Roy TT. Effect of gradation and non-plastic fines on monotonic and cyclic simple shear strength of silica sand. *Geotechnical and Geological Engineering*. 2019 Aug 15;37(4):3221-40.

41. Singh S. Liquefaction characteristics of silts. *Geotechnical & Geological Engineering*. 1996 Mar 1;14(1):1-9.

42. Bahadori H, Ghalandarzadeh A, Towhata I. Effect of non-plastic silt on the anisotropic behavior of sand. *Soils and foundations*. 2008;48(4):531-45.

43. Thevanayagam S, Shenthan T, Mohan S, Liang J. Undrained fragility of clean sands, silty sands, and sandy silts. *Journal of geotechnical and geoenvironmental engineering*. 2002 Oct;128(10):849-59.

44. Polito CP, Martin JR. A reconciliation of the effects of non-plastic fines on the liquefaction resistance of sands reported in the literature. *Earthquake Spectra*. 2003 Aug;19(3):635-51.

45. Xenaki VC, Athanasopoulos GA. Liquefaction resistance of sand–silt mixtures: an experimental investigation of the effect of fines. *Soil Dynamics and Earthquake Engineering*. 2003 Apr 1;23(3):1-2.

46. Papadopoulou A, Tika T. The effect of non-plastic fines on the liquefaction resistance of sands. *In4th International Conference on Earthquake Geotechnical Engineering*. Aristotle University of Thessaloniki, Thessaloniki; 2007 Jun 25 (pp. 25-28).

47. Porcino DD, Diano V. The influence of non-plastic fines on pore water pressure generation and undrained shear strength of sand-silt mixtures. *Soil Dynamics and Earthquake Engineering*. 2017 Oct 1;101:311-21.

48. Wang Y, Wang Y. Study of effects of fines content on liquefaction properties of sand. *InSoil Dynamics and Earthquake Engineering* 2010 (pp. 272-277).

49. Kokusho T. Liquefaction strengths of poorly-graded and well-graded granular soils investigated by lab tests. *InEarthquake géotechnical engineering* 2007 (pp. 159-184). Springer, Dordrecht.

50. Karim ME, Alam MJ. Effect of non-plastic silt content on the liquefaction behavior of sand–silt mixture. *Soil Dynamics and Earthquake Engineering*. 2014 Oct 1;65:142-50.

51. Karim ME, Alam MJ. Effect of nonplastic silt content on undrained shear strength of sand–silt mixtures. *International Journal of Geo-Engineering*. 2017 Dec 1;8(1):14.

52. Takch AE, Sadrekarimi A, El Naggar H. Cyclic resistance and liquefaction behavior of silt and sandy silt soils. *Soil Dynamics and Earthquake Engineering*. 2016 Apr 1;83:98-109.

53. ASTM D854. Standard test method for specific gravity of soil solids by water pycnometer. West Conshohocken, PA: American Society for Testing and Materials; 2005.

54. ASTM D422. Standard test method for particle-size analysis of soils. West Conshohocken, PA: American Society for Testing and Materials; 2005.

55. ASTM D2487. Standard practice for classification of soils for engineering purposes. West Conshohocken, PA: American Society for Testing and Materials; 2005.

56. Lade PV, Liggio CD, Yamamuro JA. Effects of non-plastic fines on minimum and maximum void ratios of sand. *Geotechnical testing journal*. 1998 Dec 1;21:336-47.

57. ASTM. Standard D4253: standard test methods for maximum index density and unit weight of soils using a vibratory table. In: Annual Book of ASTM standards. ASTM International West Conshohocken, PA; 2006.

58. ASTM. Standard D4254: standard test methods for maximum index density and unit weight of soils and calculation of relative density. Annual Book of ASTM standards. ASTM International West Conshohocken, PA; 2007.

59. JSSMFE, Test method for minimum and maximum densities of sands, in: JSF T-161, Japanese Society of Soil Mechanics and Foundation Engineering; 1990:174-75.

60. Monkul MM, Gültekin C, Gülver M, Akın Ö, Eseller-Bayat E. Estimation of liquefaction potential from dry and saturated sandy soils under drained constant volume cyclic simple shear loading. *Soil Dynamics and Earthquake Engineering*. 2015 Aug 1;75:27-36.

61. Monkul MM. On some of the factors influencing the fines' role on liquefaction of silty sands. *Geotechnical Special Publication No 225: State of the Art and Practice in Geotechnical Engineering, ASCE*; 2012:799-808.

62. Monkul MM, Yenigün Ş, Eseller-Bayat EE. Kuru kum numunelerin sismik sıvılaşma potansiyelinin dinamik basit kesme deneyinden belirlenmesi. *Uluslararası Katılımlı 7. Geoteknik Sempozyumu*; 22-23-24 Kasım 2017: İstanbul.

63. Dyvik R, Berre T, Lacasse S, Raadim B. Comparison of truly undrained and constant volume direct simple shear tests. *Géotechnique*. 1987; 37(1):3–10.

64. Bjerrum L, Landva A. Direct simple shear tests on a Norwegian quick clay. *Géotechnique*. 1966; 16(1):1–20.

65. Finn W.D.L, Vaid Y.P. Liquefaction potential from drained constant volume cyclic simple shear tests. *Proceedings of the sixth World Conference on Earthquake Engineering*; 1977; (3):2157–2162: New Delhi.

66. Yenigün Ş. Sismik sıvılaşma deney numuneleri için otomatik huni tasarımı ve deney sonuçlarına etkisi. Yüksek Lisans Tezi, İstanbul Teknik Üniversitesi, 2018.

67. Idriss IM, Boulanger RW. Soil liquefaction during earthquakes. *Earthquake Engineering Research Institute*; 2008.

68. Ishihara K. Liquefaction and flow failure during earthquakes. *Geotechnique*. 1993 Sep;43(3):351-451.

69. Polito CP. The effects of non-plastic and plastic fines on the liquefaction of sandy soils. Doctoral dissertation, Ph.D. thesis, Virginia Polytechnic Institute and State University, 1999.

# APPENDIX A: CHARACTERIZATION OF TESTED SOILS

Table A.1. Determination of specific gravity of sand

| Specific Gravity of Solids Determination (ASTM D 854-02) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Test No:** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** |
| **Calibration of Picynometer** | Mass of Picynometer, g | 140.89 | 173.53 | 173.53 | 173.47 | 173.47 | 173.47 | 173.47 | 173.47 |
| | Mass of Picynometer and Calibration Water, g | 638.22 | 670.88 | 670.84 | 670.86 | 670.94 | 670.86 | 670.67 | 670.72 |
| | Calibration Temperature of Water, **°C** | 23.6 | 23.3 | 22.9 | 20 | 20.3 | 20.3 | 23.1 | 22.5 |
| | Density of Water, Calibration Temperature, (g/mL) | 0.99740 | 0.99747 | 0.99756 | 0.99821 | 0.99814 | 0.99814 | 0.99752 | 0.99766 |
| | Volume of Picynometer, mL | 498.63 | 498.61 | 498.53 | 498.28 | 498.40 | 498.32 | 498.44 | 498.42 |
| **Calculation** | Mass of Soil, g | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 |
| | Test Temperature of Water, **°C** | 24.2 | 23.2 | 23.4 | 21.9 | 22.4 | 22.8 | 23.8 | 24.6 |
| | Density of Water at Test Temperature, (g/mL) | 0.9973 | 0.99749 | 0.99745 | 0.99780 | 0.99768 | 0.99759 | 0.99735 | 0.99715 |
| | Mass of Picynometer and Water at Test Temperature, g | 638.15 | 670.89 | 670.79 | 670.66 | 670.71 | 670.59 | 670.59 | 670.47 |
| | Mass of Picynometer, Water and Soil at Test Temperature, g | 700.35 | 732.90 | 732.86 | 732.91 | 732.83 | 732.87 | 732.71 | 732.66 |
| | Specific Gravity at Test Temperature, $\mathbf{G_t}$ | **2.646** | **2.632** | **2.637** | **2.649** | **2.640** | **2.651** | **2.640** | **2.645** |
| | Temperature Coefficient, **K** | 0.999 | 0.99929 | 0.99924 | 0.99959 | 0.99947 | 0.99938 | 0.99914 | 0.99894 |
| | Specific Gravity at 20°C, $\mathbf{G_{20°C}}$ | **2.643** | **2.630** | **2.635** | **2.648** | **2.638** | **2.650** | **2.638** | **2.642** |
| **Average of Specific Gravity of Solids, $\mathbf{G_{avg}}$** | | **2.641** | | | | | | | |

Table A.2. Determination of specific gravity of YET Silt

| Specific Gravity of Solids Determination (ASTM D 854-02) | | | | | |
|---|---|---|---|---|---|
| | **Test No:** | **1** | **2** | **3** | **4** |
| **Calibration of Picynometer** | Mass of Picynometer, g | 173.53 | 173.53 | 173.53 | 173.53 |
| | Mass of Picynometer and Calibration Water, g | 670.75 | 670.79 | 670.81 | 670.59 |
| | Calibration Temperature of Water, **°C** | 22.7 | 21.9 | 21.9 | 23.1 |
| | Density of Water, Calibration Temperature, (g/mL) | 0.99761 | 0.99780 | 0.99780 | 0.99752 |
| | Volume of Picynometer, mL | 498.41 | 498.36 | 498.38 | 498.30 |
| **Calculation** | Mass of Soil, g | 50.00 | 50.00 | 50.00 | 50.00 |
| | Test Temperature of Water, **°C** | 23.0 | 23.4 | 23.5 | 23.2 |
| | Density of Water at Test Temperature, (g/mL) | 0.99754 | 0.99745 | 0.99742 | 0.99749 |
| | Mass of Picynometer and Water at Test Temperature, g | 670.72 | 670.62 | 670.62 | 670.58 |
| | Mass of Picynometer, Water and Soil at Test Temperature, g | 701.80 | 701.73 | 701.74 | 701.66 |
| | Specific Gravity at Test Temperature, $\mathbf{G_t}$ | **2.643** | **2.648** | **2.648** | **2.643** |
| | Temperature Coefficient, **K** | 0.99933 | 0.99924 | 0.99921 | 0.99929 |
| | Specific Gravity at 20°C, $\mathbf{G_{20°C}}$ | **2.642** | **2.646** | **2.646** | **2.642** |
| **Average of Specific Gravity of Solids, $\mathbf{G_{avg}}$** | | **2.644** | | | |

Table A.3. Sieve analysis for sand

| Sieve Analysis: ASTM D422-63 | | | | |
|---|---|---|---|---|
| **Sieve No.** | **Sieve opening (mm)** | **Mass retained on each sieve (g)** | **Mass passing (g)** | **Percent finer by weight (%)** |
| **20** | **0.850** | **0** | **670** | **100.00** |
| **30** | **0.600** | **2.2** | **667.8** | **99.67** |
| **40** | **0.425** | **11.7** | **656.1** | **97.93** |
| **50** | **0.3** | **57.1** | **599** | **89.40** |
| **70** | **0.212** | **371.59** | **227.41** | **33.94** |
| **100** | **0.15** | **197.85** | **29.56** | **4.41** |
| **140** | **0.106** | **28.13** | **1.43** | **0.21** |
| **200** | **0.075** | **1.11** | **0.32** | **0.05** |
| **Pan** | | **0.32** | **0** | **0.00** |
| | **$M_{TOT}$ =** | **670** | | |

Table A.4. Hydrometer Analysis

| Hydrometer Analysis: ASTM D422-63 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Hydrometer Type: 152 H** | | | | **Specific Gravity of Silt: 2.644** | | | **a (correction factor): 1.002** | | | |
| **Date** | **Time** | **Elapsed Time T (min)** | **Actual Hydrometer Reading** | **Composite Correction** | **Corrected Reading, R** | **Temperature (°C)** | **Effective Depth of Hydrometer, L (cm)** | **Value of K** | **Particle Diameter D (mm)** | **Percent Finer (percent)** |
| 11.09.2019 | 10.23.00 | 0 | | | | | | | | |
| 11.09.2019 | 10.23.15 | 0.25 | 48 | 0.5 | 48.5 | 26 | 8.3 | 0.01276 | 0.0735 | 97.2 |
| 11.09.2019 | 10.23.30 | 0.5 | 45 | 0.5 | 45.5 | 26 | 8.9 | 0.01276 | 0.0538 | 91.2 |
| 11.09.2019 | 10.24.00 | 1 | 42 | 0.5 | 42.5 | 26 | 9.3 | 0.01276 | 0.0389 | 85.2 |
| 11.09.2019 | 10.25.00 | 2 | 35 | 0.5 | 35.5 | 26 | 10.5 | 0.01276 | 0.0292 | 71.1 |
| 11.09.2019 | 10.28.00 | 5 | 26 | 0.5 | 26.5 | 26 | 11.9 | 0.01276 | 0.0197 | 53.1 |
| 11.09.2019 | 10.38.00 | 15 | 18.5 | 0.5 | 19 | 26 | 13.2 | 0.01276 | 0.0120 | 38.1 |
| 11.09.2019 | 10.53.00 | 30 | 14.5 | 0.5 | 15 | 26 | 13.8 | 0.01276 | 0.0087 | 30.1 |
| 11.09.2019 | 11.23.00 | 60 | 10.5 | 0.5 | 11 | 25 | 14.5 | 0.0129 | 0.0063 | 22.0 |
| 11.09.2019 | 12.23.00 | 120 | 8.5 | 0.5 | 9 | 24 | 14.8 | 0.01305 | 0.0046 | 18.0 |
| 11.09.2019 | 14.23.00 | 240 | 6.5 | 0.5 | 7 | 23 | 15.2 | 0.01321 | 0.0033 | 14.0 |
| 12.09.2019 | 10.23.00 | 1440 | 5 | 0.5 | 5.5 | 21 | 15.4 | 0.01351 | 0.0014 | 11.0 |

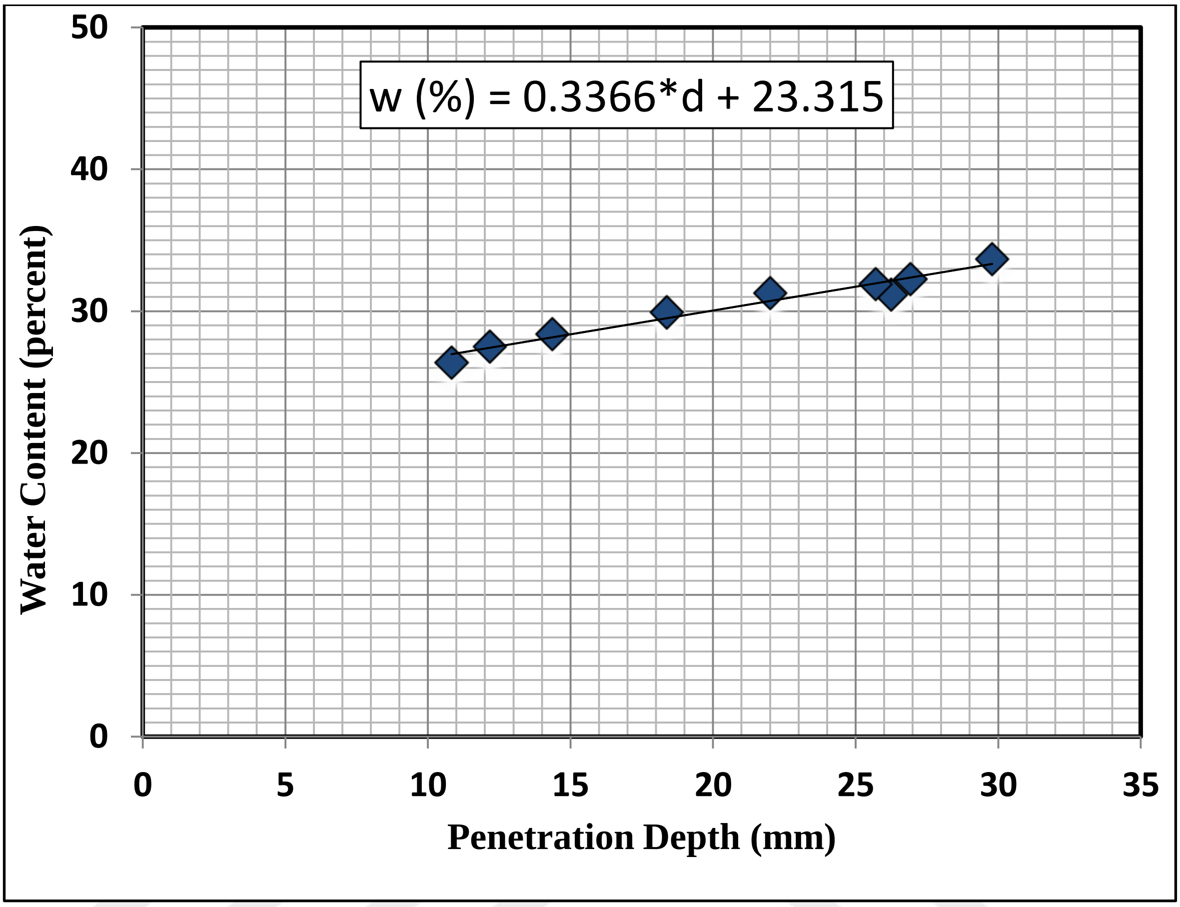


Figure A.1. Fall cone test for determination of liquid limit

Table A.5. Minimum and maximum void ratios for clean sand

| Type | | $V_{read}$ on clinder each edge on cylinder (mL) | | | | $V_{read}$ on cylinder (mL) | $M_{sand}$ in cylinder (g) | $Gs=r_s$ (g/mL) | $V_{sand}$ (mL) | $e_{min}$ | $e_{max}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Silica Sand (Clean Sand) | emin | 540.00 | 530.43 | 526.09 | 543.48 | 535.00 | 822.0 | 2.641 | 311.2 | 0.719 | - |
| | emax | 639.13 | 628.26 | 634.78 | 643.48 | 636.41 | 822.0 | 2.641 | 311.2 | - | 1.045 |
| | | 641.30 | 632.61 | 630.43 | 634.78 | 634.78 | 822.0 | 2.641 | 311.2 | - | 1.039 |
| | | 636.96 | 623.91 | 628.26 | 643.48 | 633.15 | 822.0 | 2.641 | 311.2 | - | 1.034 |
| | | 640.00 | 621.74 | 628.26 | 652.17 | 635.54 | 822.0 | 2.641 | 311.2 | - | 1.042 |
| | | 639.13 | 626.09 | 634.78 | 641.30 | 635.33 | 822.0 | 2.641 | 311.2 | - | 1.041 |
| | | | | | | | | | | | **1.040** |
| | emin | 534.78 | 523.91 | 526.09 | 550.00 | 533.70 | 822.0 | 2.641 | 311.2 | 0.715 | - |
| | emax | 634.78 | 610.87 | 632.61 | 652.17 | 632.61 | 822.0 | 2.641 | 311.2 | - | 1.033 |
| | | 620.00 | 613.04 | 632.61 | 652.17 | 629.46 | 822.0 | 2.641 | 311.2 | - | 1.022 |
| | | 640.00 | 643.48 | 623.91 | 623.91 | 632.83 | 822.0 | 2.641 | 311.2 | - | 1.033 |
| | | 647.83 | 617.39 | 626.09 | 643.48 | 633.70 | 822.0 | 2.641 | 311.2 | - | 1.036 |
| | | 632.61 | 628.26 | 639.26 | 639.13 | 641.30 | 822.0 | 2.641 | 311.2 | - | 1.060 |
| | | | | | | | | | | | **1.037** |
| | | | | | | | | | **Avg=** | **0.717** | **1.039** |

Table A.6. Minimum and maximum void ratios for silty sand with 5% FC

| Type | | $V_{read}$ on clinder each edge on cylinder (mL) | | | | $V_{read}$ on cylinder (mL) | $M_{sand}$ in cylinder (g) | $Gs=r_s$ (g/mL) | $V_{sand}$ (mL) | $e_{min}$ | $e_{max}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Silica Sand + %5 YET Silt | emin | 504.35 | 513.04 | 500.00 | 515.22 | 508.15 | 822.0 | 2.641 | 311.2 | 0.633 | - |
| | emax | 608.70 | 613.04 | 630.43 | 626.09 | 619.57 | 822.0 | 2.641 | 311.2 | - | 0.991 |
| | | 613.04 | 606.52 | 630.43 | 630.43 | 620.11 | 822.0 | 2.641 | 311.2 | - | 0.992 |
| | | 608.70 | 606.52 | 621.74 | 626.09 | 615.76 | 822.0 | 2.641 | 311.2 | - | 0.978 |
| | | 600.00 | 634.78 | 639.13 | 604.35 | 619.57 | 822.0 | 2.641 | 311.2 | - | 0.991 |
| | | 610.87 | 639.13 | 632.61 | 604.35 | 621.74 | 822.0 | 2.641 | 311.2 | - | 0.998 |
| | | | | | | | | | | | **0.990** |
| | emin | 513.04 | 504.35 | 502.17 | 510.87 | 507.61 | 822.0 | 2.641 | 311.2 | 0.631 | - |
| | emax | 604.35 | 606.52 | 639.13 | 630.43 | 620.11 | 822.0 | 2.641 | 311.2 | - | 0.992 |
| | | 621.74 | 641.30 | 600.00 | 600.00 | 615.76 | 822.0 | 2.641 | 311.2 | - | 0.978 |
| | | 630.43 | 628.26 | 604.35 | 613.04 | 619.02 | 822.0 | 2.641 | 311.2 | - | 0.989 |
| | | 615.22 | 626.09 | 626.09 | 621.74 | 622.29 | 822.0 | 2.641 | 311.2 | - | 0.999 |
| | | 626.09 | 628.26 | 613.04 | 613.04 | 620.11 | 822.0 | 2.641 | 311.2 | - | 0.992 |
| | | | | | | | | | | | **0.990** |
| | | | | | | | | | **Avg=** | **0.632** | **0.990** |

Table A.7. Minimum and maximum void ratios for silty sand with 15% FC

| Type | | $V_{read}$ on clinder each edge on cylinder (mL) | | | | $V_{read}$ on cylinder (mL) | $M_{sand}$ in cylinder (g) | $Gs=r_s$ (g/mL) | $V_{sand}$ (mL) | $e_{min}$ | $e_{max}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Silica Sand + %15 YET Silt | emin | 486.96 | 480.43 | 478.26 | 478.26 | 480.98 | 822.0 | 2.641 | 311.2 | 0.545 | - |
| | emax | 565.21 | 608.70 | 608.70 | 578.26 | 590.22 | 822.0 | 2.641 | 311.2 | - | 0.896 |
| | | 595.65 | 586.96 | 578.26 | 600.00 | 590.22 | 822.0 | 2.641 | 311.2 | - | 0.896 |
| | | 600.00 | 582.61 | 586.96 | 604.35 | 593.48 | 822.0 | 2.641 | 311.2 | - | 0.907 |
| | | 600.00 | 602.17 | 580.43 | 578.26 | 590.22 | 822.0 | 2.641 | 311.2 | - | 0.896 |
| | | 591.30 | 591.30 | 591.30 | 595.65 | 592.39 | 822.0 | 2.641 | 311.2 | - | 0.903 |
| | | | | | | | | | | | **0.900** |
| | emin | 482.61 | 482.61 | 469.57 | 482.61 | 479.35 | 822.0 | 2.641 | 311.2 | 0.540 | - |
| | emax | 573.91 | 586.96 | 602.17 | 600.00 | 590.76 | 822.0 | 2.641 | 311.2 | - | 0.898 |
| | | 591.30 | 589.13 | 591.30 | 604.35 | 594.02 | 822.0 | 2.641 | 311.2 | - | 0.909 |
| | | 600.00 | 600.00 | 586.96 | 591.30 | 594.57 | 822.0 | 2.641 | 311.2 | - | 0.910 |
| | | 595.65 | 606.52 | 582.61 | 580.43 | 591.30 | 822.0 | 2.641 | 311.2 | - | 0.900 |
| | | 578.26 | 586.96 | 600.00 | 600.00 | 591.31 | 822.0 | 2.641 | 311.2 | - | 0.900 |
| | | | | | | | | | | | **0.903** |
| | | | | | | | | | **Avg=** | **0.543** | **0.902** |

Table A.8. Minimum and maximum void ratios for silty sand with 35% FC

| Type | | $V_{read}$ on clinder each edge on cylinder (mL) | | | | $V_{read}$ on cylinder (mL) | $M_{sand}$ in cylinder (g) | $Gs=r_s$ (g/mL) | $V_{sand}$ (mL) | $e_{min}$ | $e_{max}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Silica Sand + %35 YET Silt | emin | 460.00 | 456.52 | 443.48 | 454.35 | 453.59 | 822.0 | 2.642 | 311.1 | 0.458 | - |
| | emax | 569.57 | 563.04 | 573.91 | 578.26 | 571.20 | 822.0 | 2.642 | 311.1 | - | 0.836 |
| | | 571.74 | 567.39 | 578.26 | 565.22 | 570.65 | 822.0 | 2.642 | 311.1 | - | 0.834 |
| | | 571.74 | 560.87 | 569.57 | 571.74 | 568.48 | 822.0 | 2.642 | 311.1 | - | 0.827 |
| | | 573.91 | 578.26 | 569.57 | 567.39 | 572.28 | 822.0 | 2.642 | 311.1 | - | 0.839 |
| | | 565.22 | 582.61 | 571.74 | 556.52 | 569.02 | 822.0 | 2.642 | 311.1 | - | 0.829 |
| | | | | | | | | | | | **0.833** |
| | emin | 447.83 | 452.17 | 452.17 | 460.87 | 453.26 | 822.0 | 2.642 | 311.1 | 0.457 | - |
| | emax | 569.57 | 565.22 | 573.91 | 580.43 | 572.28 | 822.0 | 2.642 | 311.1 | - | 0.839 |
| | | 565.22 | 565.22 | 565.22 | 569.57 | 566.31 | 822.0 | 2.642 | 311.1 | - | 0.820 |
| | | 547.83 | 560.87 | 600.00 | 578.26 | 571.74 | 822.0 | 2.642 | 311.1 | - | 0.838 |
| | | 573.91 | 526.09 | 580.43 | 604.35 | 571.20 | 822.0 | 2.642 | 311.1 | - | 0.836 |
| | | 565.82 | 573.91 | 560.87 | 578.26 | 569.72 | 822.0 | 2.642 | 311.1 | - | 0.831 |
| | | | | | | | | | | | **0.833** |
| | | | | | | | | | **Avg=** | **0.457** | **0.833** |

Table A.9. Minimum and maximum void ratios for YET Silt

| Type | | $V_{read}$ on clinder each edge on cylinder (mL) | | | | $V_{read}$ on cylinder (mL) | $M_{sand}$ in cylinder (g) | $Gs=r_s$ (g/mL) | $V_{sand}$ (mL) | $e_{min}$ | $e_{max}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| %100 YET Silt | emin | 569.57 | 569.57 | 573.91 | 569.57 | 570.66 | 822.0 | 2.644 | 310.9 | 0.836 | - |
| | emax | 747.83 | 765.22 | 756.52 | 752.17 | 755.44 | 822.0 | 2.644 | 310.9 | - | 1.430 |
| | | 763.04 | 765.22 | 780.43 | 760.87 | 767.39 | 822.0 | 2.644 | 310.9 | - | 1.468 |
| | | 747.83 | 765.22 | 786.96 | 760.87 | 765.22 | 822.0 | 2.644 | 310.9 | - | 1.461 |
| | | 752.17 | 776.09 | 773.91 | 782.61 | 771.20 | 822.0 | 2.644 | 310.9 | - | 1.481 |
| | | 767.39 | 767.39 | 771.74 | 767.39 | 768.48 | 822.0 | 2.644 | 310.9 | - | 1.472 |
| | | | | | | | | | | | **1.462** |
| | emin | 569.57 | 569.57 | 573.91 | 560.87 | 568.48 | 822.0 | 2.644 | 310.9 | 0.829 | - |
| | emax | 752.17 | 782.61 | 780.43 | 773.91 | 772.28 | 822.0 | 2.644 | 310.9 | - | 1.484 |
| | | 750.00 | 793.48 | 747.83 | 786.96 | 769.57 | 822.0 | 2.644 | 310.9 | - | 1.475 |
| | | 765.22 | 791.30 | 771.74 | 769.57 | 774.46 | 822.0 | 2.644 | 310.9 | - | 1.491 |
| | | 760.00 | 760.87 | 776.09 | 771.74 | 767.18 | 822.0 | 2.644 | 310.9 | - | 1.468 |
| | | 765.22 | 778.26 | 782.61 | 767.39 | 773.37 | 822.0 | 2.644 | 310.9 | - | 1.488 |
| | | | | | | | | | | | **1.481** |
| | emax | 756.52 | 756.52 | 753.91 | 765.21 | 758.04 | 822.0 | 2.644 | 310.9 | - | 1.438 |
| | | 760.00 | 773.91 | 765.21 | 773.91 | 768.26 | 822.0 | 2.644 | 310.9 | - | 1.471 |
| | | 767.39 | 773.91 | 786.96 | 760.87 | 772.28 | 822.0 | 2.644 | 310.9 | - | 1.484 |
| | | 767.39 | 765.21 | 776.09 | 776.09 | 771.20 | 822.0 | 2.644 | 310.9 | - | 1.481 |
| | | 760.00 | 776.09 | 778.26 | 769.57 | 770.98 | 822.0 | 2.644 | 310.9 | - | 1.480 |
| | | | | | | | | | | | **1.471** |
| | | | | | | | | | **Avg=** | **0.832** | **1.471** |

# APPENDIX B: CDSS TEST RESULTS

Table B.1. CDSS tests for clean sand under CSR = 0.1

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 68 | 94.8 | 1.039 | 0.717 | 48.86 |
| 2 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 28 | 98.3 | 1.039 | 0.717 | 37.39 |
| 3 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 27 | 94.3 | 1.039 | 0.717 | 34.48 |
| 4 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 39 | 96.7 | 1.039 | 0.717 | 44.85 |
| 5 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 261 | 96.2 | 1.039 | 0.717 | 70.74 |
| 6 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 57 | 94.4 | 1.039 | 0.717 | 54.73 |
| 7 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 80 | 93.1 | 1.039 | 0.717 | 49.33 |
| 8 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 75 | 98.1 | 1.039 | 0.717 | 60.90 |
| 9 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 103 | 95.0 | 1.039 | 0.717 | 52.41 |
| 10 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 78 | 95.6 | 1.039 | 0.717 | 53.05 |
| 11 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 92 | 96.0 | 1.039 | 0.717 | 61.59 |
| 12 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 284 | 96.2 | 1.039 | 0.717 | 79.26 |
| 13 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 116 | 94.9 | 1.039 | 0.717 | 73.80 |
| 14 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 76 | 99.0 | 1.039 | 0.717 | 68.61 |
| 15 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 75 | 96.3 | 1.039 | 0.717 | 64.99 |
| 16 | Clean Sand | 0.1 | 100 | 2.641 | 100 | 75 | 97.6 | 1.039 | 0.717 | 63.60 |

Table B.2. CDSS tests for clean sand under CSR = 0.12

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 17 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 15 | 93.8 | 1.039 | 0.717 | 36.64 |
| 18 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 12 | 96.0 | 1.039 | 0.717 | 31.25 |
| 19 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 19 | 90.9 | 1.039 | 0.717 | 40.46 |
| 20 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 18 | 91.9 | 1.039 | 0.717 | 43.06 |
| 21 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 11 | 94.5 | 1.039 | 0.717 | 48.27 |
| 22 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 44 | 88.4 | 1.039 | 0.717 | 61.73 |
| 23 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 45 | 87.2 | 1.039 | 0.717 | 46.08 |
| 24 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 107 | 88.4 | 1.039 | 0.717 | 63.92 |
| 25 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 47 | 86.1 | 1.039 | 0.717 | 55.97 |
| 26 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 30 | 88.1 | 1.039 | 0.717 | 55.53 |
| 27 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 124 | 88.2 | 1.039 | 0.717 | 73.94 |
| 28 | Clean Sand | 0.12 | 100 | 2.641 | 100 | 72 | 88.1 | 1.039 | 0.717 | 59.30 |

Table B.3. CDSS tests for clean sand under CSR = 0.14

| **Test No** | **Soil Type** | **CSR** | **Vertical Stress (kPa)** | **$G_s$** | **$W_{solid}$ (g)** | **$N_L$** | **Excess Pore Pressure (kPa)** | **$e_{max}$** | **$e_{min}$** | **$D_r$ (%)** |
|---|---|---|---|---|---|---|---|---|---|---|
| 29 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 10 | 95.8 | 1.039 | 0.717 | 43.40 |
| 30 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 10 | 96.3 | 1.039 | 0.717 | 44.23 |
| 31 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 12 | 93.8 | 1.039 | 0.717 | 42.06 |
| 32 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 14 | 92.8 | 1.039 | 0.717 | 58.83 |
| 33 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 12 | 88.7 | 1.039 | 0.717 | 61.01 |
| 34 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 22 | 93.4 | 1.039 | 0.717 | 70.45 |
| 35 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 29 | 89.0 | 1.039 | 0.717 | 69.23 |
| 36 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 13 | 97.6 | 1.039 | 0.717 | 57.86 |
| 37 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 22 | 88.1 | 1.039 | 0.717 | 70.04 |
| 38 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 20 | 88.3 | 1.039 | 0.717 | 61.63 |
| 39 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 15 | 87.9 | 1.039 | 0.717 | 58.23 |
| 40 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 9 | 97.8 | 1.039 | 0.717 | 49.00 |
| 41 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 8 | 100.0 | 1.039 | 0.717 | 49.10 |
| 42 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 8 | 99.0 | 1.039 | 0.717 | 37.31 |
| 43 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 17 | 88.1 | 1.039 | 0.717 | 58.85 |
| 44 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 7 | 93.7 | 1.039 | 0.717 | 29.43 |
| 45 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 11 | 97.1 | 1.039 | 0.717 | 50.37 |
| 46 | Clean Sand | 0.14 | 100 | 2.641 | 100 | 17 | 94.5 | 1.039 | 0.717 | 57.70 |

Table B.4. CDSS tests for silty sand with 5% FC under CSR = 0.1

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 47 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 33 | 100.4 | 0.990 | 0.632 | 37.59 |
| 48 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 37 | 100.1 | 0.990 | 0.632 | 41.88 |
| 49 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 88 | 96.0 | 0.990 | 0.632 | 55.00 |
| 50 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 33 | 97.8 | 0.990 | 0.632 | 34.80 |
| 51 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 159 | 93.1 | 0.990 | 0.632 | 54.15 |
| 52 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 444 | 96.6 | 0.990 | 0.632 | 61.30 |
| 53 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 90 | 96.7 | 0.990 | 0.632 | 53.17 |
| 54 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 144 | 97.5 | 0.990 | 0.632 | 52.67 |
| 55 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 107 | 100.0 | 0.990 | 0.632 | 46.60 |
| 56 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 61 | 95.6 | 0.990 | 0.632 | 43.07 |
| 57 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 89 | 97.0 | 0.990 | 0.632 | 47.43 |
| 58 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 501 | 93.6 | 0.990 | 0.632 | 59.03 |

Table B.5. CDSS tests for silty sand with 5% FC under CSR = 0.12

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 59 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 13 | 93.8 | 0.990 | 0.632 | 27.96 |
| 60 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 17 | 98.2 | 0.990 | 0.632 | 32.13 |
| 61 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 19 | 94.8 | 0.990 | 0.632 | 36.98 |
| 62 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 37 | 87.3 | 0.990 | 0.632 | 47.30 |
| 63 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 90 | 89.1 | 0.990 | 0.632 | 57.90 |
| 64 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 81 | 89.1 | 0.990 | 0.632 | 54.79 |

Table B.6. CDSS tests for silty sand with 5% FC under CSR = 0.14

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 65 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 7 | 100.0 | 0.990 | 0.632 | 32.22 |
| 66 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 9 | 95.7 | 0.990 | 0.632 | 38.33 |
| 67 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 12 | 88.3 | 0.990 | 0.632 | 44.10 |
| 68 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 21 | 87.1 | 0.990 | 0.632 | 51.26 |
| 69 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 22 | 87.3 | 0.990 | 0.632 | 47.69 |
| 70 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 23 | 86.6 | 0.990 | 0.632 | 53.87 |
| 71 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 40 | 86.6 | 0.990 | 0.632 | 54.31 |
| 72 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 87 | 85.8 | 0.990 | 0.632 | 54.51 |
| 73 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 13 | 88.8 | 0.990 | 0.632 | 38.43 |
| 74 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 25 | 88.5 | 0.990 | 0.632 | 46.06 |
| 75 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 127 | 87.1 | 0.990 | 0.632 | 67.04 |

Table B.7. CDSS tests for silty sand with 15% FC under CSR = 0.1

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 76 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 40 | 99.9 | 0.902 | 0.543 | 50.91 |
| 77 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 37 | 98.7 | 0.902 | 0.543 | 48.96 |
| 78 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 38 | 97.8 | 0.902 | 0.543 | 51.17 |
| 79 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 53 | 97.8 | 0.902 | 0.543 | 53.78 |
| 80 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 289 | 96.4 | 0.902 | 0.543 | 69.01 |
| 81 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 241 | 96.9 | 0.902 | 0.543 | 67.93 |
| 82 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 159 | 96.0 | 0.902 | 0.543 | 63.68 |
| 83 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 427 | 96.3 | 0.902 | 0.543 | 70.18 |
| 84 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 48 | 97.5 | 0.902 | 0.543 | 43.07 |
| 85 | Silty Sand | 0.1 | 100 | 2.641 | 100 | 27 | 100.0 | 0.902 | 0.543 | 40.40 |

Table B.8. CDSS tests for silty sand with 15% FC under CSR = 0.12

| Test No | Soil Type | CSR | Vertical Stress (kPa) | $G_s$ | $W_{solid}$ (g) | $N_L$ | Excess Pore Pressure (kPa) | $e_{max}$ | $e_{min}$ | $D_r$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 86 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 13 | 98.1 | 0.902 | 0.543 | 43.75 |
| 87 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 22 | 93.5 | 0.902 | 0.543 | 46.02 |
| 88 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 93 | 92.7 | 0.902 | 0.543 | 66.49 |
| 89 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 46 | 95.1 | 0.902 | 0.543 | 53.76 |
| 90 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 12 | 100.0 | 0.902 | 0.543 | 37.89 |
| 91 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 128 | 92.8 | 0.902 | 0.543 | 65.29 |
| 92 | Silty Sand | 0.12 | 100 | 2.641 | 100 | 65 | 91.3 | 0.902 | 0.543 | 62.83 |

Table B.9. CDSS tests for silty sand with 15% FC under CSR = 0.14

| **Test No** | **Soil Type** | **CSR** | **Vertical Stress (kPa)** | **$G_s$** | **$W_{solid}$ (g)** | **$N_L$** | **Excess Pore Pressure (kPa)** | **$e_{max}$** | **$e_{min}$** | **$D_r$ (%)** |
|---|---|---|---|---|---|---|---|---|---|---|
| 93 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 15 | 98.7 | 0.902 | 0.543 | 47.12 |
| 94 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 9 | 95.7 | 0.902 | 0.543 | 45.24 |
| 95 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 10 | 88.7 | 0.902 | 0.543 | 48.83 |
| 96 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 26 | 90.1 | 0.902 | 0.543 | 53.06 |
| 97 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 23 | 88.4 | 0.902 | 0.543 | 57.41 |
| 98 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 35 | 88.1 | 0.902 | 0.543 | 65.95 |
| 99 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 5 | 98.5 | 0.902 | 0.543 | 36.45 |
| 100 | Silty Sand | 0.14 | 100 | 2.641 | 100 | 13 | 88.2 | 0.902 | 0.543 | 55.75 |

Table B.10. CDSS tests for silty sand with 35% FC under CSR = 0.1

| **Test No** | **Soil Type** | **CSR** | **Vertical Stress (kPa)** | **$G_s$** | **$W_{solid}$ (g)** | **$N_L$** | **Excess Pore Pressure (kPa)** | **$e_{max}$** | **$e_{min}$** | **$D_r$ (%)** |
|---|---|---|---|---|---|---|---|---|---|---|
| 101 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 21 | 99.2 | 0.833 | 0.457 | 63.63 |
| 102 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 40 | 97.7 | 0.833 | 0.457 | 63.25 |
| 103 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 35 | 98.1 | 0.833 | 0.457 | 71.27 |
| 104 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 66 | 95.2 | 0.833 | 0.457 | 78.55 |
| 105 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 23 | 100.0 | 0.833 | 0.457 | 63.59 |
| 106 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 325 | 95.0 | 0.833 | 0.457 | 87.23 |
| 107 | Silty Sand | 0.1 | 100 | 2.642 | 100 | 108 | 95.8 | 0.833 | 0.457 | 81.36 |

Table B.11. CDSS tests for silty sand with 35% FC under CSR = 0.12

| **Test No** | **Soil Type** | **CSR** | **Vertical Stress (kPa)** | **$G_s$** | **$W_{solid}$ (g)** | **$N_L$** | **Excess Pore Pressure (kPa)** | **$e_{max}$** | **$e_{min}$** | **$D_r$ (%)** |
|---|---|---|---|---|---|---|---|---|---|---|
| 108 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 11 | 99.8 | 0.833 | 0.457 | 62.37 |
| 109 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 11 | 96.1 | 0.833 | 0.457 | 62.60 |
| 110 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 27 | 95.6 | 0.833 | 0.457 | 75.27 |
| 111 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 31 | 95.2 | 0.833 | 0.457 | 75.33 |
| 112 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 34 | 92.0 | 0.833 | 0.457 | 75.91 |
| 113 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 88 | 93.9 | 0.833 | 0.457 | 90.55 |
| 114 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 25 | 96.9 | 0.833 | 0.457 | 79.64 |
| 115 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 16 | 94.3 | 0.833 | 0.457 | 74.51 |
| 116 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 16 | 93.6 | 0.833 | 0.457 | 69.58 |
| 117 | Silty Sand | 0.12 | 100 | 2.642 | 100 | 67 | 93.2 | 0.833 | 0.457 | 83.38 |

Table B.12. CDSS tests for silty sand with 35% FC under CSR = 0.14

| **Test No** | **Soil Type** | **CSR** | **Vertical Stress (kPa)** | **$G_s$** | **$W_{solid}$ (g)** | **$N_L$** | **Excess Pore Pressure (kPa)** | **$e_{max}$** | **$e_{min}$** | **$D_r$ (%)** |
|---|---|---|---|---|---|---|---|---|---|---|
| 118 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 10 | 91.5 | 0.833 | 0.457 | 67.68 |
| 119 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 9 | 95.2 | 0.833 | 0.457 | 75.72 |
| 120 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 15 | 90.6 | 0.833 | 0.457 | 78.43 |
| 121 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 24 | 87.9 | 0.833 | 0.457 | 90.65 |
| 122 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 16 | 92.1 | 0.833 | 0.457 | 82.53 |
| 123 | Silty Sand | 0.14 | 100 | 2.642 | 100 | 7 | 100.0 | 0.833 | 0.457 | 64.67 |