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M.Sc. in Civil Engineering

SÜLEYMAN DEMİR

UNIVERSITY OF GAZİANTEP

GRADUATE SCHOOL OF

NATURAL & APPLIED SCIENCES

LIQUEFACTION POTENTIAL OF SAND WITH FINES USING BENDER ELEMENT TEST

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BY

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Liquefaction Potential of Sand with Fines Using Bender Element Test

M.Sc. Thesis

in

Civil Engineering

University of Gaziantep

Supervisor

Prof. Dr. Ali Fırat ÇABALAR

by

Süleyman DEMİR

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Name of the student: Süleyman DEMİR

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Approval of the Graduate School of Natural and Applied Sciences

Prof. Dr. Ahmet Necmeddin YAZICI
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Abdulkadir ÇEVİK
Head of Department

This is to certify that we have read this thesis and that in our majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Prof. Dr. Ali Fırat ÇABALAR
Supervisor

Examining Committee Members:	Signature
Assist. Prof. Dr. Eyyüb KARAKAN	....
Assoc. Prof. Dr. Esra Mete GÜNEYİSİ	....
Prof. Dr. Ali Fırat ÇABALAR	....

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Süleyman DEMİR

ABSTRACT

LIQUEFACTION POTENTIAL OF SAND WITH FINES USING BENDER ELEMENT TEST

DEMİR, Süleyman
M.Sc. in Civil Engineering
Supervisor: Prof. Dr. Ali Fırat ÇABALAR
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This study aims to evaluate the maximum dynamic shear modulus (G_{max}) based liquefaction potentials of sand with various fines using bender element test. The fines (CL) was added to two different particle gradations (0.15-0.30 mm and 1.0-2.0 mm) of clean sand having distinct shapes (rounded and angular) at mixture ratio of 5%, 10%, 15%, 20%, 30% and 40%. The results indicated that the G_{max} values and liquefaction resistance were decreased up to 20% fines content then increased. The liquefaction resistance of the mixtures with coarse sand grains were found to be greater than those with fine sand grains at a given fines content.

Keywords: Liquefaction, bender element, sand, fines.

ÖZET

BENDER ELEMAN TESTİ KULLANILARAK KUM-İNCE DANE KARIŞIMLARININ SIVILAŞMA POTANSİYELLERİ

DEMİR, Süleyman

Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü

Tez Danışmanı: Prof. Dr. Ali Fırat ÇABALAR

Aralık 2017

67 Sayfa

Bu çalışma kum-ince dane karışımlarının en büyük kayma modülü (G_{max}) temelli sıvılaşma potansiyellerini bender eleman deneyi ile değerlendirmeyi amaçlamaktadır. İnce dane olarak kullanılan zemin (<0.075mm) farklı şekil özelliklerine sahip olan (yuvarlak ve köşeli) iki farklı gradasyonlu (0.15-0.30 mm, 1.0-2.0 mm) temiz kuma kuru ağırlık itibariyle %5,%10 %15, %20, %30 ve %40 oranlarında eklenmiştir. Sonuçlar sıvılaşma direncinin %20 ince dane oranına kadar azalırken bu orandan sonra arttığını göstermiştir. İri daneli kumların oluşturduğu karışımların sıvılaşma direnci ince daneli kumların oluşturduğu karışımlardan fazla olmuştur.

Anahtar kelimeler: Sıvılaşma, bender eleman, kum, ince dane.

To my unique parents

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I am very grateful to my parents especially to my mother, my father and my little brother Ahmet DEMİR for their continuous love, support and dedication and for providing me with everything to pursue my studies and career.

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LIST OF SYMBOLS/ ABBREVIATIONS

CSS	Crushed Stone Sand
NS	Narlı Sand
CSR	Cyclic Stress ratio
MDD	Maximum dry density (gr/cm^3)
OMC	Optimum moisture content (%)
G_s	Specific gravity
ASTM	American society for testing and materials
FC	Fines content
PI	Plasticity Index (%)
PL	Plastic Limit (%)
LL	Liquid Limit (%)
OCR	Over consolidation ratio
K_0	Lateral earth pressure at rest
CL	Low plasticity clay
USCS	Unified soil classification system
SPT	Standart penetration test

CHAPTER 1

INTRODUCTION

1.1 General

Liquefaction phenomenon is one of the most important problems of geotechnical engineering. Liquefaction is a geotechnical problem that causes the soil to lose its effective bearing capacity and act as a liquid during earthquake because excess pore water pressure increase under repeated loads due to the water inside soil which are incompressible practically. Table 1.1 shows some earthquakes suffer from liquefaction. The problem of liquefaction aroused great interest after the 1964 Niigata earthquake. In the 1960's, many studies investigated clean sandy soil liquefaction potential (Terzaghi and Peck, 1948; Magomi and Kubo, 1953; Seed and Lee, 1966; Seed and Idriss, 1971; Castro, 1975). Then, it was noticed even some fine soils could liquefiable. The liquefaction potential of the sand-clay mixtures began to be investigated both in the earthquake field area (Seed, 1979; Tokimatsu and Yoshimi, 1983) and in the laboratory (Kramer and Seed, 1988; Ishihara, 1993). However, as a result of the studies, there seems to be inconsistency results about the effect of fine grain on the clean sand liquefaction potential. Some liquefaction criterions also were proposed for clayey sand soils.

1.2 Research Aim

In this study, the fines (CL) was added to two different particle gradations (0.15-0.30 mm, 1.0-2.0 mm) of clean sand having distinct shapes (rounded and angular) with mixture ratio of 5%, 10%, 15%, 20%, 30% and 40%. Bender element tests were performed on the mixtures with the different frequency (10 kHz, 12 kHz, 15 kHz, 17 kHz, 20 kHz) and the small strain shear strength values G_{max}, were used to evaluate the liquefaction potential.

Table 1.1 Some earthquakes suffer from liquefaction

References	Earthquakes	Country	Magnitude	a_{max} (g)	CSR
Ishihara and Koga (1981)	Niigata, 1964	Japan	7.50	0.16	0.170
Ohsaki (1970)	Tokachi-oki, 1968	Japan	8.30	0.23	0.250
Moss et al. (2006)	Tansgan, 1976	China	7.60	0.04	0.360
Moss et al. (2006)	Imperial Valley, 1979	USA	6.53	0.51	0.440
Moss et al (2006)	Victoria, 1980	Mexico	6.30	0.19	0.160
Moss al (2006)	Borah Peak, 1983	USA	6.88	0.5	0.460
Yasuda and Tohno (1988)	Nihonkai-Chubu, 1983	Japan	7.70	0.28	0.310
Zhaou et al. (1997)	Edgecumbe, 1987	New Zeland	6.60	0.44	0.510
Toprak and Holzer (2003)	Loma Prieta, 1989	USA	6.93	0.47	0.391
Moss al (2006)	Hyogoken Nanbu, 1995	Japan	6.90	0.7	0.548
Bray et al. (2004)	Kocaeli, 1999	Turkey	7.50	0.4	0.440
Moss et al. (2006)	Chi-Chi, 1999	Taiwan	7.60	0.6	0.619
Green et al. (2014)	Darfield, 2010	New Zeland	7.10	0.24	0.212
Green et al. (2014)	Christchurch, 2011	New Zeland	6.20	0.46	0.446
Cox et al. (2013)	Tohoku, 2011	Japan	9.00	0.25	0.276

1.3 Outline of the Thesis

This thesis consists of 6 chapters. In Chapter 1, Introduction gives information about the aims and the authenticity of the thesis. Chapter 2 provides information on the studies in the literature on liquefaction. In Chapter 3, the materials used in the experimental study and the methods used are introduced. The test results in Chapter 4 are given together with the interpretation of these test results and their comparison with the results in the literature. Chapter 5 summarizes the results of the thesis study. Chapter 6 also contains suggestions for possible future work.

CHAPTER 2

LITERATURE REVIEW

2.1 Liquefaction Potential of Soils

The liquefaction phenomenon was first used by Hazen (1920) The liquefaction phenomenon was first used by, the researcher describe the Calaveras dam in California behavior as “liquefied”.

Terzaghi and Peck (1948) used the expression "spontaneous liquefaction" for the phenomenon of sudden loss of strength in loose sandy soils.

Mogami and Kubo (1953) stated that dry sand acted like liquid after a point when vibration was applied and used the term "liquefaction" for this phenomenon.

Seed and Lee (1966) conducted several triaxial experiments on saturated sand samples and observed that as the number of cycles increased, the pore water pressure increased. When the pore water pressure is equal to or greater than the applied confining pressure, the sand is designated as "initial liquefaction". Besides, they found that the most important criterion causes of liquefaction are the void ratio, confining pressure and cyclic stress. It was showed that by increasing the void ratio, decreased confining pressure causes more the liquefaction potential.

Seed and Idriss (1971) gave the formula to be mentioned in the equation 2.1 for cyclic stress ratio (CSR) to assess liquefaction potential.

$$CSR = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right)r_d \tag{2.1}$$

where a_{max}= peak horizontal acceleration at ground surface generated by the earthquake, g= acceleration of gravity, σ_{vo} and σ'_{vo} total and effective vertical overburden stresses, respectively, and r_d= stress reduction coefficient dependent on depth.

Seed and Peadock (1971) stated that as the over consolidation ratio (OCR) increases and the Ko value decreases the liquefaction resistance increases.

Ishihara and Li (1972) performed an isotropically consolidation, anisotropically consolidation, and an anisotropic consolidation test with inhibited lateral movement and indicated that the pore water pressure increase was due to the repeated loading and the change of lateral earth pressure at rest (K_0) values. As K_o (0.5, 0.75 and 0.1) values increases, pore water pressure value increased while shear stress value decreases. In addition, anisotropic consolidation samples are more resistant to liquefaction than isotropic consolidation. At a given relative density and confining pressure, values of pore water pressure were most increased in the lateral confined anisotropic test, isotropic test and anisotropic test, respectively.

Castro (1975) conducted a cyclic triaxial test on undisturbed sand samples of different density and stated that there may be liquefaction or cyclic mobility for any sand sample. While the loose sand was completely lost and subjected to liquefaction, there was a reduction in shear strength without failure of the sand samples at medium and dense density. In this behavior the researcher called as "cyclic mobility". The loose sand sample exhibits contractive behavior during the test, whereas medium or dense sand exhibits volumetric expansion. In this expansion, the pore water pressure was slightly scattered and the soil failed to exhibit temporary softening behavior.

Alba et al. (1976) carried out various SPT to enlarge the SPT-CSR graphs and compared them with CSR values by applying various earthquake effects to these soils.

Castro and Poulos (1977) studied the differences between features in liquefaction and cyclic mobility and indicated that a triaxial experiment of a sample of sand starting at point C in Figure 2.1, the researchers noted that the pore water pressure increased and the effective stress decreased to soil come to the point where the phenomenon liquefaction is started was called A. Sample started from the experiment under the steady state line, arrived at point B from point D to the point where the behavior is defined as cyclic mobility expressed as softening behavior. They stated that the steady-state line depends on soil types and to evaluate liquefaction potential steady state line and relative density need to be evaluated together.

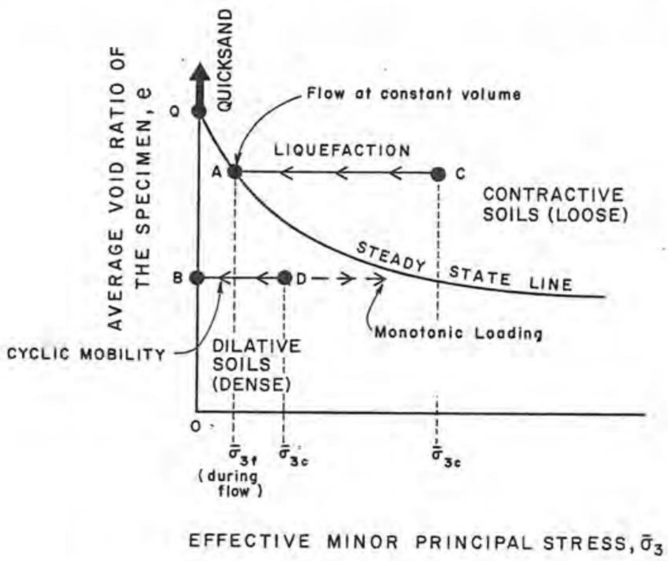


Figure 2.1 Cyclic mobility and liquefaction behavior (Castro and Poulos, 1977)

Seed (1979) conducted triaxial tests on sand samples observed liquefaction and cyclic mobility behaviors in their studies. Experiments on different density of sand samples have shown that the sand samples with a density less than 45% exhibit a contractive behavior and at a rate of 45% and above the sandy exhibit cyclic mobility. In addition, SPT were conducted on undisturbed specimens collected from the liquefied areas of the Earthquake. Finally, the SPT-CSR relationship was pointed out.

Ishihara and Koga (1981) conducted some SPT and dutch cone tests on soils and gathered from two regions of liquefaction and non-liquefaction region of 1964 Niigata earthquake were performed cyclic triaxial test on undisturbed soils. In the SPT, they pointed out that the soil was very loose at certain depths and the SPT values were found to be too low. The triaxial test results were determined as liquefaction zones are clean sands with medium density.

Ishıhara et al. (1981) conducted a study on the measurement of pore water pressure formation in the liquefied soils. In their study, the excess pore water pressure distribution at a certain depth, which contains higher silty soil took more time to distribution pore water pressure than those contain less silty soils.

Youd and Bennent (1983) determined the soil types at liquefied and non-liquefying regions in a magnitude of 6.6 California earthquakes. The SPT and CPT on these soil indicated that the SPT and CPT could be used to evaluate the liquefaction potential. They found that the soil is loose and liquefied in the earthquake where the SPT-N values are low.

Tokimatsu and Yoshimi (1983) examined the liquefaction and non-liquefying soils of many earthquakes and found that the fines content (FC) increased the liquefaction resistance of sand clay mixtures increased at a given SPT-N. If the fines are greater than 20% the soil is hardly liquefied.

Seed et al. (1983) gathered magnitude 7.5 earthquake records in some earthquakes evaluating SPT, CPT and Vs values of the soil and found that if the fines are larger than 20% or if the water content of the clayey soil is <0.9LL considered as non-liquefied.

Seed et al. (1985) determined the SPT-N in the soils with liquefied and non-liquefied region in the earthquake areas. It seems obvious in their work that when the SPT-$N_{1,60}$ are more than 25, soil is hardly liquefied. As the fines increased the CSR increases at a given SPT-$N_{1,60}$.

Tatsuoka et al. (1986) studied the effects of sample preparation techniques on the results performed triaxial and torsional shear tests with air pluviation, wet tamping, wet vibration and water vibration methods. In the results, air pluvation was the most susceptible to liquefaction whereas the wet strength method was the most resistant.

Skempton (1986) studied the relative density, overburden pressure, OCR, particle size and aging effects on liquefaction potential found OCR, relative density, median diameter (D_{50}), overburden pressure (r_v) increased, liquefaction resistance also increased.

Kramer and Seed (1988) observed the effects of relative density confining pressure and initial stress on liquefaction with the two different sands by performing static liquefaction experiments. One of the sand is sieved by No.50 and the other is silty sand and contains about 12% fine grains. Results showed that sand samples liquefied at 32% and 37% relative density values whereas 44% and 47% relative density did not. As the relative density increased, the formation of pore water pressure decreased and the liquefaction resistance increased. The results also showed that the

liquefaction resistance increased with increased confining pressure. Pore water pressure increased as initial stress increased.
Yoshimi et al. (1989) determined that the SPT-N are in good correlation with the liquefaction resistance and that the liquefaction resistance is very close until to a certain SPT-N but the resistance suddenly increased after a critical SPT-N.

Ishihara (1993) investigated many factors that influence liquefaction found the CSR value is constant until 10% plasticity index value of soil, but increased after 10%. It was seen that silty sand samples have more CSR value than clean sand at a given CPT resistance values. In addition, volumetric deformation properties after liquefaction were also investigated and in safety coefficient against liquefaction-volumetric strain graph, the volumetric strain increased as the density increased at the same factor of safety value. According to the borehole from the Niigata earthquake, it was seen that liquefaction occurs in the soil of SPTN values less than 10.

Boulanger et al. (1997) showed that the SPT values has relation with CPT values and shear wave velocity (Vs) values results according to experimental results from soils taken Loma Prieta earthquake areas. In addition, the researchers stated that liquefaction in the earthquake was especially happened in shallow depths.

Sancio et al. (2002) collected soil profile and CPT values from four liquefied regions in Adapazarı, Turkey earthquakes. Because there was liquefaction occurred even in places with high fines content. They stated that fine grain minerology is more important than fine grain ratio.

Olson and Stark (2002) drawn the ratio of undrained shear strength / effective stress, known as the critical shear stress to initiate flow liquefaction in the literature depends on the values of cone resistance (q_c) and SPT-$N_{1,60.}$

Bray et al. (2004) proposed a new liquefaction criterion. According to the authors, soils with plasticity index (IP) <12 and water content (WC)> 0.8LL are susceptible to potential liquefaction.

Bray and Sancio (2006) conducted triaxial experiments on samples from Adapazarı region of Turkey and indicated that non-plastic silts and clayey silts at shallow depths and the soils with WC/LL > 0.85 are liquefied. They evaluated the liquefaction potential of silty soils with a graph.

Boulanger and Idriss (2006) expressed that the behavior of fine grained soils is sometimes acted like clean sand, sometimes only as clean clay or as a combination of two, and graph that distinguish these behaviors was drawn (Figure 2.2). In the case of a sand-clay mixture based on data collected from other studies, the PI> 7 indicates that the mixture is act as clayey soil, whereas if PI <4, it behaves just like sandy soil (Figure 2.2).

Boulonger and Idriss (2007) indicated that normally consolidated clays and silts were susceptible to liquefaction and CRR increased as the over consolidation ratio (OCR) increased.

Bol et al. (2010) expressed that in the case that LL<35, clay content <10%, D_{50}> 0.02 and liquidity index>0.9 in a soil, it is possible that the soil can be liquefied at magnitude 7 or a major earthquake as a result from Adapazarı liquefied soil.

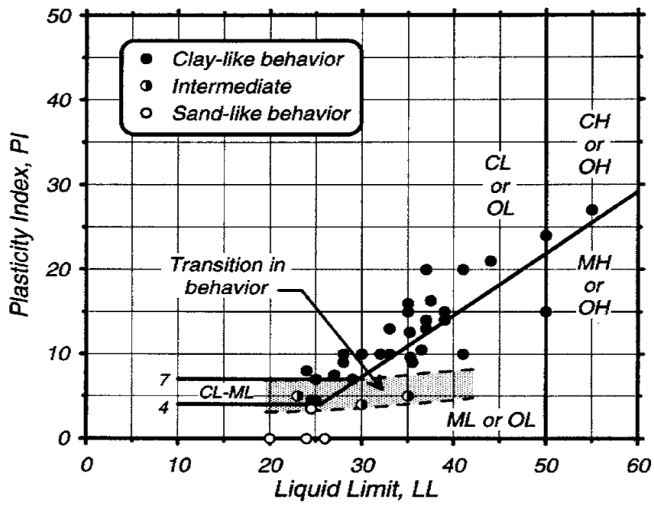


Figure 2.2 Different behavior of fine grained soils (Boulanger and Idriss, 2006)

Monkul et al. (2014) observed that silt additions increased the liquefaction potential as a result of tests on soils prepared at various FC. In addition, they found determined that soil was liquefied when the volumetric compressibility (mV) values exceeded 0.23 as a result from compression test.

Monkul et al. (2015) tested cyclic simple shear in saturated and unsaturated conditions of clean sand, silty sand and clayey soils. The results clearly indicated that the saturation of the mixtures increased the liquefaction potential whereas the silty sand mixtures had little effect of saturation. In addition, they found the intergranular void ratio and fine grain ratio increased, the liquefaction potential also increased.

2.2 Factors Affecting Liquefaction

Tang et al. (2016) studied the main factors that influence liquefaction by examining past studies. The most important factors affecting liquefaction are; The degree of consolidation ratio, the thickness of sand layer, the depth of sand layer, and the groundwater table, earthquake magnitude, epicentral distance, duration and drainage condition and fines content. In this part of the thesis, various factors affecting liquefaction was examined in the light of past studies.

2.2.1 Earthquake parameters

2.1.1.1 Magnitude

Seed and Idriss (1971) stated that the liquefaction occurred at the moment when the ground motion exceeded 0.13g. In laboratory studies, it is clear that as the number of cycles increases, the pore water pressure increased (Seed and Lee, 1966).

Seed et al. (1983) found the pore water pressure caused to liquefaction is small for large earthquakes (Figure 2.3).

2.1.1.2 Epicentral distance

Kuribayashi and Tatsuoka (1975) and Papadopoulos and Lefkopoulos (1993) found that the liquefaction is occurred around the the earthquake center.

Kuribayashi and Tatsuoka (1975) gave the formula LogR = 0.77M-0.36 between the potential liquefaction zones where earthquake magnitude M, which is R km away from the center of the earthquake.

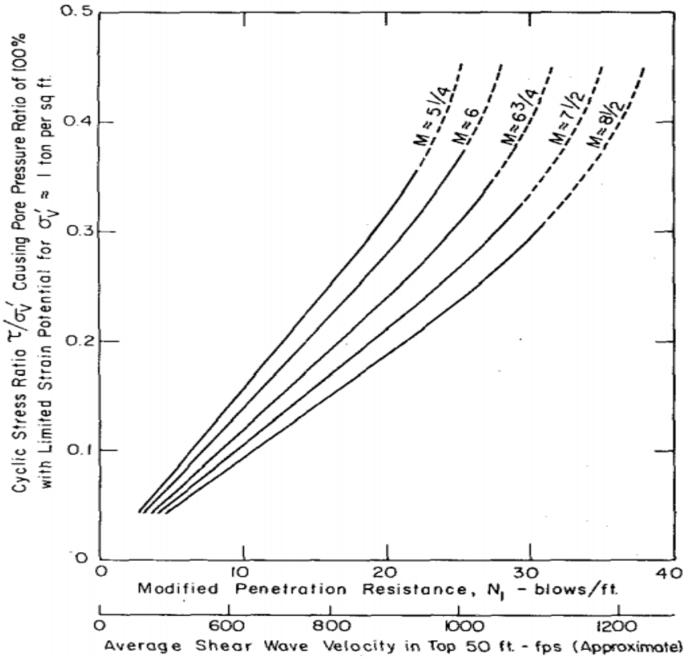


Figure 2.3 Effects of earthquake magnidute on liquefaction (Seed et al., 1983)

2.2.2 Soil parameters

2.2.2.1 Fines and plasticity of fines

Stark and Olson (1985) showed that sand clay mixtures have more resistance to liquefaction than clean sand and clean gravel (Figure 2.4).

Amini and Qui (2000) found that as the silt content increased, the liquefaction resistance increased (Figure 2.5) The approach of such behavior is observed by some researchers (Seed et al., 1985, Seed et al., 1986; Takch et al., 2016).

Some researchers indicated that the FC reduced resistance to liquefaction. Chien et al. (2002) mixed fines with sandy soil and found that CSR decreased with FC (Figure 2.6). Similar results were found by Lade and Yahamuro, (1997); Carraro et al., (2003).

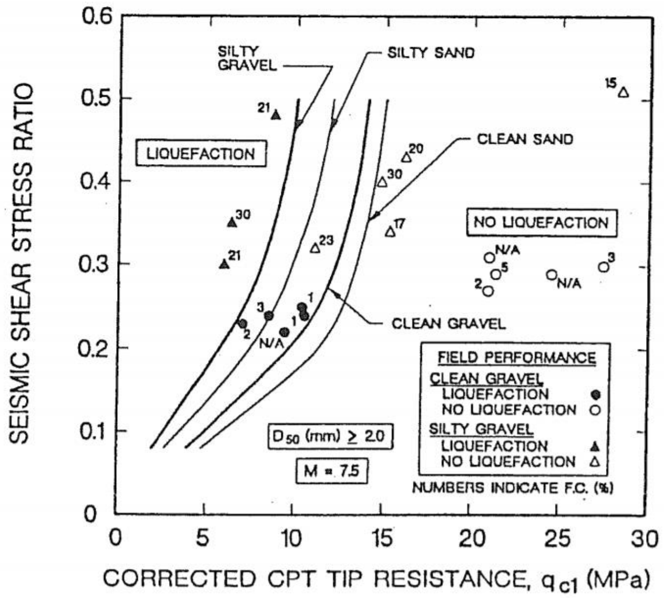


Figure 2.4 Effects of fines content on liquefaction (Stark and Olson, 1995)

In recent years, some researchers found that the FC reduces resistance to liquefaction to a transition fines content (FC_t), but increased after that. FC_t found by Chang et al. (1982) to be 10%, by Thevanayagam et al. (2000) 25%, by Polito and Martin (2000) 35%, by Xenaki and Athanasopoulos (2003) 44%, by Ueng et al. (2004) 20%, by Popodopulo and Tika, (2008) % 35, by Chang and Hong (2008) from 17 to 26%, by Dash and Sitharam (2011) 21%, by Karim and Alam (2014) 30%.

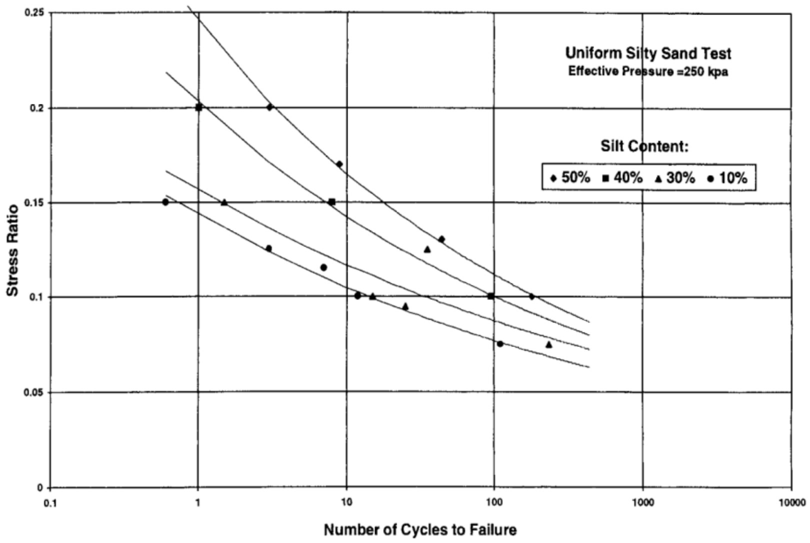


Figure 2.5 Effects of fines content on liquefaction (Aminu and Qui, 2000)

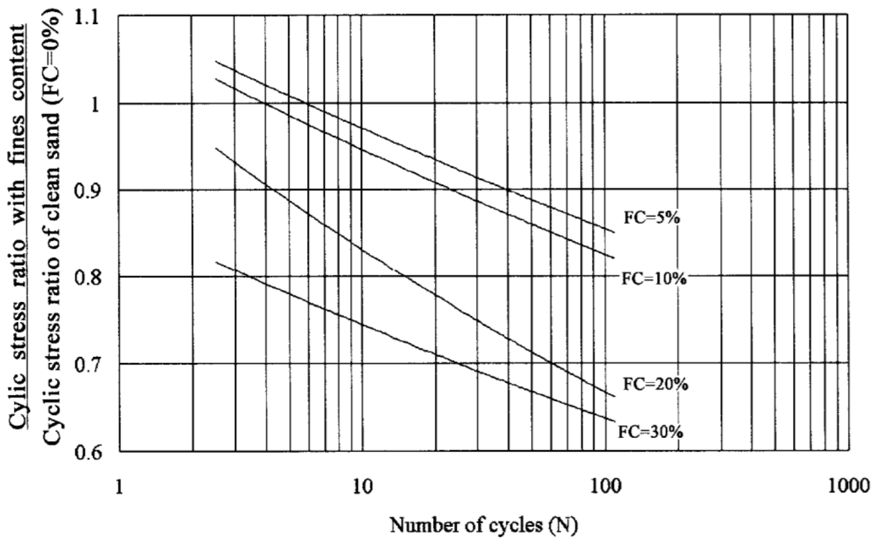


Figure 2.6 Effects of fines content on liquefaction (Chien et al., 2002)

Ishihara (1993) found that the liquefaction resistance increased with the plasticity index values for a range of >10.

Gratchev (2006) were mixed bentonite, illinite and kaolinite as fine grain with sand. In the results, high-plastic bentonite-sand mixture showed the highest resistance to liquefaction in the same fine grain ratio. The importance of the plasticity value of the fine added to the mixtures are noted and evaluated according to the plasticity value as in Figure 2.8.

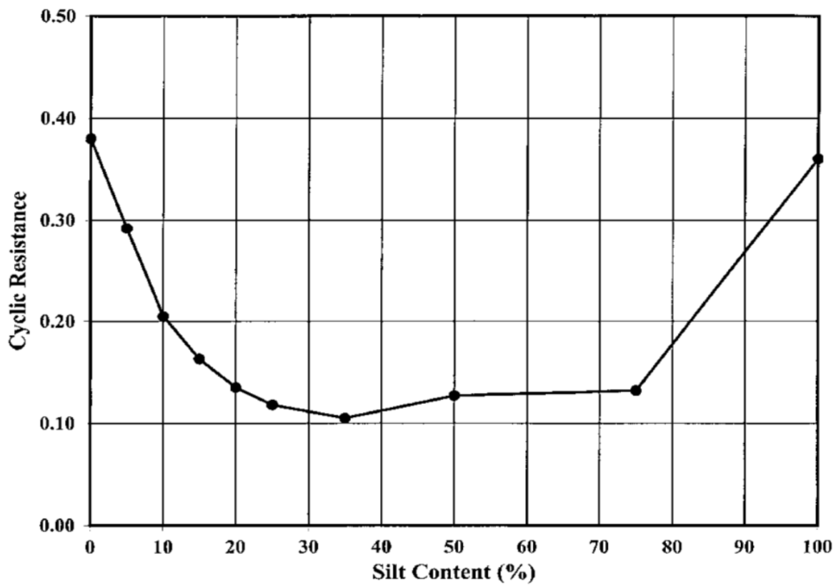


Figure 2.7 Effects of fines content on liquefaction (Polito and Martin, 2001)

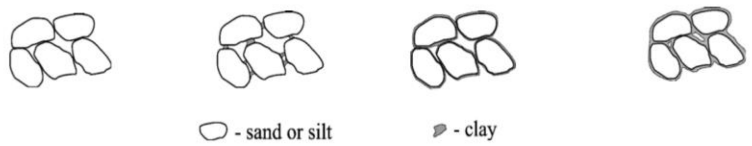
Name	Non-plastic	Low plasticity clayey sand	Medium plasticity clayey sand	High plasticity clayey sand
Plasticity Index, PI	0	≤4	≈5–14	≥15
Type of cyclic behavior	liquefaction	Rapid liquefaction	Liquefaction; liquefaction resistance increased	No liquefaction
Schematic representation of particle arrangement		- sand or silt	- clay	
Special microfabric features	Sand-to-sand contacts	Open microfabric, low strength "clay bridges"	Open microfabric, sand and silt are clothed in clay	Clay matrix
Samples	S7, B3, HT, Dn, T1	B5, B7, K15, 115	B9, T4	B11, B15, T2, T3, Hg

Figure 2.8 Effects of plasticity of fine grains on liquefaction (Gratchev et al., 2006)

2.2.2.2 Particle size

Tsuchida (1970) evaluated the liquefaction potential according to grain size in Figure 2.9.

Tokimatsu and Yoshimi (1983) stated that soils with D_{50} < 0.15 mm showed more resistance to liquefaction than silty sand mixtures D_{50}>0.25 mm.

Stark and Olson (1995) showed that the liquefaction potential of clean sands for the M=7.5 magnitude earthquake as D_{50} value increases, the liquefaction resistance decreased (Figure 2.10).

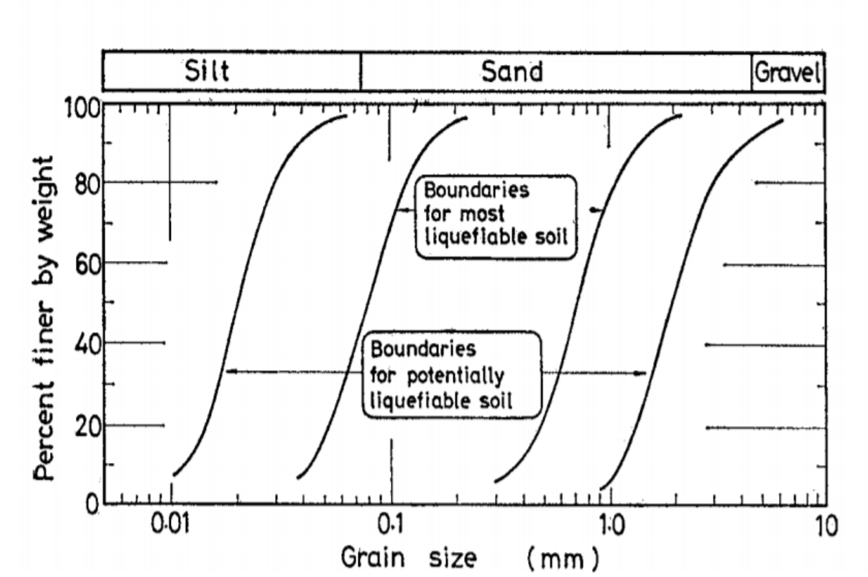


Figure 2.9 Effects of particle size on liquefaction (Tsuchida, 1970)

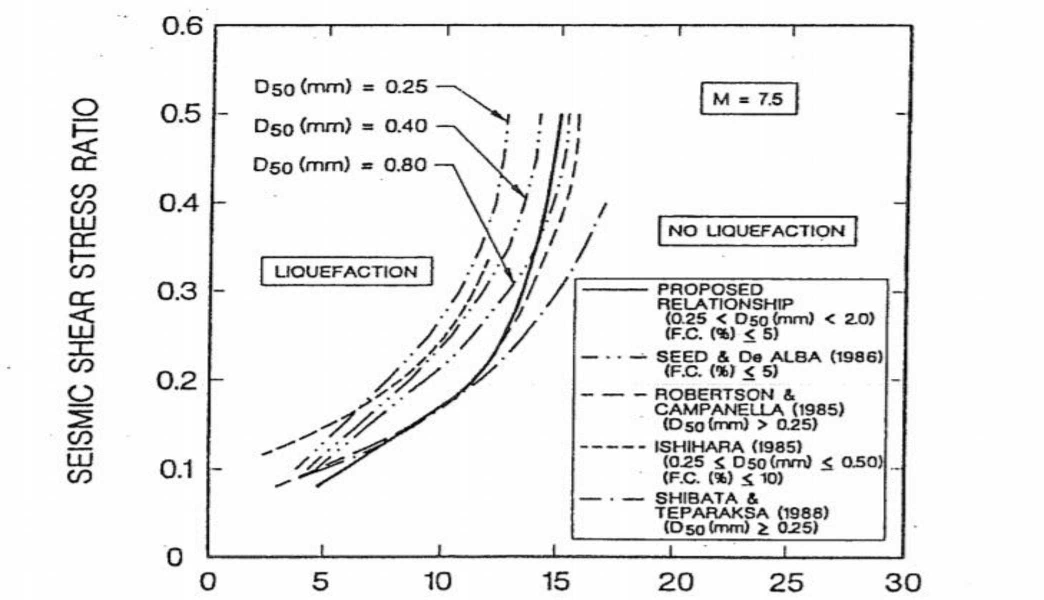


Figure 2.10 Effects of particle size on liquefaction (Stark and Olson, 1995)

2.2.2.3 Particle shape

Vaid et al. (1985) carried out cyclic triaxial experiments on different confining and relative density sands of round ottowa sand and angular tailings found that rounded ottowa sands have less void ratio than the angular tailing at the same confining pressure. However, angular particles showed more liquefaction resistant than rounded particles at low confining pressure values. At high confining pressures rounded particles have more liquefaction resistance than angular particles. They concluded that angular sands susceptible to liquefaction.

2.2.2.4 Relative density

Tatsuoko et al. (1986) observed that as the relative density increased, the CSR values increased in four different sample preparation techniques (Figure 2.11**).** Same conclusion were founded by De alba et al. (1976); Ishihara et al. (1979); Youd and Bennent (1983); Ishihara et al. (1980); Ishihara et al. (1985); Umehera et al. (1985); Vaid et al. (1985); Peadock and Seed (1986); Skempton 1986.

2.2.2.5 Drainage condition

Umehara et al. (1985) stated that under the conditions of undrained and partially drained, the pore water pressure can be dissipated in the drained conditions as a result of the experiments on the clean sands.

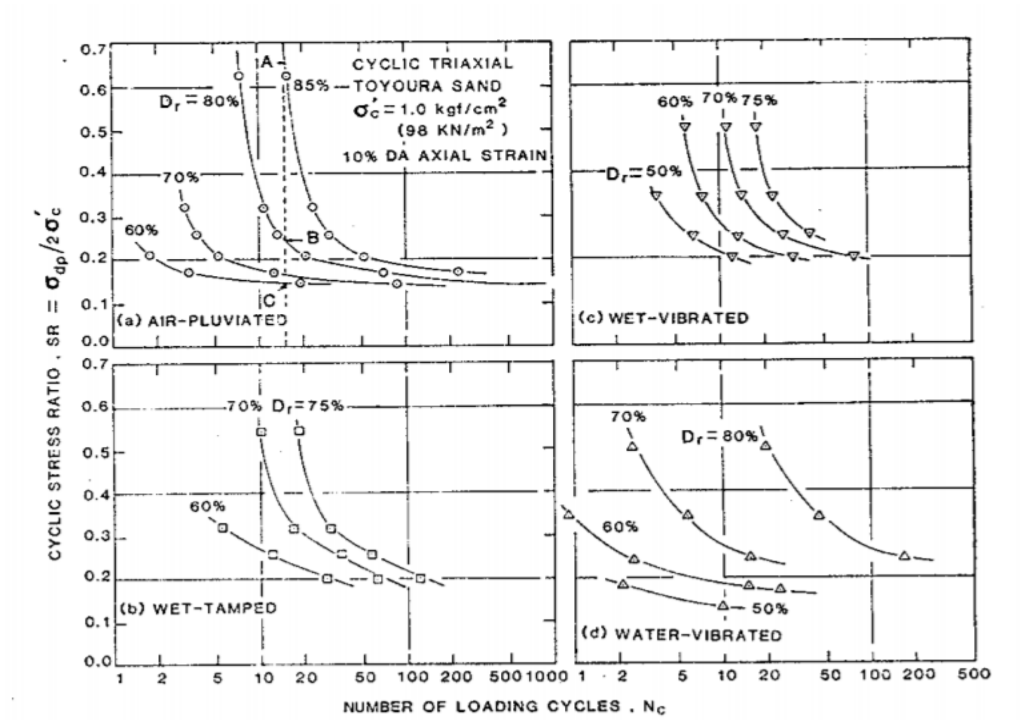


Figure 2.11 Effects of relative density on liquefaction (Tatsuoka, 1986)

2.2.2.6 Degree of consolidation

Ishihara and Takatsu (1979) studied both factors of K_0 and OCR on liquefaction found CSR as OCR increases at a given K_0 values (Figure 2.12). Similar results have been obtained by Finn (1981); Ishıhara et al., (1978); Salgado et al., (1997); Koseki and Ohta, (2001).

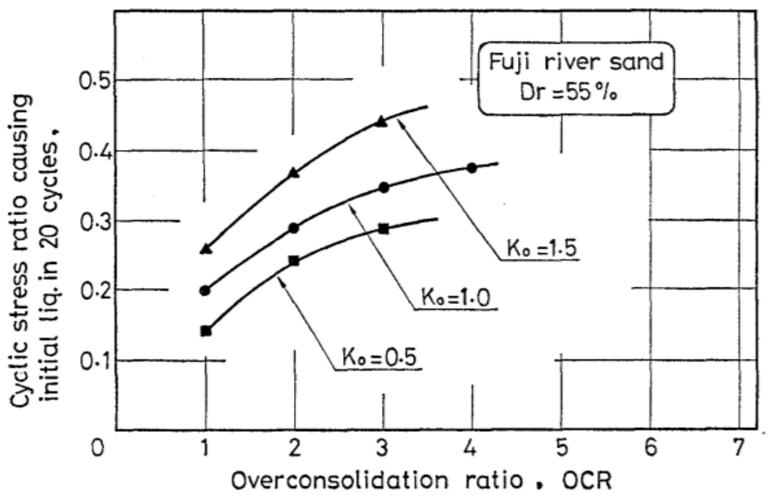


Figure 2.12 Effects of OCR on liquefaction (Ishihara and Takatsu, 1979)

2.2.2.7 Sample distrubance

Yoshimi et al. (1989) found the high undisturbed soil in situ freezing method showed more liquefaction resistance than tube sample samples (Figure 2.13) Similarly, undisturbed soils have observed more liquefaction resistance or shear force than disturbed soils found by researches such as Tokimatsu and Hosaka, (1986); Kiyota et al.,2009.

2.2.2.8 Sample preparation methods

Tatsuoka et al. (1986) performed triaxial and torsional shear tests on sand samples prepared with air pluviation, wet tamping, wet vibration and water vibration methods. In the results, air pluviation was the most susceptible to liquefaction, while wet vibration method was the most resistant (Figure 2.14).

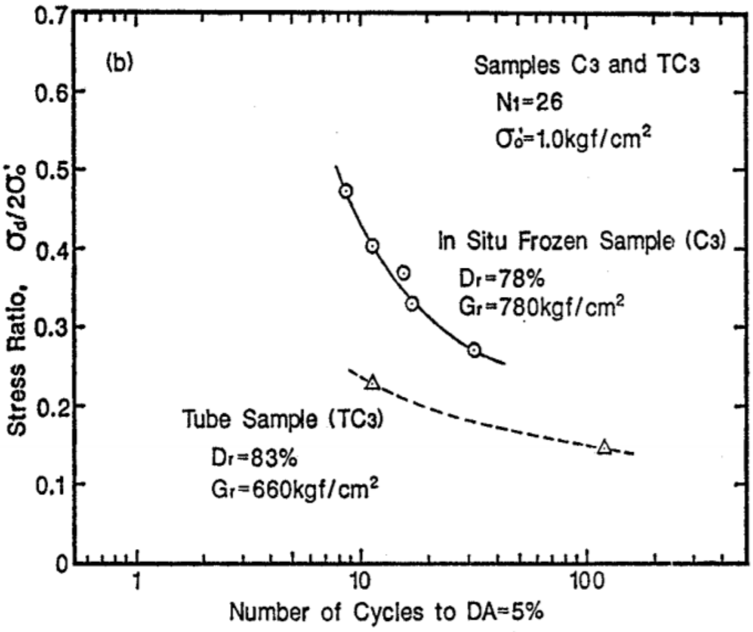


Figure 2.13 Effects of disturbance on liquefaction (Yoshimi et al., 1989)

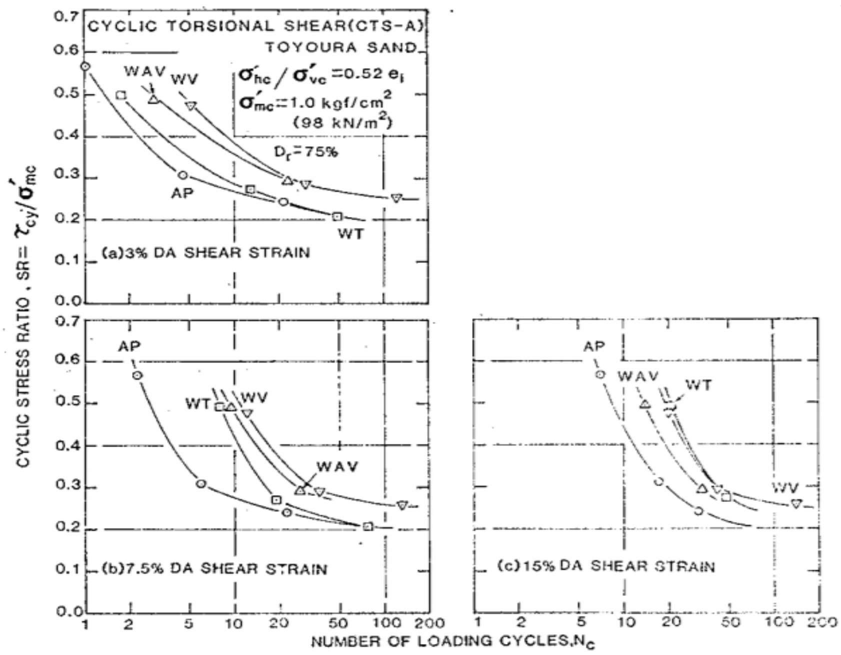


Figure 2.14 Effects of sample preparation methods on liquefaction (Tatsuoka et al., 1986)

2.2.2.9 Confining pressure

Bray et al. (2002) expressed that with the high the confining pressure, susceptible to liquefaction is increased.

Vaid et al. (1985) showed that as the initial confining pressure (r_v) increased, the liquefaction potential may increase or decrease, but in general the liquefaction potential increased. Similar result was found by Amini and Qui (2000).

2.2.3 Site conditions

2.2.3.1 Groundwater

As the groundwater level increases, the soil becomes weaker and liquefaction potential increases (Hannich et al., 2006).

2.3 Liquefaction Criterion of Sand-Clay Mixtures

For the sand clay-silt mixtures in the literature, charts drawn by various factors have been drawn and the liquefaction potential has been assessed depending on some factors.

Tokimatsu and Yoshimu (1983) collected a lot of data stated that liquefaction occurred in <5% FC situations. A triangular liquefaction potential was suggested for liquefaction (Figure 2.15).

Bray et al. (2002) found that even the so-called non-liquefied soils on Chinese criterion were liquefied in the Adapazarı earthquake and that Adapazarı had small confining stress levels according to liquefaction conditions as shown Figure 2.16.

Andrews and Martin (2000) reported that the liquefaction potential for clayey soils in their studies is as shown in Table 2.1.

Bol et al. (2010) advise liquefaction criterion for silty soils in Adapazarı earthquake (Figure 2.17).

Seed et al. (2003) proposed Figure 2.19 where PI <12 and LL <37 as the potential liquefaction interval (Figure 2.18).

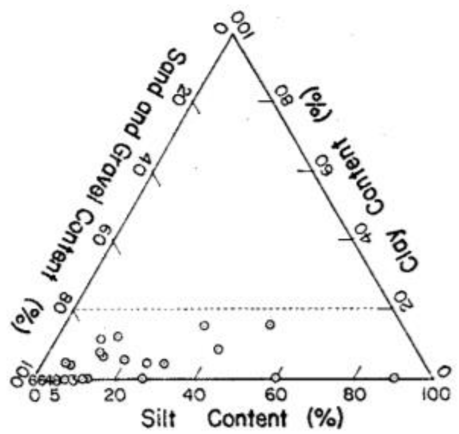


Figure 2.15 Liquefaction criterion of sand-clay mixtures (Tokimatsu and Yoshimi, 1983)

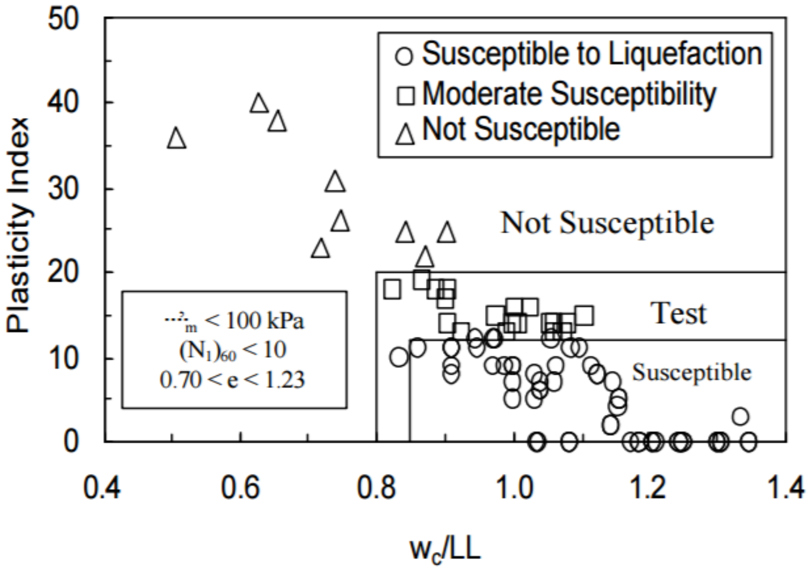


Figure 2.16 Liquefaction criterion of sand-clay mixtures (Bray et al., 2002)

Table 2.1 Liquefaction criterion of sand-clay mixtures (Andrews and Martin, 2000)

	Liquid Limit < 32 (1)	Liquid Limit ≥ 32
Clay Content < 10% (2)	Susceptible	Further Studies Required *(Considering plastic non-clay sized grains - such as Mica)*
Clay Content ≥ 10%	Further Studies Required *(Considering non-plastic clay sized grains – such as mine and quarry tailings)*	Not Susceptible

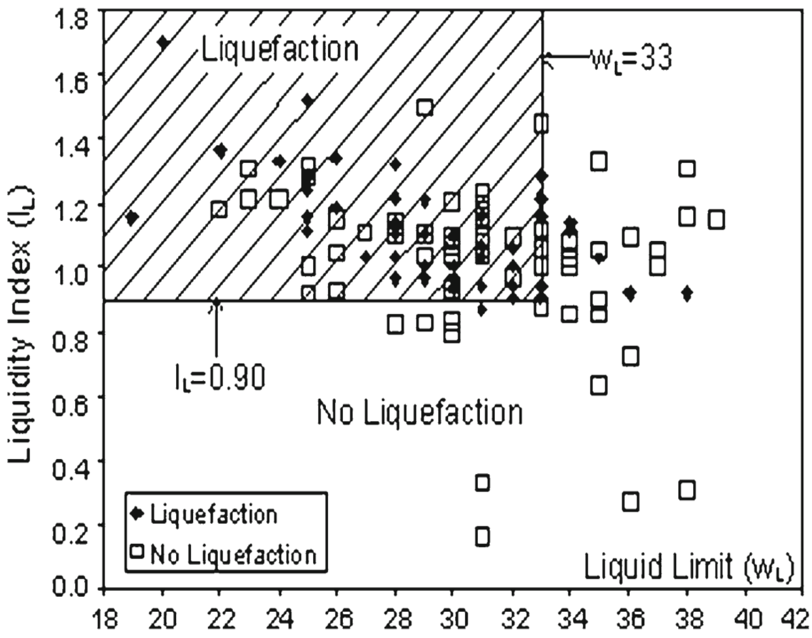


Figure 2.17 Liquefaction criterion of sand-clay mixtures (Bol et al., 2010)

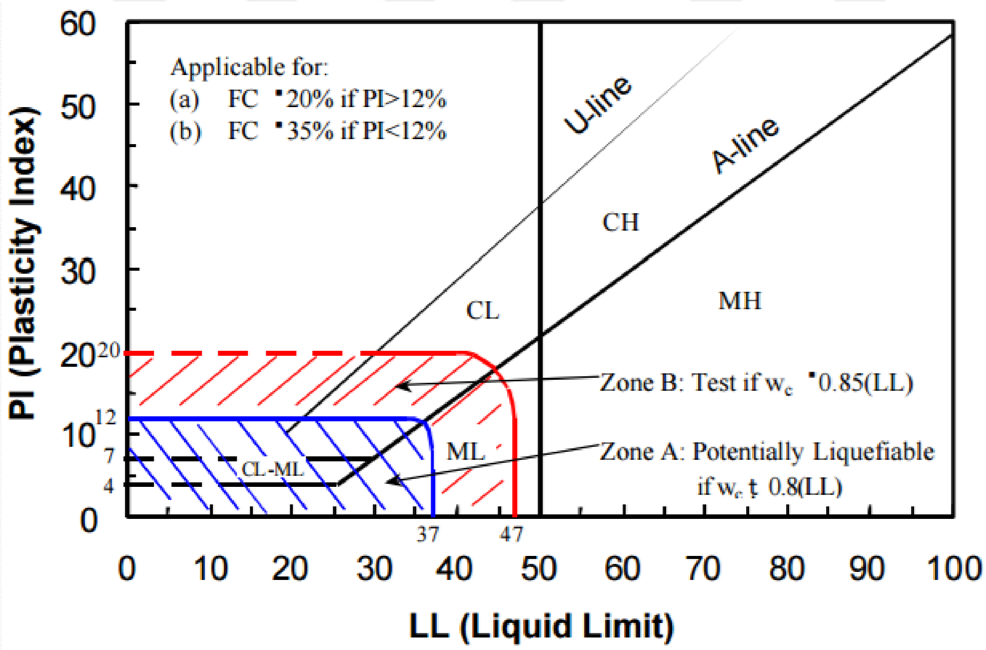


Figure 2.18 Liquefaction criterion of sand-clay mixtures (Seed et al., 2003)

CHAPTER 3

EXPERIMENTAL STUDY

3.1 Materials

3.1.1 Fines

The fines obtained from the Gaziantep University campus sieved from a No.200 (0.075 mm) and found to be low plasticity clay (CL) according to unified soil classification system (USCS). Some geotechnical specification of fines is shown in Table 3.1. The scanning electron microscope (SEM) picture of the fines is shown in Figure 3.1. The maximum dry density (MDD) and the optimum moisture content (OMC) values of fines were found to be 1.67 gr/cm^3 and 19.30%, respectively.

Table 3.1 Geotechnical specification of fines used during experimental study

G_s	ρ_{dmax} (gr/cm^3)	W_{opt} (%)	LL (%)	PL (%)	PI (%)	Activity	Swelling percentage (%)	Permeability (cm/sn)
2.61	1.67	19.30	39	24.37	14.6	0.25	1.5	1.04 *10^{-6}

G_s: specific gravity; ρ_{dmax}: Maximum dry density; W_{opt}: Optimum moisture content; LL: Liquid limit; PL: Plastic limit; PI: Plastisity index

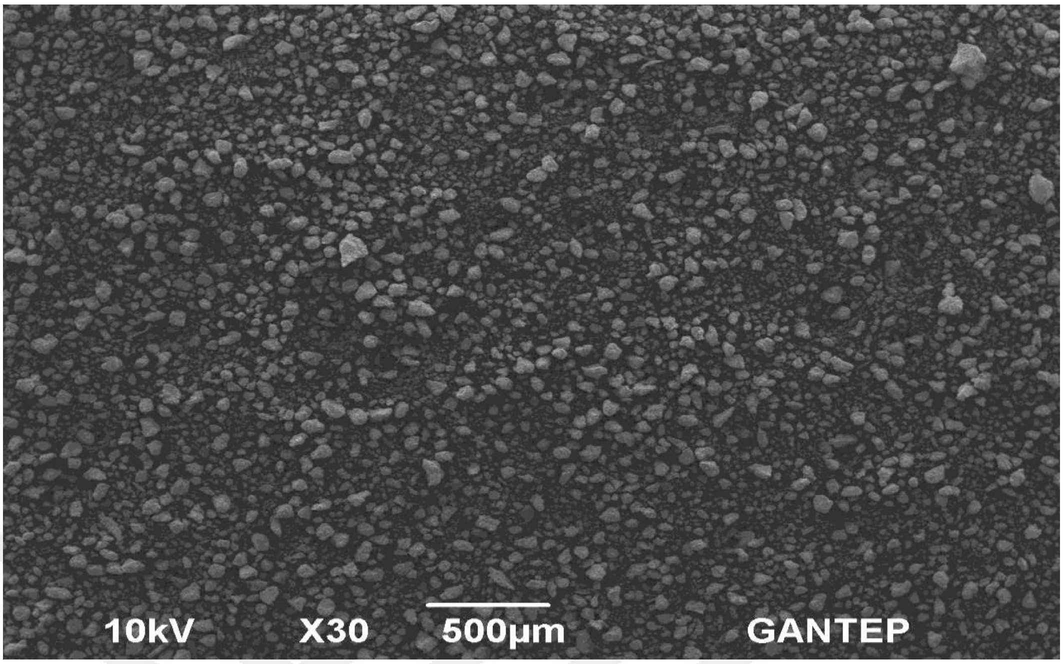


Figure 3.1 SEM picture of fines used during experimental study

Figure 3.2 Fines used during experimental study

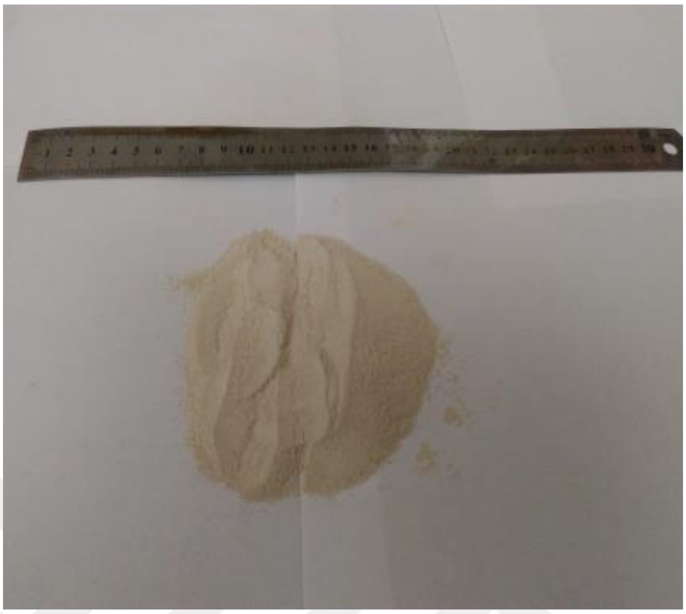
3.1.2 Sands

Two sands (Figure 3.3) were used in the present study, narlı sand (NS) and crushed stone sand (CSS). NS was collected from the Aksu River bank in Narli region in Kahramanmaras, Turkey. The Aksu River starts in northwest of Kahramanmaras City, which lies in southern Turkey (37°36′N; 36°55′E) and bounded by hills or mountains on all sides. A commercially available CSS was obtained from southern-central of Turkey, which is widely used in earthworks in Gaziantep City. The specific gravity of NS and CSS found to be 2.65 and 2.68 respectively. As can be seen from the SEM pictures in Figure 3.4 NS grains are rounded, while CSS are angular. Two different gradations (0.15-0.30 mm and 1.0-2.0 mm) of the NS and CSS mixed with fines. Sieve analysis and some geotechnical properties of sands are given in Figure 3.5 and Table 3.2, respectively. Cabalar and Akbulut (2016) found roundness (R) and sphericity (S) estimations based on the study by Muszynski and Vitton (2012) 0.43, 0.67, and 0.16, 0.55 for the NS and CSS geomaterials, respectively.

a)

b)

c)



d)

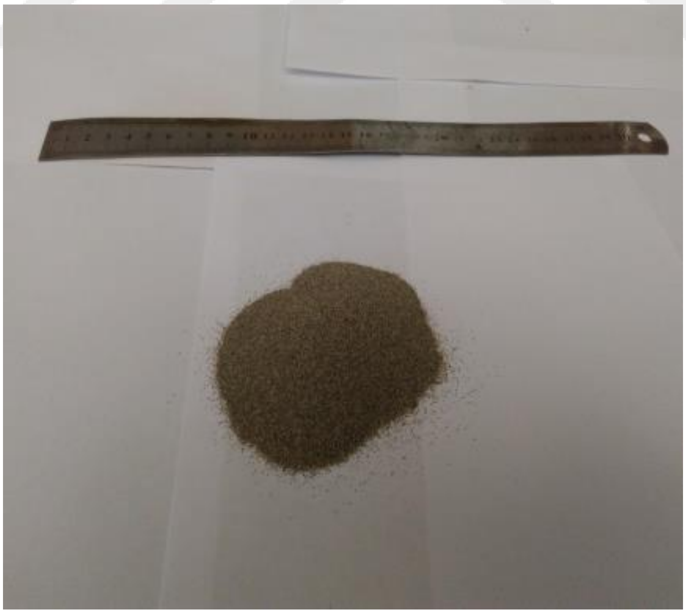


Figure 3.3 Sands used during experimental study a.CSS (1.0-2.0 mm) b.NS (1.0-2.0 mm) c.CSS (0.15-0.30 mm) d.NS (0.15-0.30)

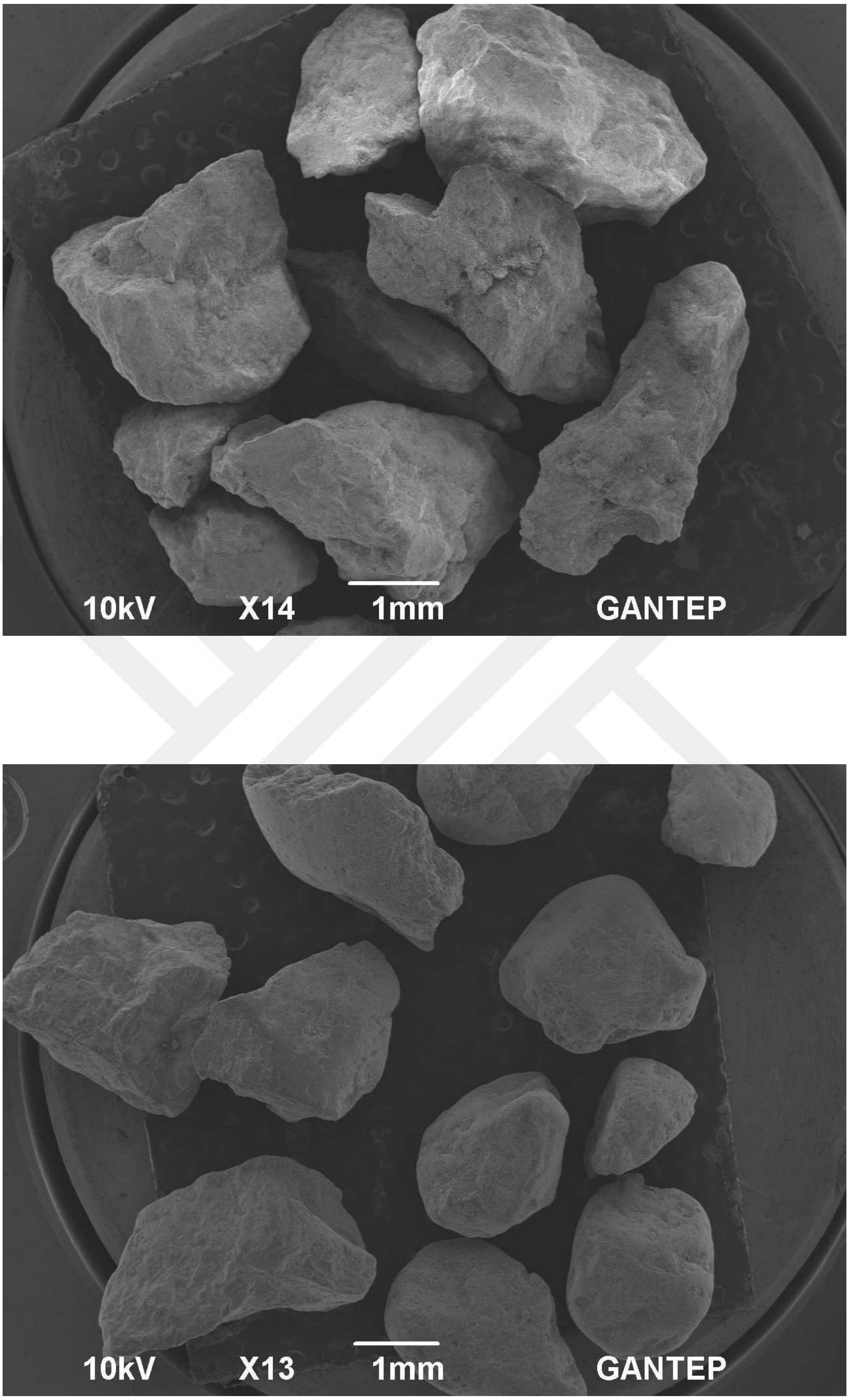


Figure 3.4 SEM pictures of the (top) CSS and (bottom) NS used during the experimental study

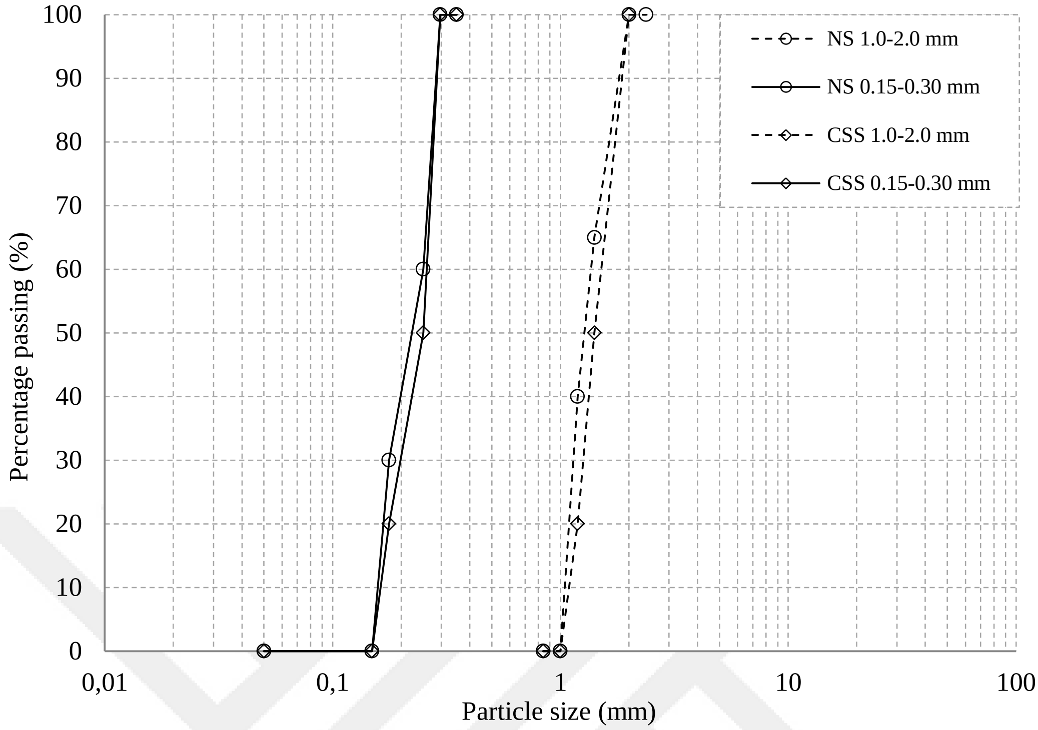


Figure 3.5 Particle size distribution of clean sands used during experimental study

Table 3.2 Properties of sands used the experimental study

Sands	Gradation (mm)	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	C_u	C_c	USCS
NS	0.15-0.30	0.16	0.18	0.22	0.25	1.388	0.81	SP
NS	1.0-2.0	1.10	1.30	1.40	1.50	1.153	1.02	SP
CSS	0.15-0.30	0.17	0.20	0.25	0.26	1.300	0.90	SP
CSS	1.0-2.0	1.20	1.40	1.50	1.60	1.142	1.09	SP

NS: Narli sand; CSS: Crushed stone sand; SP: Poorly graded sand; USCS: Unified Soil Classification System.

3.2 Testing and Characterization

3.2.1 Specimen preparation

According to the specifications of each test, the materials were sieved, and the size of particles was limited because each test has a specific range of particle size. All materials were put in the oven before using them in the experiments because the materials must be dry before using it in the experiments. The samples were prepared by adding the fines (<0.075mm) to two different particle gradations (0.15-0.30 mm, 1.0-2.0 mm) of clean sand having distinct shapes (rounded and angular) with mixture ratio of 5%, 10%, 15%, 20%, 30% and 40%

3.2.2 Modified compaction test

ASTM D1557 was used to find out the maximum dry unit weight and optimum water content. The special molds with a diameter of 4.2 cm, and a height of 10.2 cm (Figure 3.6) designed. The specimens were compressed in the mold in 5 layers by sticking 31 blows for each layer with an effort equivalent to modified proctor compaction one ASTM D1557 utilizing special designed hammer defined by Cabalar et al. (2014). The experiment was repeated with different water content. The point of peak gave the MDD and OMC.

Figure 3.6 The plastic split mold and the compaction hammer for the modified compaction test

3.2.3 Bender element

Soil stiffness is generally measured using triaxial test. Because of the fact that some problem G_{max} measurement in small strain (Jardine et al., 1984) to obtain more accurate results local strain testing device are use on the specimen. Dynamics and wave propagation was obtain small shear strain values more accurately. Resonant column test (Drenevich et al., 1978) and bender element test (Viggiani and Atkinson, 1995) were used to measure G_{max}. Bender element technique was proposed by Shirley and Hampton (1978) and used in tests such as triaxial test, odometer test(Salgado et al.,2000; Ghayoomi and McCartney, 2011). In this study, bender element was performed to the mixtures. A bender element test detailed set up is gave in Figure 3.7. Since the sample placed between the two ends of the bender element cannot hold itself because of cohesionless, a new mold design was design to hold soil during experimental study. The detailed design of mold (a) Picture of mold b) detailed three dimensional frontal view of mold c) two dimensional frontal view of mold d) plan view of mold e) detailed 1-1 section of mold) shown in Figure 3.8. It was formed from a plastic and a metal surrounding the plastic. Since the diameter of the signal element of the bender element is 7 cm, the diameter of plastic part covering the soil was design as 7 cm diameter. Because of the low electrical conductivity of plastic, plastic was used in the area of diameter of bender element. The inside diameter of the metal mold surrounding the plastic is designed to be equal to the outer diameter of the plastic. The reason why the metal mold is used is to prevent from deform the plastic outwardly when the vertical stress is applied from top of soil. The soil was put into the mold using california bearing ratio (CBR) machine. During the compression with CBR, a collar is placed on the top of the mod and the soil was put into 3 layers. When the bender element test was carried out, the triaxial load ram was used to apply only a vertical load which was 2.3 kn at each experiment. Considering that a lateral load was applied to the soil due to the vertical load on the soil, it can be assumed that vertical stress and confining stress were applied like the studies in the literature. The G_{max} values were calculated through the equation commonly used in the literature according to Equation 3.1.

$$G_{max} = \rho\, Vs^2 = \rho\, \left(\frac{L}{\Delta t}\right)^2 \qquad (3.1)$$

where ρ is the density of soil, L is defined as the distance between tips of bender elements. Δt is the time domain method and frequency domain method for treveal time in the literature (Viggiani and Atkinson, 1995). In this study, peak to peak (Figure 3.9) which the intervals between characteristic peak points of the input and output waves is considered to the travel time of the shear wave (Viggiani and Atkinson, 1995; Arulnathan et al., 1998).

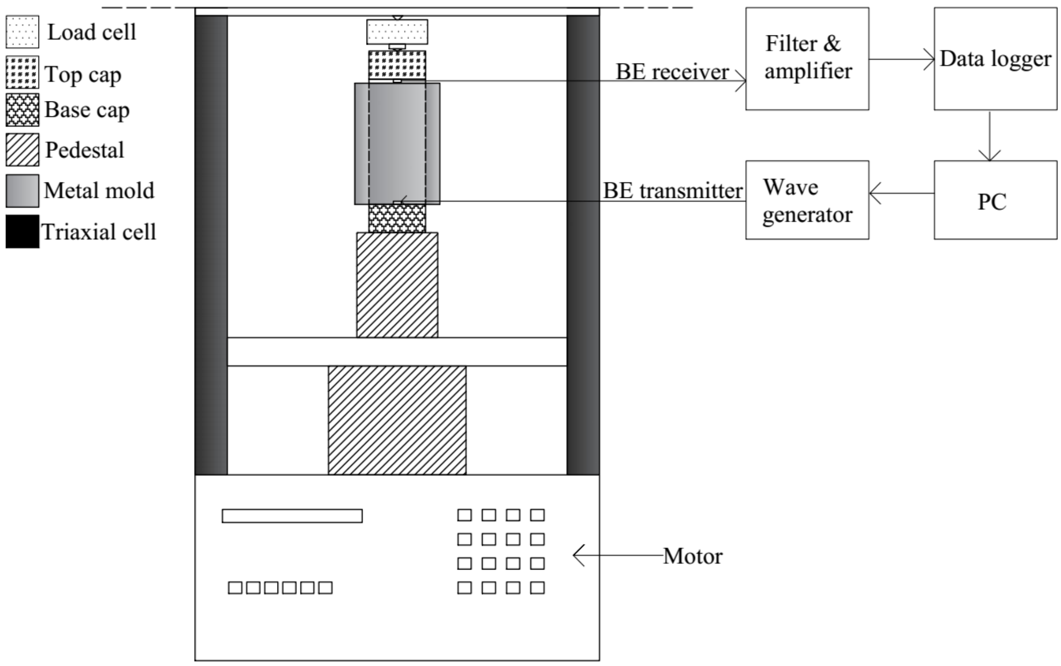
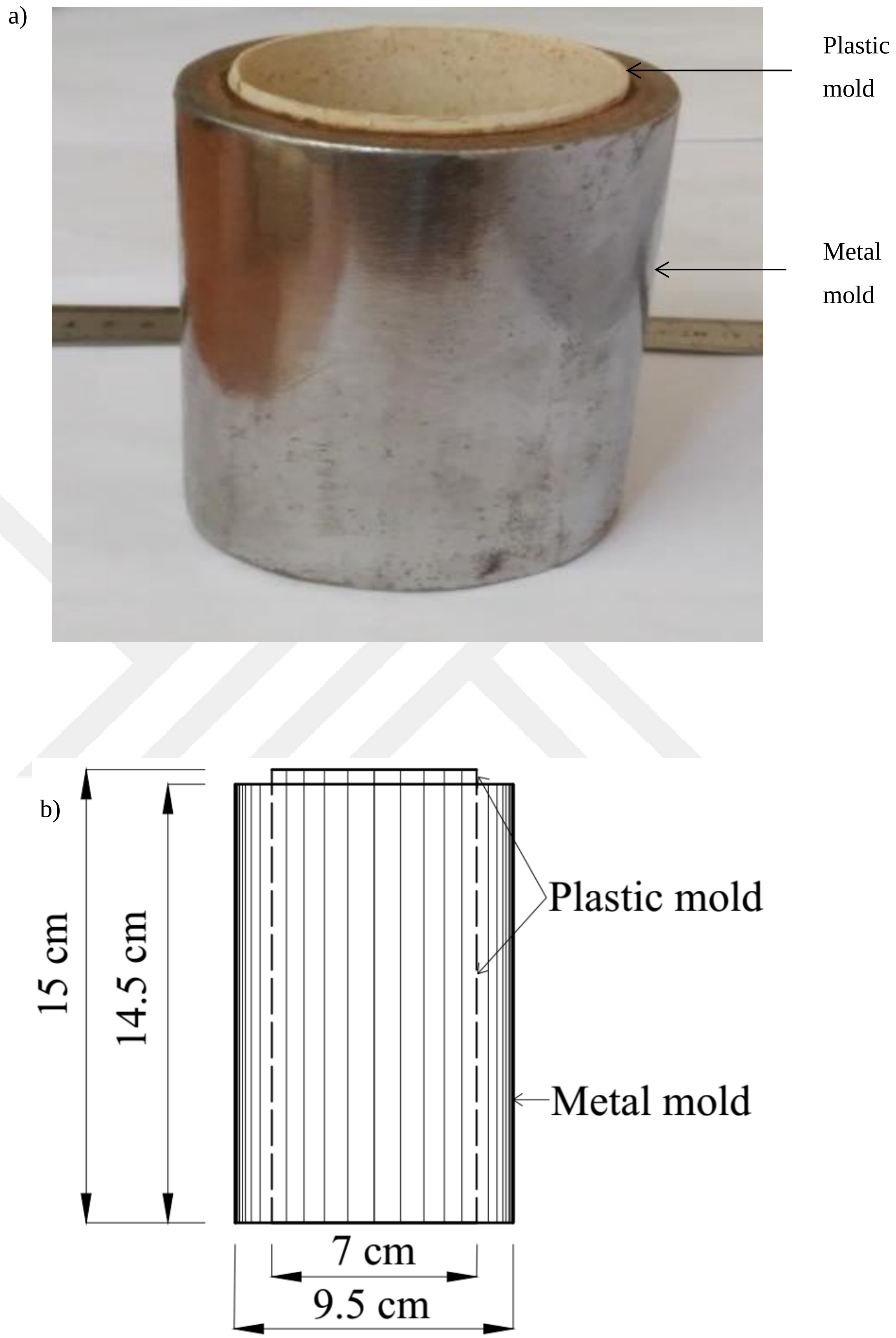
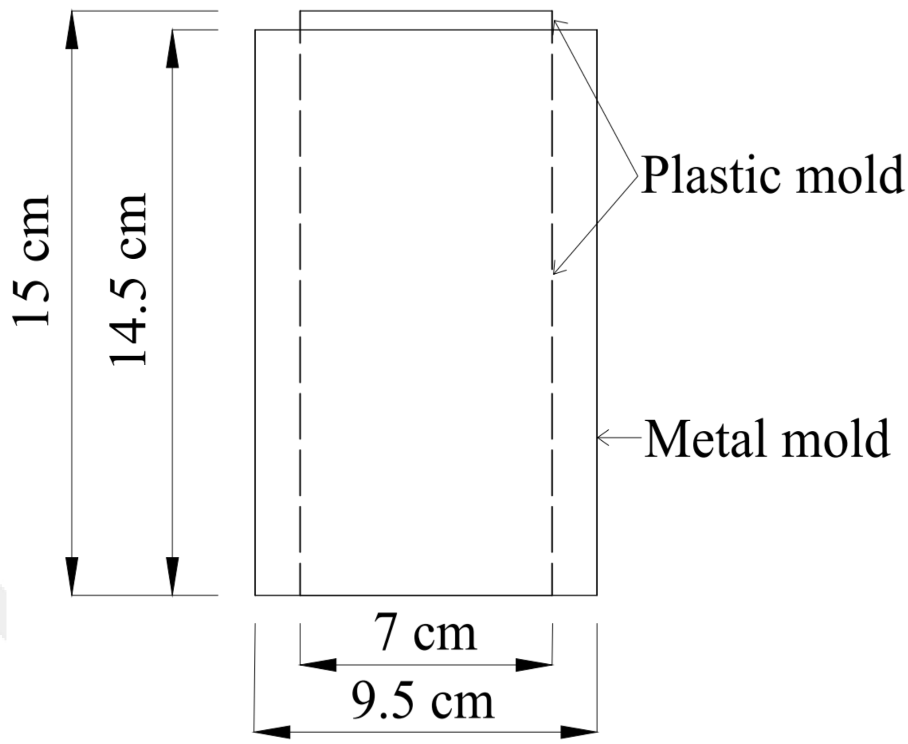


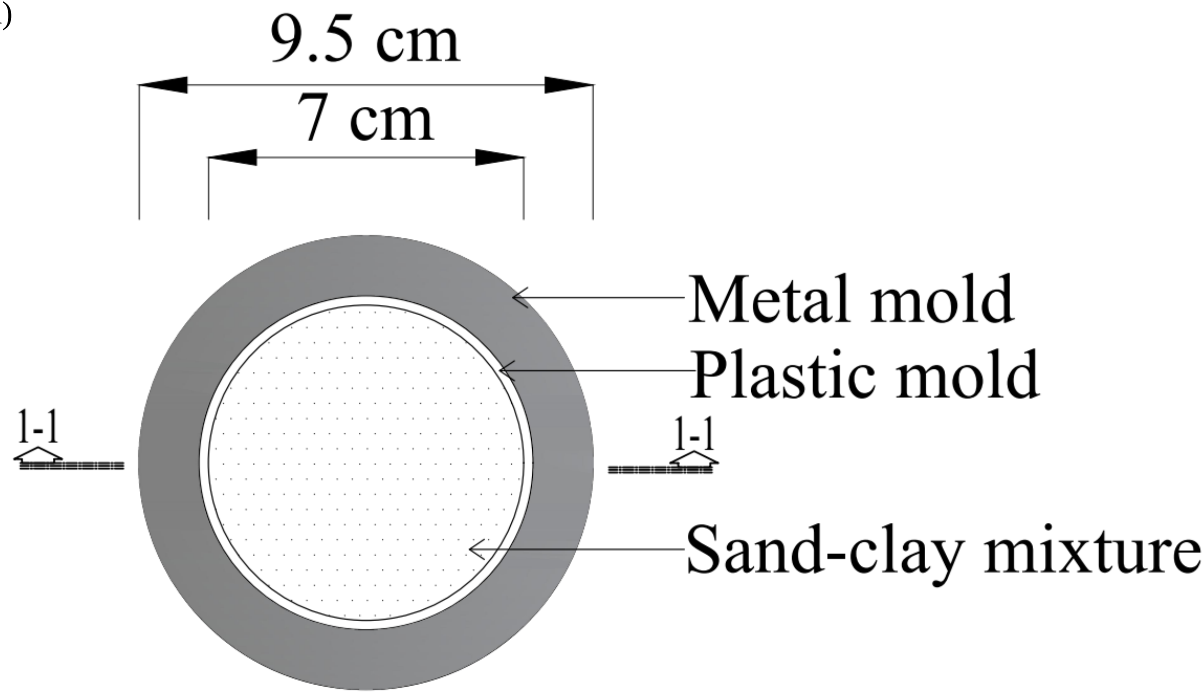
Figure 3.7 Detailed drawing of bender element


a)
Plastic mold
Metal mold
b)
15 cm
14.5 cm
Plastic mold
Metal mold
7 cm
9.5 cm

c)


15 cm
14.5 cm
Plastic mold
Metal mold
7 cm
9.5 cm

d)


9.5 cm
7 cm
Metal mold
Plastic mold
1-1
1-1
Sand-clay mixture

e)

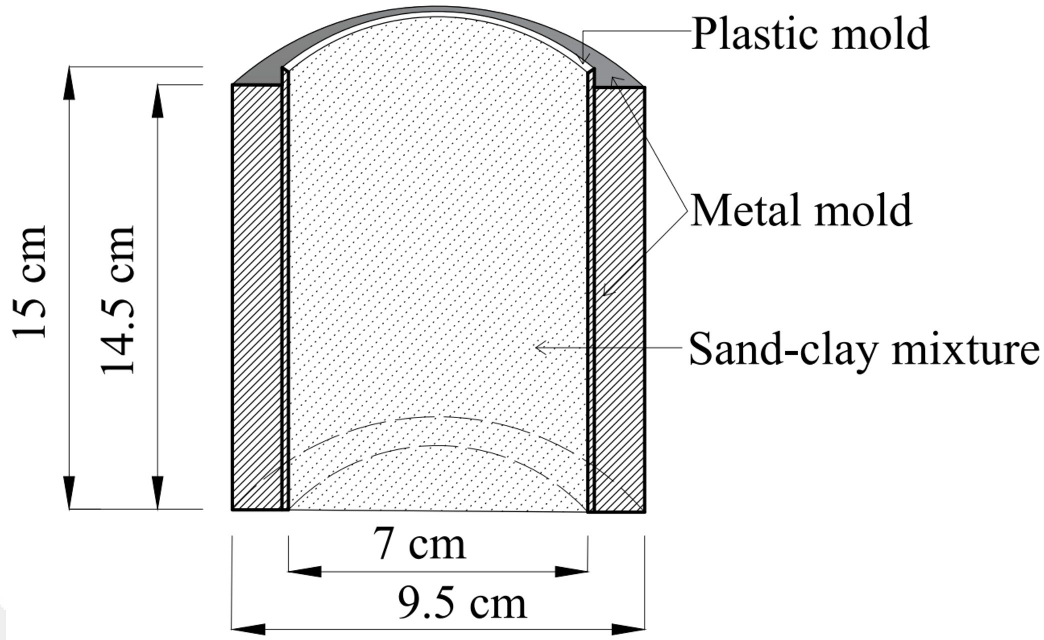


Figure 3.8 Detailed design drawing of mold used during experimental study a) picture of mold b) detailed three dimensional frontal view of mold c) two dimensional frontal view of mold d) plan view of mold e) detailed 1-1 section of mold

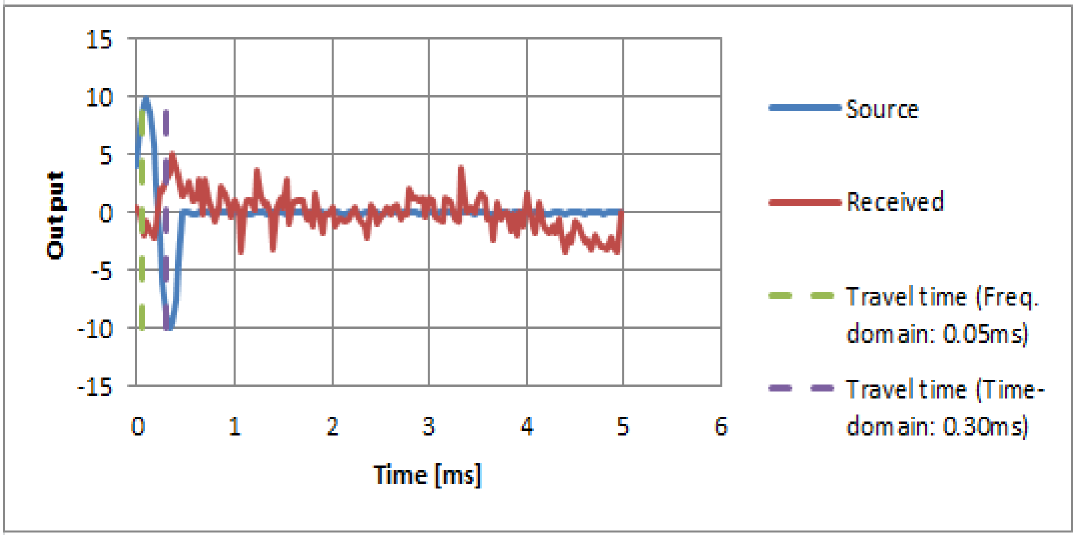


Figure 3.9 Treval time (Peak to peak)

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 Compaction Test

4.1.1 Effect of fines content

In the present study compaction test was first conducted to mixtures because of the fact that the Bender element experiments were carried out at MDD and OMC values. Compaction values of mixtures showed from Figure 4.1 to Figure 4.4. As the FC increased, OMC decreased and MDD increased. The author attributed this behavior to the fact that the fines fill the voids into the loose and poorly graded clean sand. Similar results were observed in studies conducted by Kenney et al. (1992); Komine (2004). In these studies, the presence of fines increased the MDD of the sand fines mixtures up to a certain ratio and then reduced the OMC, increased OMC. This rate was found to be 20% by Kenny et al. (1992) and 30% by Komine (2004). Such a rate was not seen in this study. Because, researchers used high plasticity bentonite as fines and the clean sand had well gradated. In this study, a low plastic fines ware used and the clean sand was considered to have a high void ratio due to its poorly graded. For this reason, the addition of 40% FC was not completely filling the voids of the clean sand. Compaction results of mixtures can be seen in Table 4.1.

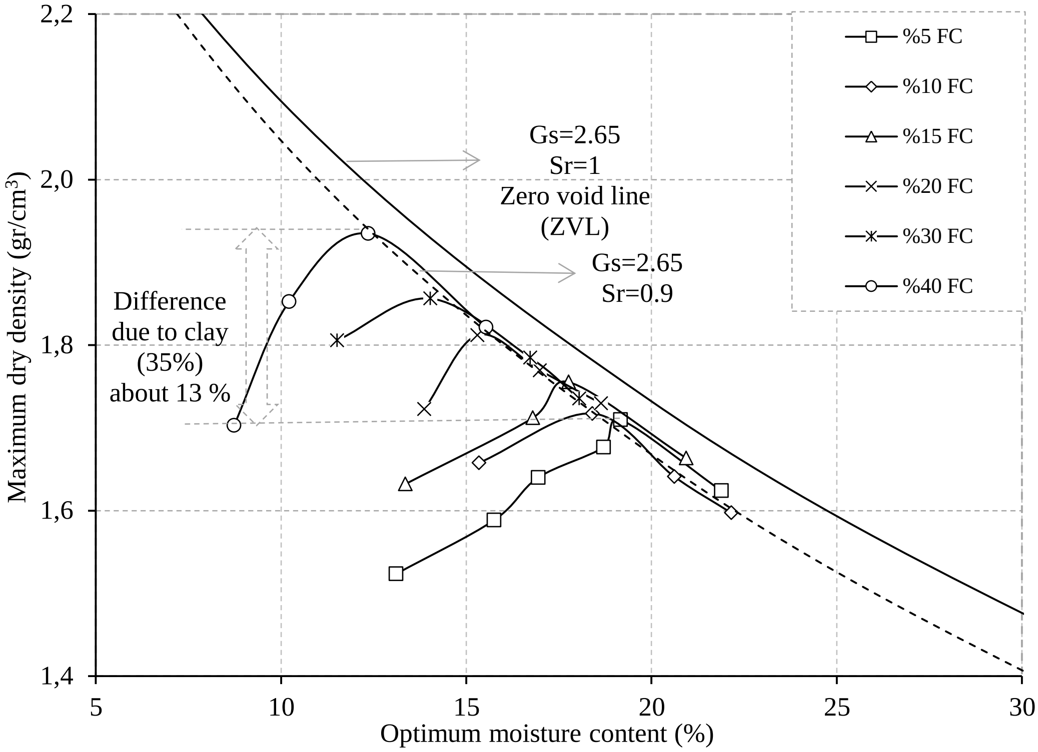


Figure 4.1 Variation of OMC with MDD for the clay with 0.15-0.30 mm NS

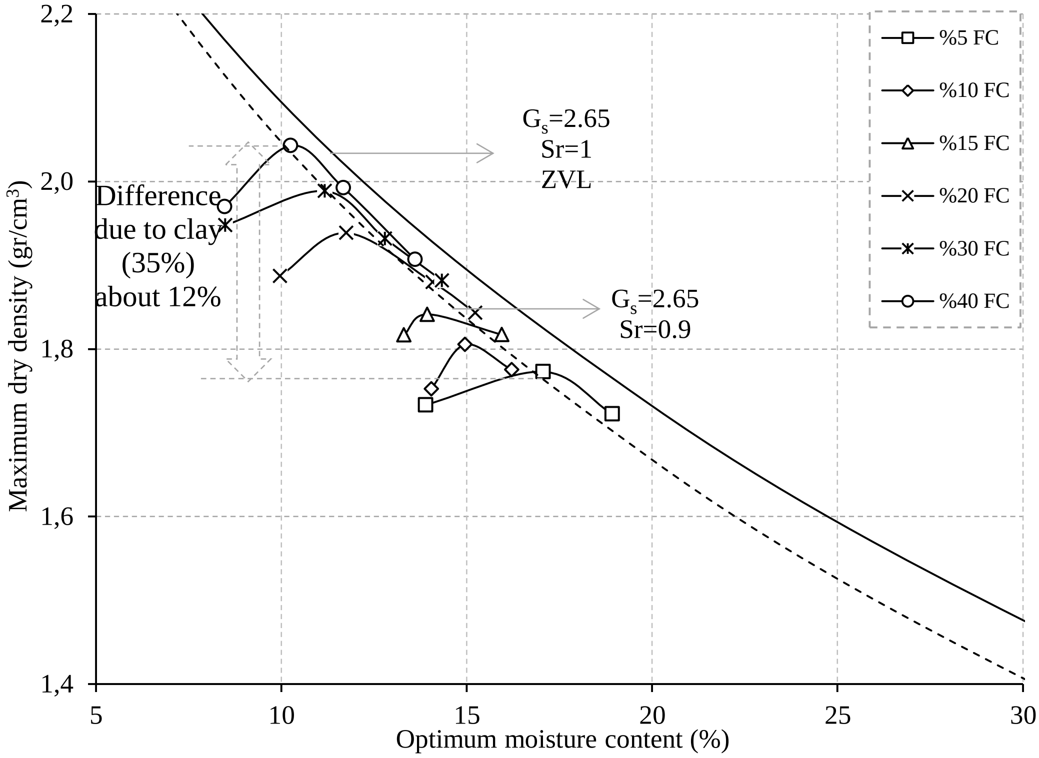


Figure 4.2 Variation of OMC with MDD for the clay with 1.0-2.0 mm NS

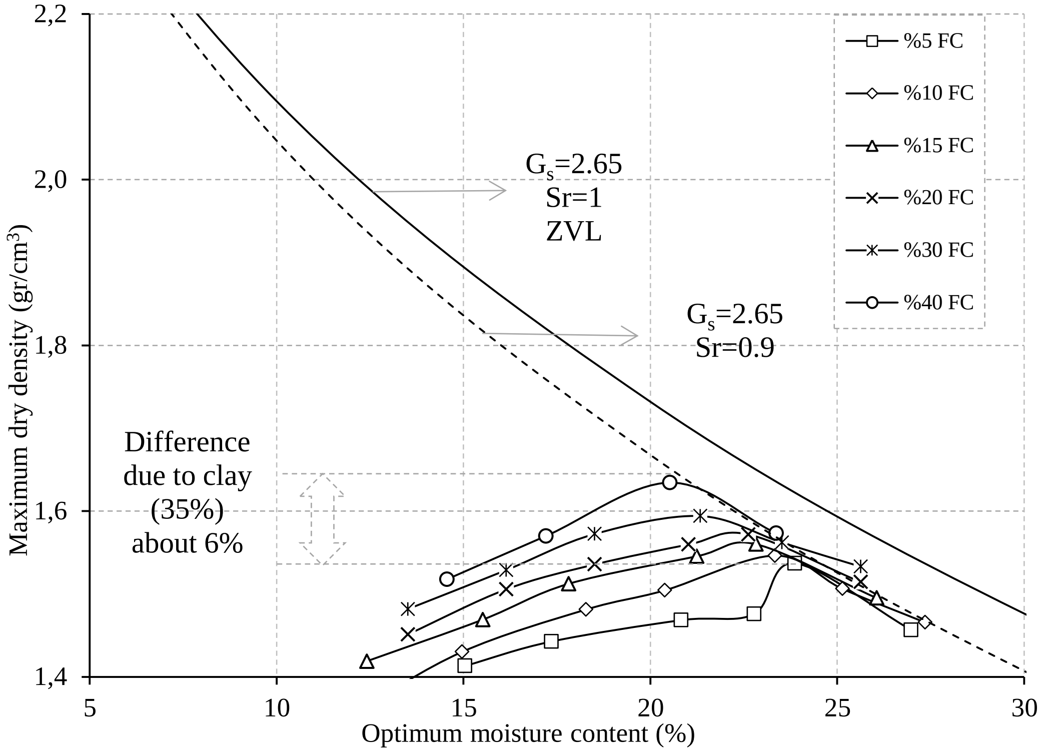


Figure 4.3 Variation of OMC with MDD for the clay with 0.15-0.30 mm CSS

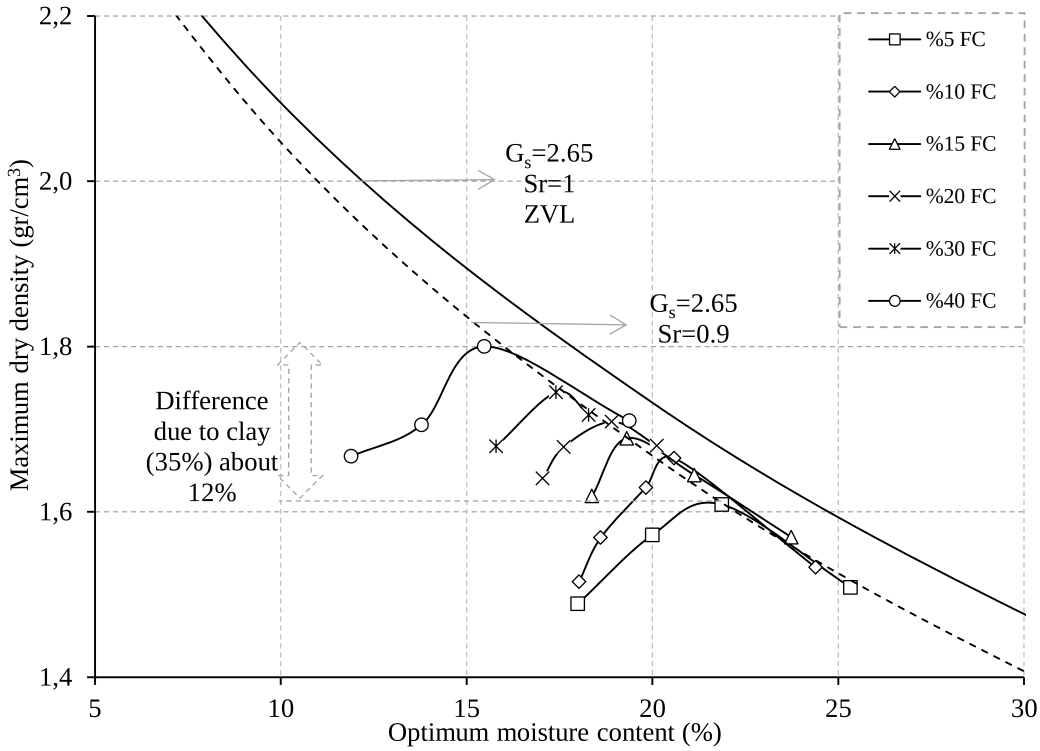


Figure 4.4 Variation of OMC with MDD for the clay with 1.0-2.0 mm CSS

4.1.2 Effect of sand gradation and shape

In order to observe the shape and gradation effects of the clean sand on compaction values, from Figure 4.5 to Figure 4.10 presents the compaction results of various fines content. At the special FC and gradation, mixtures prepared with NS have greater MDD and less OMC than those with CSS. The reason for NS have more MDD than CSS could be explain that the rounded particles have smaller void ratio and more interparticle contact area than the angular particles (Lee et al., 2017). Yun and Santamira (2008) and Lee et al. (2017) expressed that with the roundness of the grains, denser packing and interparticle contact area would increase. In addition, as Cho et al. (2006); Cavaretta et al. (2010); Shin and Santamira (2012) and Cabalar and Hasan (2013) pointed out, roundness particles less compressible than angular particles could be the reason why roundness particles has higher MDD. At a constant FC and shape characteristic, mixtures with coarse graded sands have higher MDD density and less OMC than those with fine graded. The author attributed this behavior to coarse particles are less compressible than fine as Cabalar and Hasan (2013) concluded. A similar result was also observed in Shakoor and Cook (1990)'s study that the bigger particles has more MDD and less OMC value. Figures 4.11 and Figure 4.12 also showed the MDD and OMC of mixtures at different fines content. In these figures, the mixtures prepared with NS 1.0-2.0 mm sands have the highest MDD value and smallest OMC, whereas the mixtures prepared with CSS 0.15-0.30 mm sand have the smallest MDD and the greatest OMC. The effects of gradation and shape of particles on compaction values are consistent in the literature. For instance, Shakoor and Cook (1990) proved that rounded stone particles have more MDD and less OMC than angular articles.

4.2 Minimum Void Ratio (e_{min})

The FC effect on the minimum void ratio of the mixtures was studied by many researchers. Polito and Martin (2000); Xenaki and Athanasopoulos (2003), Takch et al. (2016) expressed that the presence of fines decreased e_{min} of mixtures until a certain ratio (%10-%30) which beyond this ratio increased. Cubrinovksi and Ishihara (2002) explained this behavior fines were filled the sand voids up to a certain extent and that after the transition zone, the fines takes general behavior and the void ratio increases with fine grain. The e_{min} values obtained by Equation 4.1 are given in Figure 4.13.

$$e_{min} = \frac{\rho_s}{\rho_{dmax}} \tag{4.1}$$

Where, ρ_s is density of soil, ρ_{dmax} is maximum dry density obtained from compaction results. It was found that the fines have consistently reduced the e_{min} values. However, there is a continuous decrease in the e_{min} value of up to 40%, which is due to the fact that the used sand samples are poorly graded, so that the void ratio is high. A similar behavior was observed in the Cabalar and Hasan (2013) odometer test, where the void ratio of the mixtures was reduced to 30%. The conclusion of fine grains filling the voids of clean sand is consistent with the literature. This conclusion can also be observed in compaction results, where MDD continued to increase continuously with the addition of fine grains (Figure 6). It was observed that the rounded particles have a lower void ratio than angular particles. Vaid et al. (1985) performed a cyclic triaxial experiment on different confining and relative density sands of round ottova sand and angular tailings. They observed that the ottowa sand having rounded shape properties has a lower void ratio than a tailing having angular shape properties at a certain confining pressure. Bui et al. (2007) was stated that sphere or rounded particulars had fewer voids than angular particulars. Cho et al. 2006 also stated that the e_{min} value will increase with the reduction of the roundness of the particles. It was observed that coarse sand particles in the same fines content having the same shape characteristics have a smaller void ratio than fine sand particles. A similar result was obtained by Clayton et al. (2004).

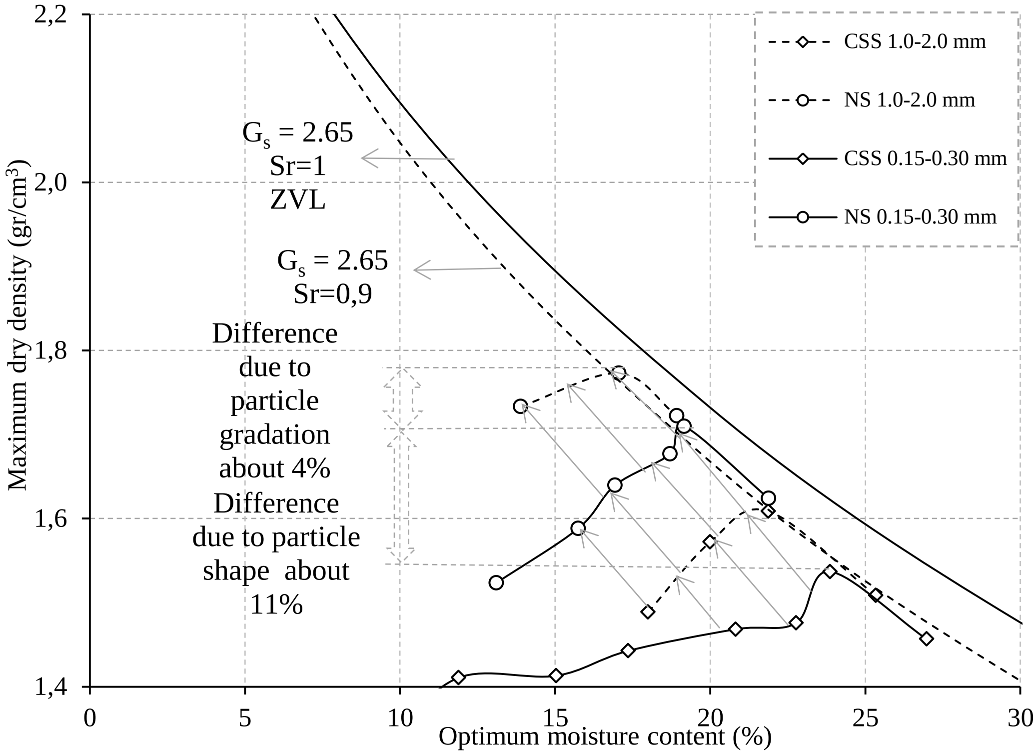


Figure 4.5 Variation of OMC with MDD for %5 the fines with different sand

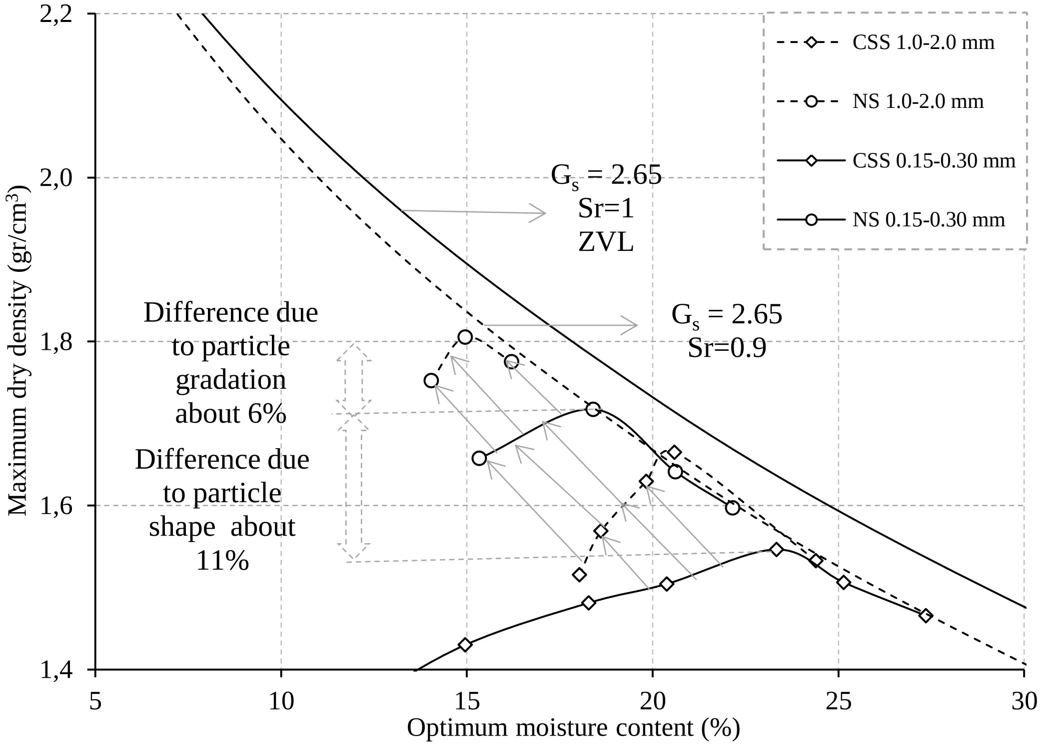


Figure 4.6 Variation of OMC with MDD for %10 the fines with different sand

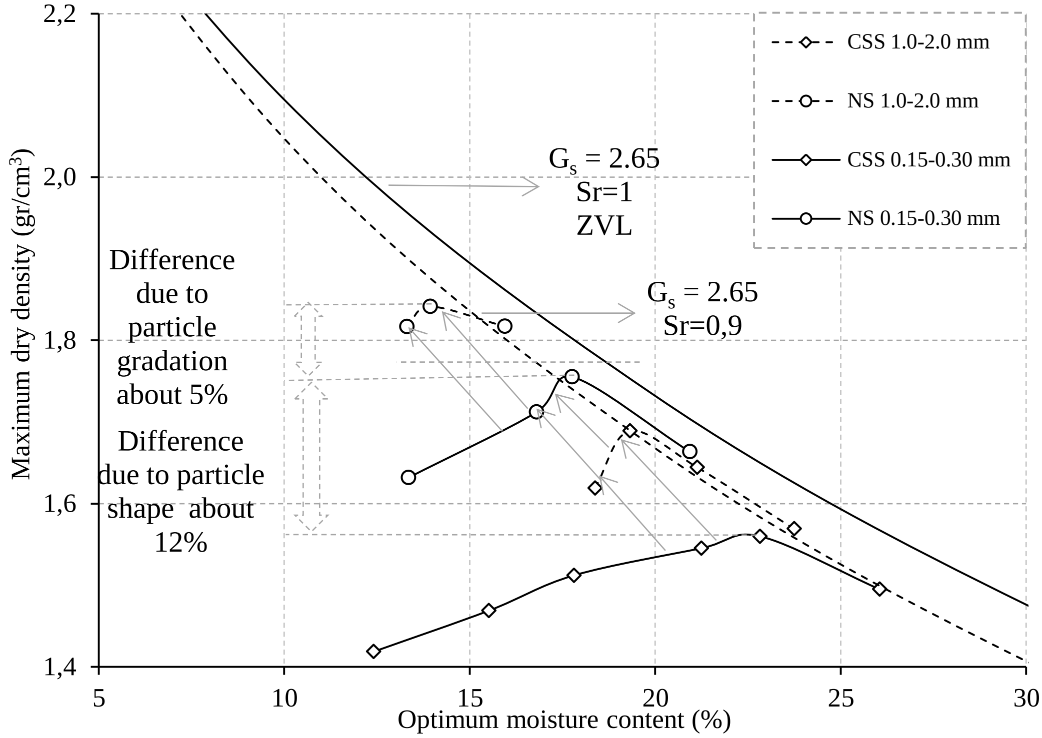


Figure 4.7 Variation of OMC with MDD for %15 the fines with different sand

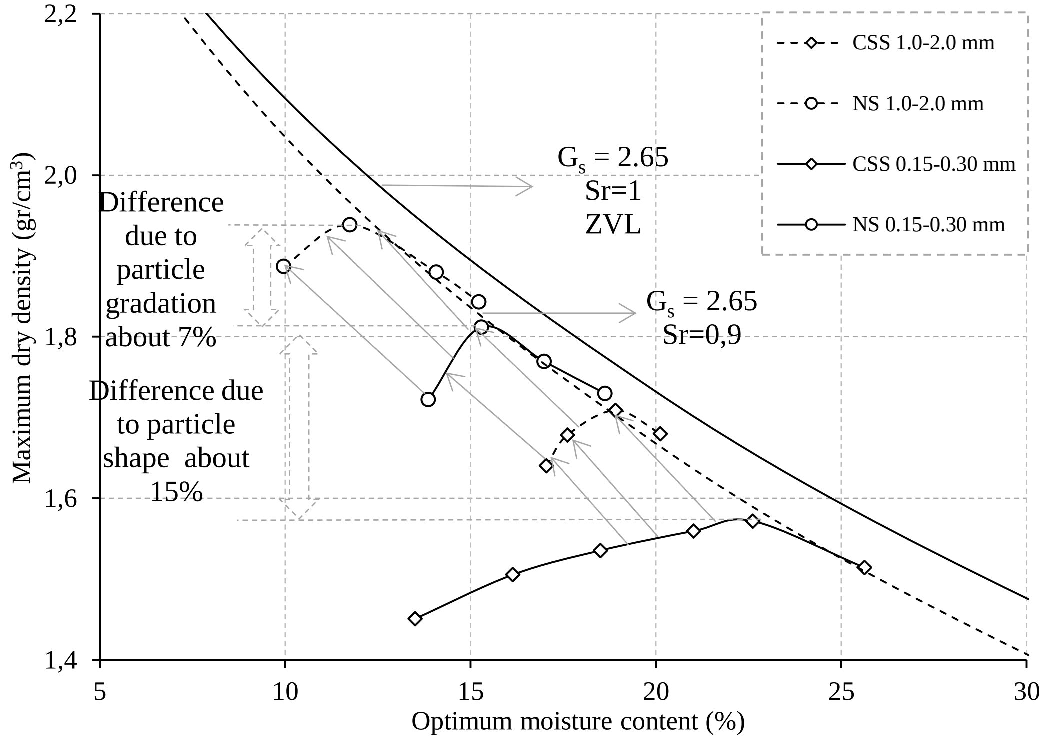


Figure 4.8 Variation of OMC with MDD for % 20 the fines with different sand

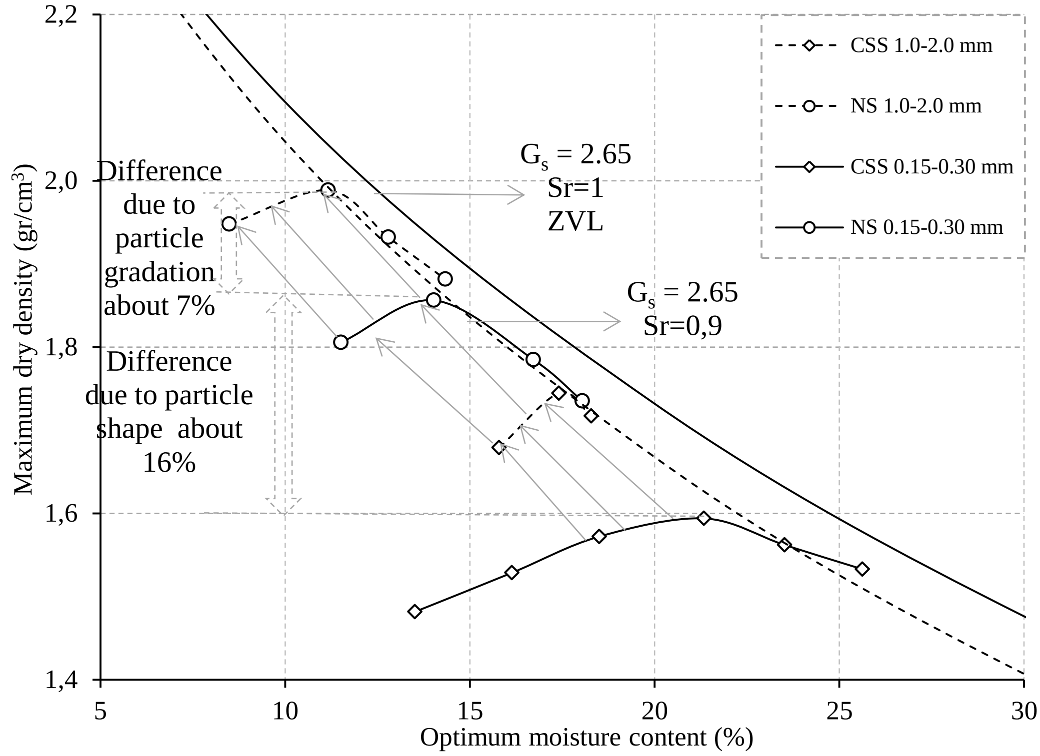


Figure 4.9 Variation of OMC with MDD for % 30 the fines with different sand

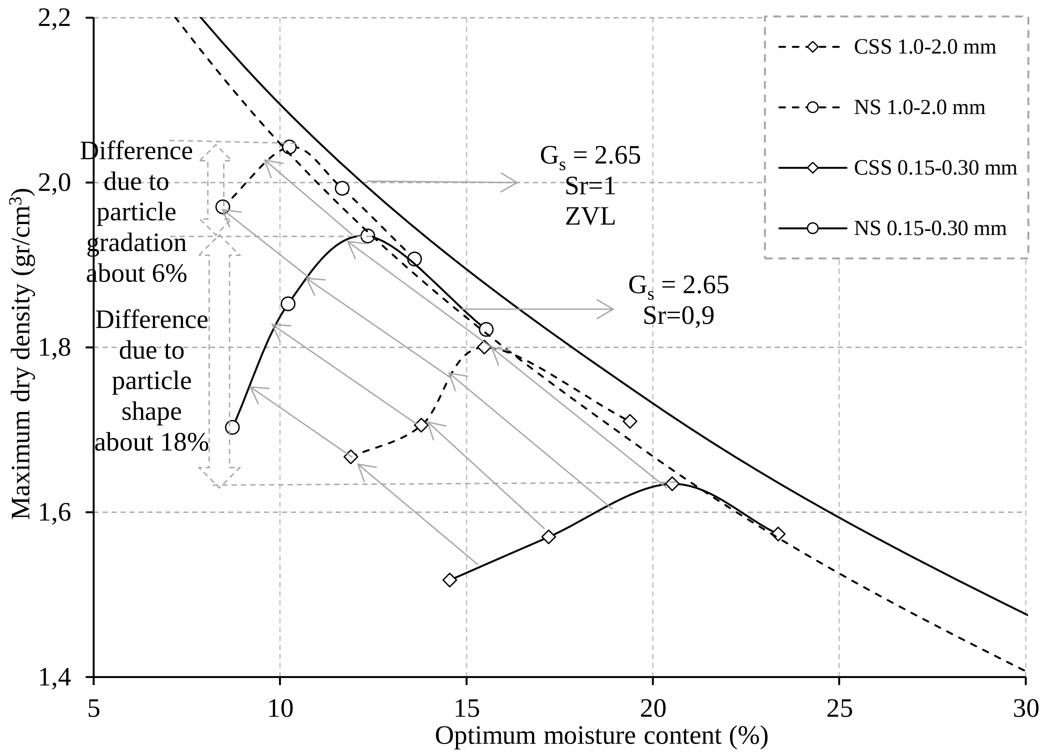


Figure 4.10 Variation of OMC with MDD for %40 the fines with different sand

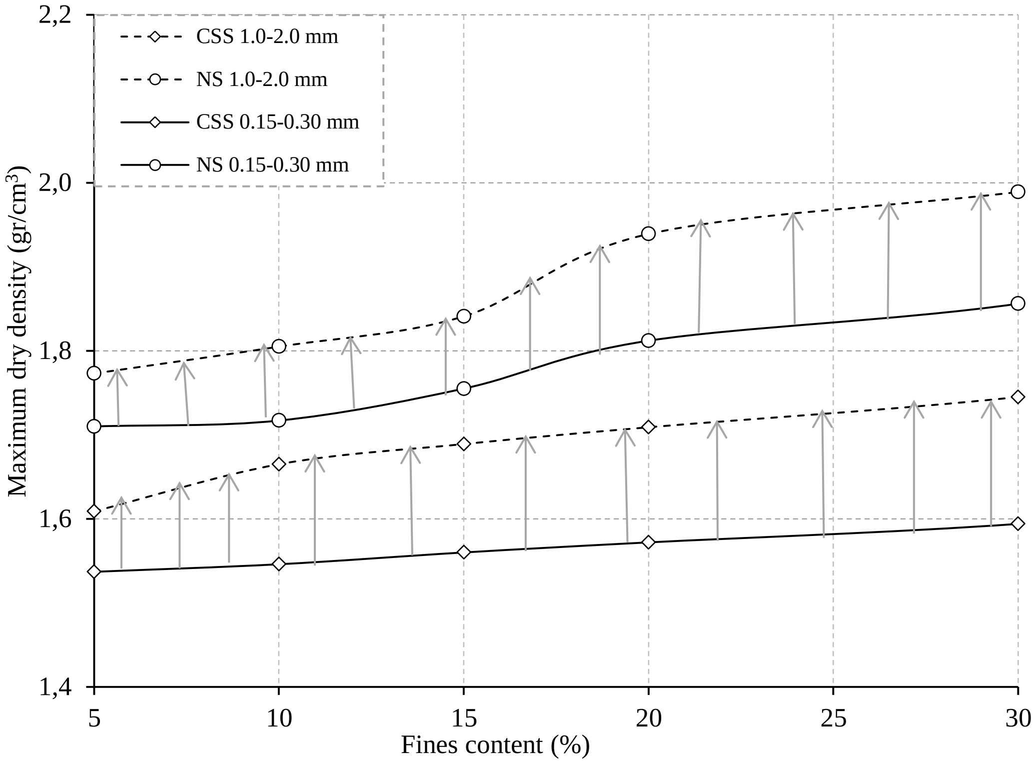


Figure 4.11 Variation of MDD for different mixtures

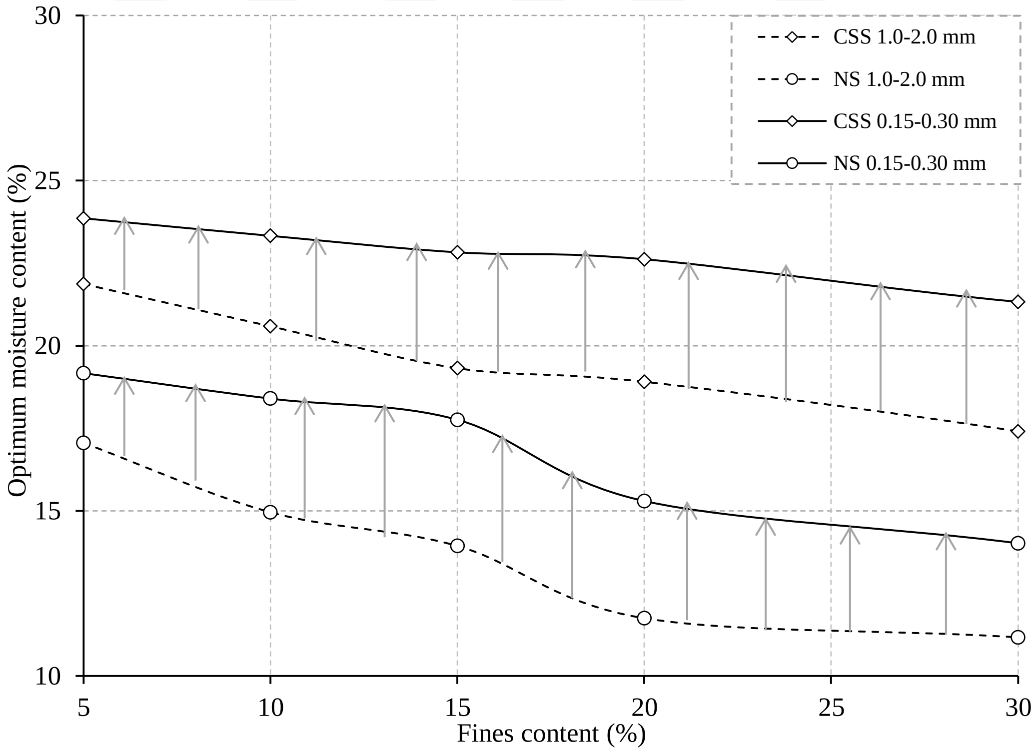


Figure 4.12 Variation of OMC for different mixtures

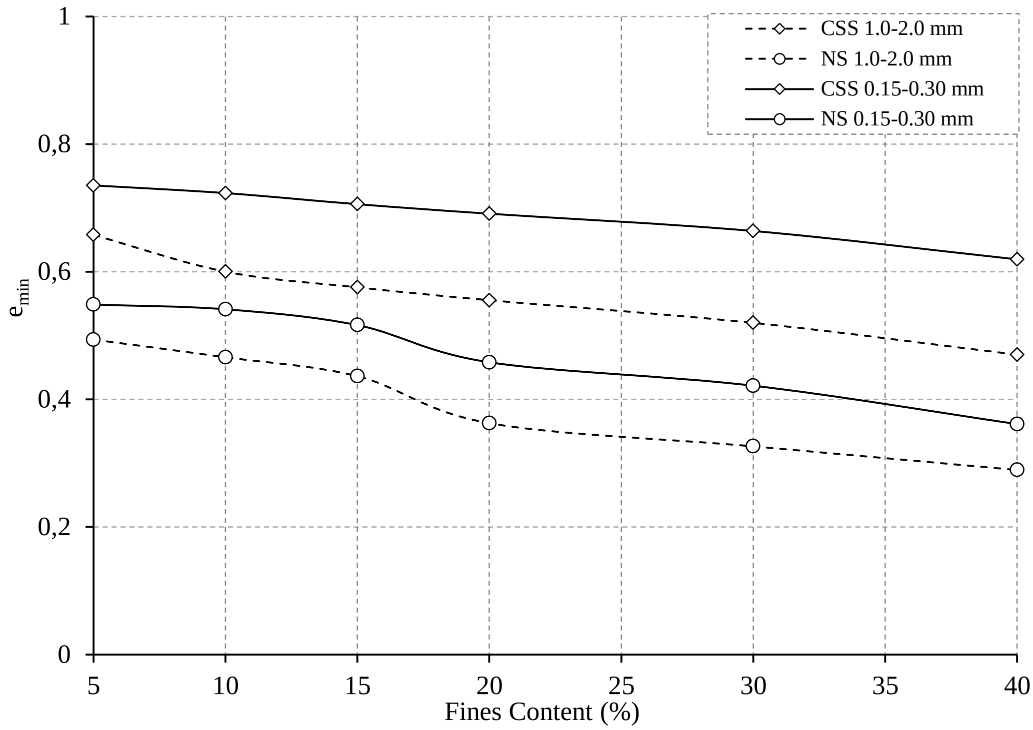


Figure 4.13 Variation of e_{min} for different mixtures

4.3 Bender Element

4.3.1 Effects of fines content

G_{max} value is an important criterion in the area of site response analysis and soil dynamics, where there were studies showing that G_{max} can be found by the Bender element, resonant colum test and torsional shear test (Youn et al., 2008). Among the factors affecting the G_{max} value confining stress, void ratio are come forward (Clayton, 2011). In addition to these effects, there are studies about the effect of FC on the G_{max} value. Salgado et al. (2000), Yang and Liu (2016); Goudarzy et al. (2016) expressed that G_{max} decreased with FC whereas Chien et al. (2002) found it increased to certain FC then decreased. In this study, the bender element test was performed at MDD and OMC as described in the previous chapter. There are examples in the literature of bender element experiments in both dry and saturated conditions (Cai et al., 2015). In addition Wiebe et al. (1998) and Cho and Santamarina (2001) indicated that there could be difference between unsaturated G_{max} and saturated G_{max}. Bender element experiment was performed at different frequency (10 kHz, 12 kHz, 15 kHz, 17 kHz, 20 kHz) values. In Figures from 4.14 to Figure 4.17, there are G_{max} values of different mixtures are shown. As the FC increased, the G_{max} values decreased and this behavior reversed after 20% FC. A similar result was found by Campbell (2006). Such behavior is consistent with many researchers who expressed that addition small amount of fines diminish the contact points between the coarse grains, and advocating that some of the geotechnical properties such as liquefaction resistance, undrained shear strength would decrease in this part (Thevenagam et al., 2000; Xenaki and Athanadpoulos, 2003). These researchers used the concept of intergranular void ratio (e_g) or interfine void ratio (e_f) instead of classical void ratio (e). According to this approach, the soil is considered to be coarse grains, fines grains and voids. When the fines were low, the fines float in the voids of the coarse grains, so the mechanical behavior is mainly governed by the coarse grain particle contact points. In this part, the e_g which reflecting to active particle contact points between coarse grains was used. The strength of the soil is reduced with addition of fines in this part. Fines fill void between coarse grains, fine grains govern the main behavior of soil when fines content exceeds the threshold fines content (FC_t). In this section, the concept of e_f is used which neglects dispersed coarse particles therefore soil strength increase with the addition of fines. These results are partially contradicted by researchers who say that the value of G_{max} is reduced by the FC. Carraco et al. (2009); Salgado et al. (2000); Yang and Liu (2016) were concluded that the G_{max} values continuously decreased 15%, 20% and 30%, FC respectively. In this study G_{max} values decreased to 20%, but then increased. The reason for this could be that most studies use non-plastic (Carraco et al., 2009; Salgado et al., 2000) whereas fines used in this study is a low plastic clay.

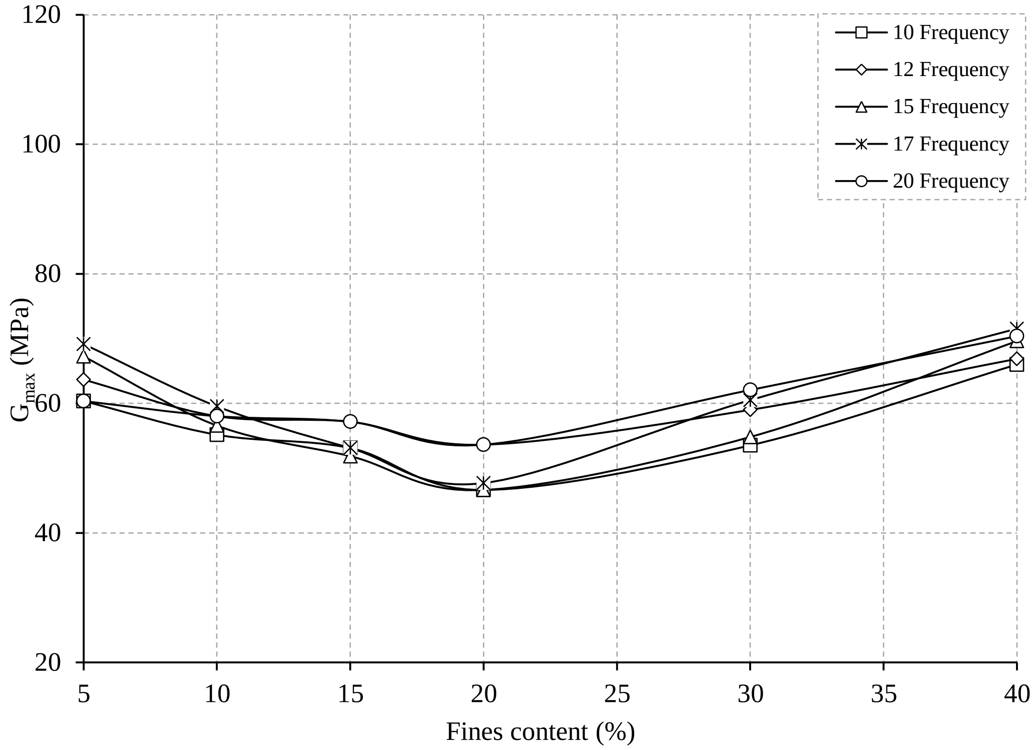


Figure 4.14 Variation of fines content with G_{max} for different frequency with 1.0-2.0 mm CSS

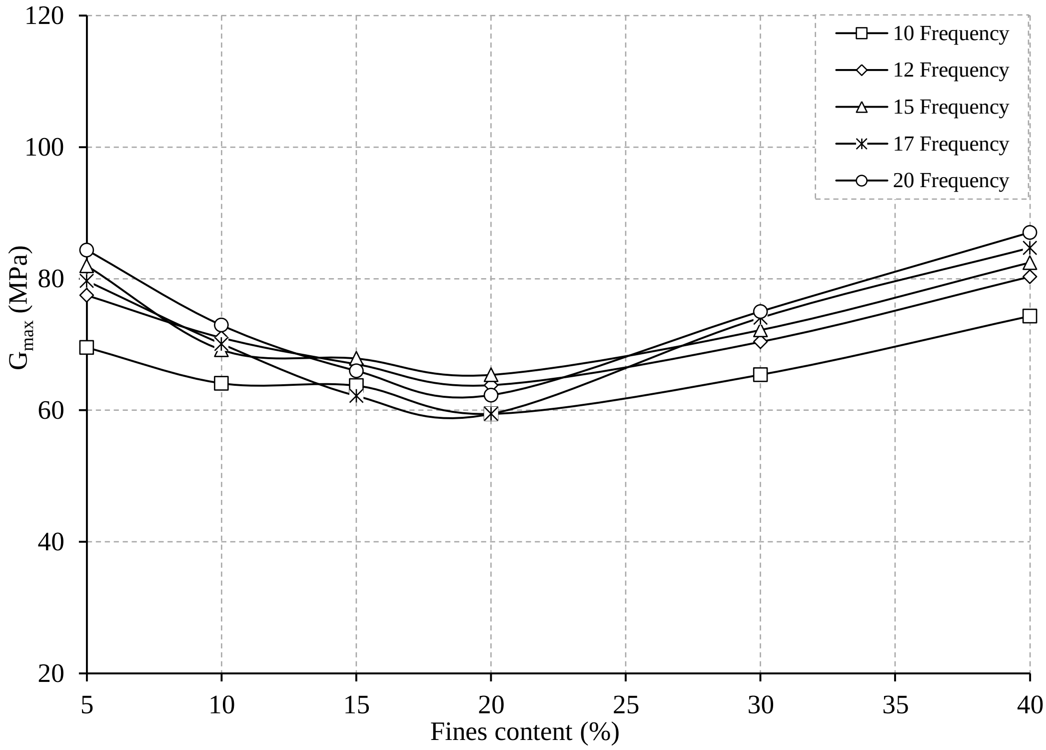


Figure 4.15 Variation of fines content with G_{max} for different frequency with 1.0-2.0 mm NS

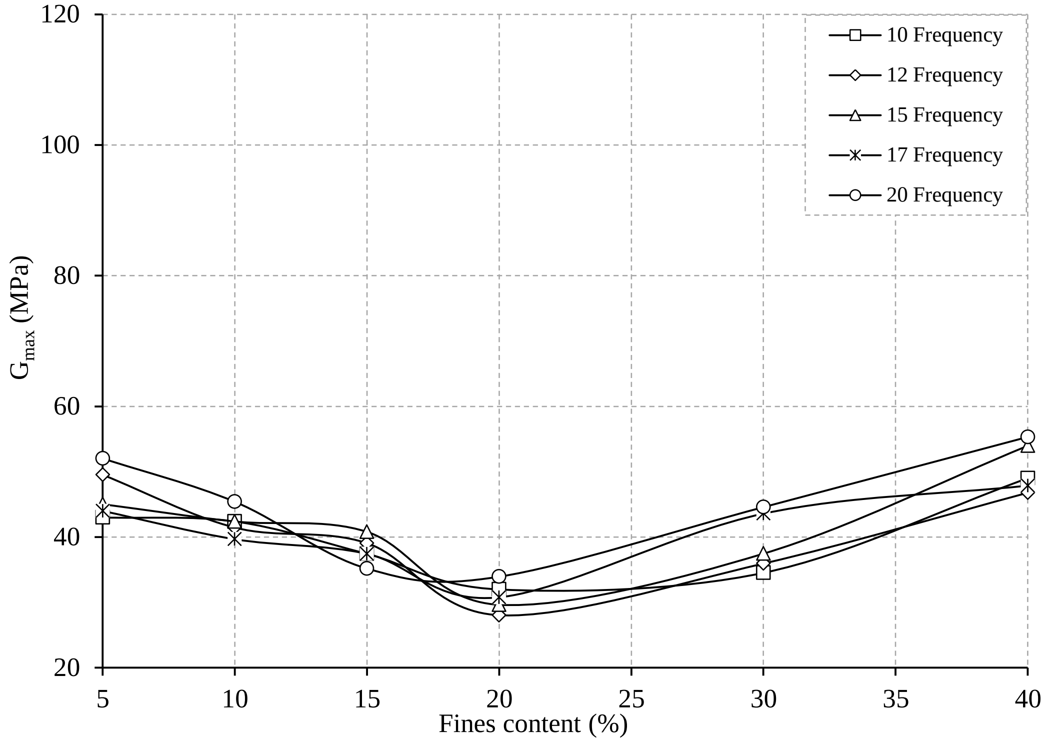


Figure 4.16 Variation of fines content with G_{max} for different frequency with 0.15-0.30 mm CSS

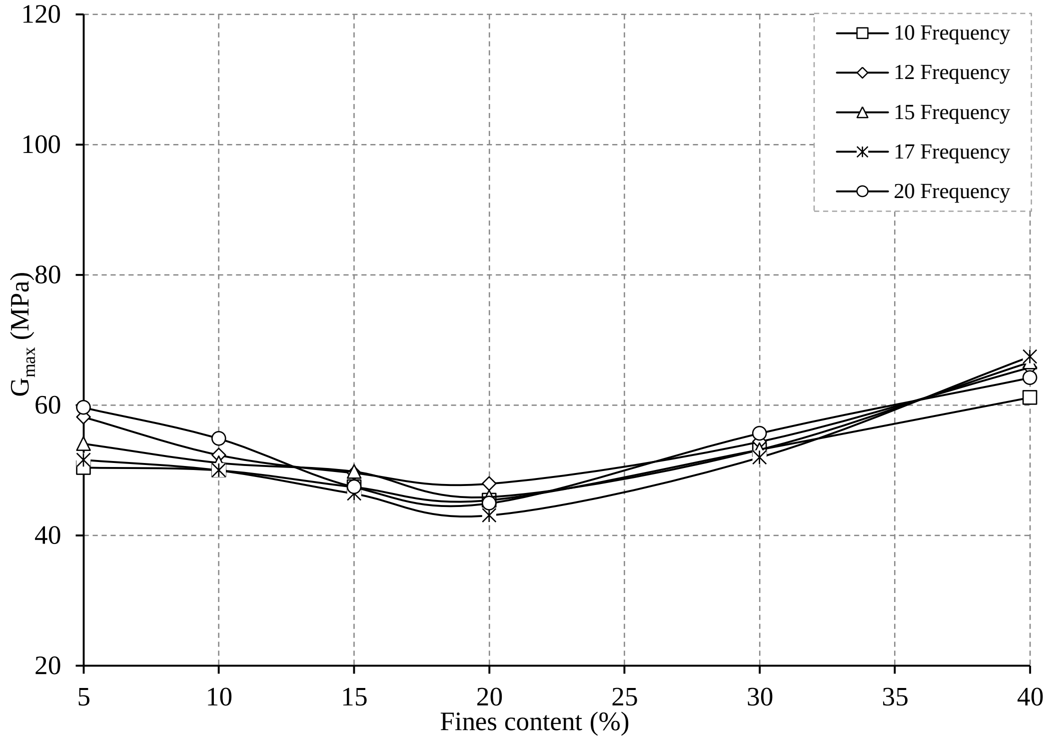


Figure 4.17 Variation of fines content with G_{max} for different frequency with 0.15-0.30 mm NS

4.3.2 Effects of frequency

It can be said that as the frequency increased, the G_{max} value increased in many part, but the consistency frequency effect cannot be observed. Similar results were found by Cai et al. (2015). The researchers found that although G_{max} values were increased with the frequency values at some unconfined stresses in the bender element test, a consistent relationship was not observed. Besides, Presti et al. (1997) stated that frequency effect of G_{max} value of dry sand samples were not observed.

4.3.3 Effect of sand gradation and shape

There are some studies in the literature about the effect of the coefficient of uniformity (Wichtmann and Triantafyllidis, 2009), D_{50} (Hardin and Kalinski, 2005) on G_{max} values. From Figure 4.18 to Figure 4.22 shown the effect of different shape and gradation of sand in the mixtures, presents the variation of G_{max} with the FC of four different mixtures types for various frequencies. The results indicated that mixtures with coarse graded of clean sands have greater G_{max} values than those with fine graded sands at a given fines content. Bui (2009) explained this behavior that as increased friction strength and G_{max} due to the fact that particle gradation decrease contact point therefore increase contact area and contact force (Figure 4.23). A similar result was observed by Kumar and Madhusubhan (2016). In their study, fine grain, medium grain and coarse grain sands were used that as for in this study, it could be considered that 0.15-0.30 sand as fine grain 1.0-2.0 sand as coarse grain. The results of the researchers indicated that at various confining pressures the course grain particles have greater G_{max} than the fine grains. This difference is especially obvious in high confining pressure. Wichtmann et al. (2015) concluded that at a given FC sands in the gradation of 0.04mm $<d<$ 0.063mm has greater G_{max} values than d $<$0.04mm. Hardin and Kalinski (2005), Menq and Stokoe (2003) also stated that the coarse gradation have high G_{max} values. Regarding the effect of the shape of the grains on the G_{max}, the mixtures prepared with rounded grain sand have higher G_{max} values than the angular ones. In the literature, it is clear that in many studies round particles have less voids than angular particles (Vaid et al., 1985; Bui et al., 2007). There are many studies in which the G_{max} values are inversely proportional to the void ratio parameter (Bui et al., 2007; Presti et al., 1997). Therefore it is reasonable that mixtures with rounded particles obtained higher G_{max} values. Besides, the G_{max} values are small for CSS sand because the CSS has rough surface.

Santamarina and Cascante (1998) performed a resonant column test to examine of the effects of different surface roughness concluded that as the roughness of the surfaces increased, the G_{max} value decreased. Bui (2009) Lee et al. (2017) found that rounded particles have higher G_{max} values than angular particles. Bui (2009) was attributed decreasing particle sphericity cause decrease G_{max} to because as the particles angularity lead to irregularity increase, therefore number of contact point increase lead to decrease number of contact area (Figure 4.24). Cho et al. (2006) indicated that as roundness, sphericity and regularity decreased, the shear wave velocity decreased. There are several methods in the literature to evaluate liquefaction resistance (Boulanger et al., 1997).

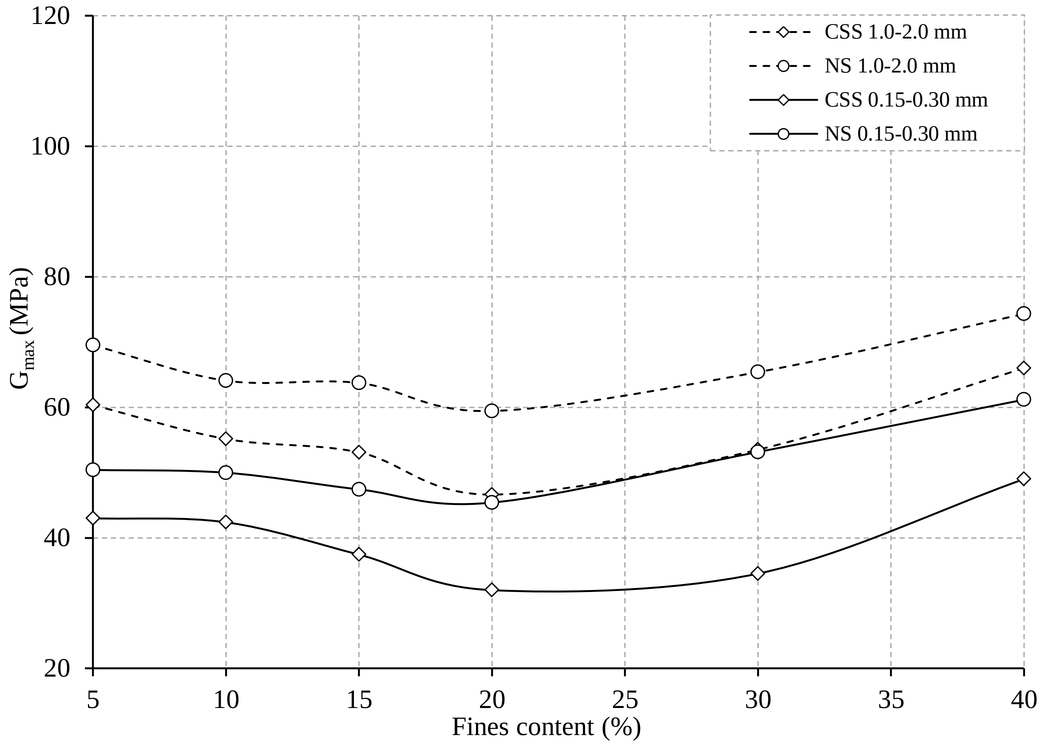


Figure 4.18 Varition of fines content with G_{max} for 10 frequency with different mixtures

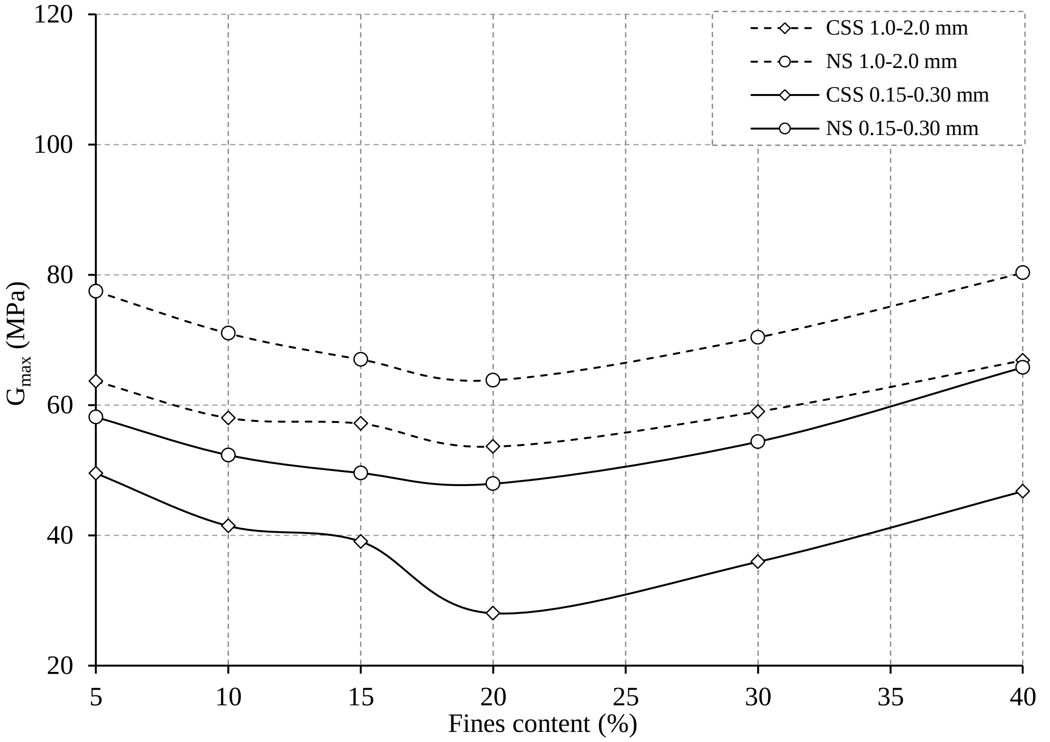


Figure 4.19 Varition of fines content with G_{max} for 12 frequency with different mixtures

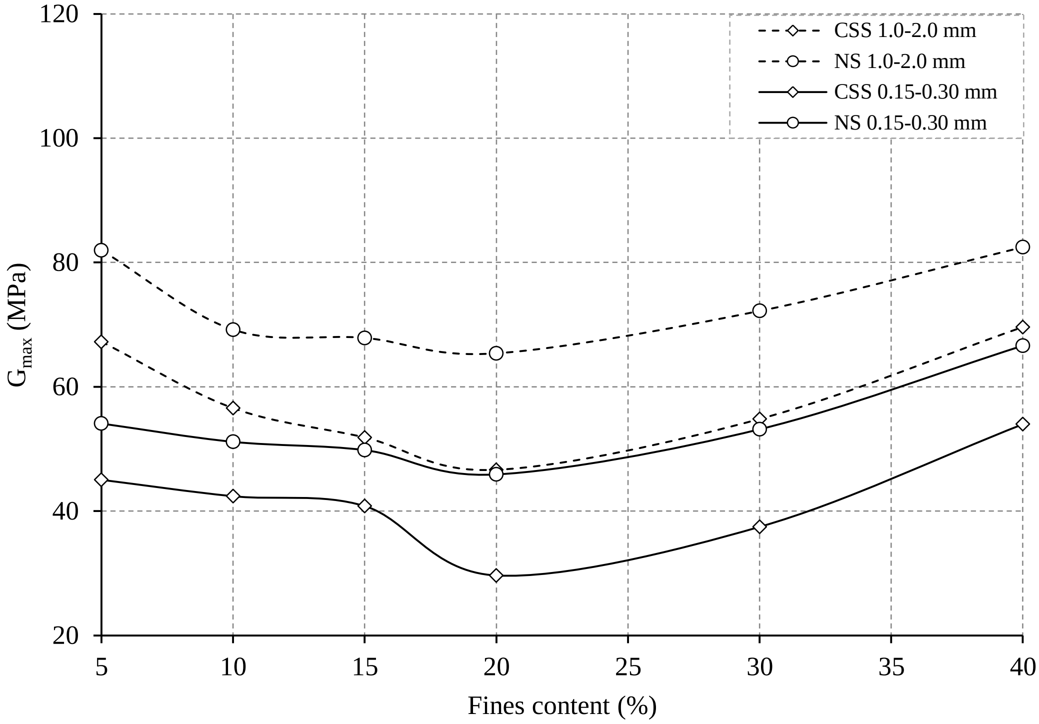


Figure 4.20 Varition of fines content with G_{max} for 15 frequency with different mixtures

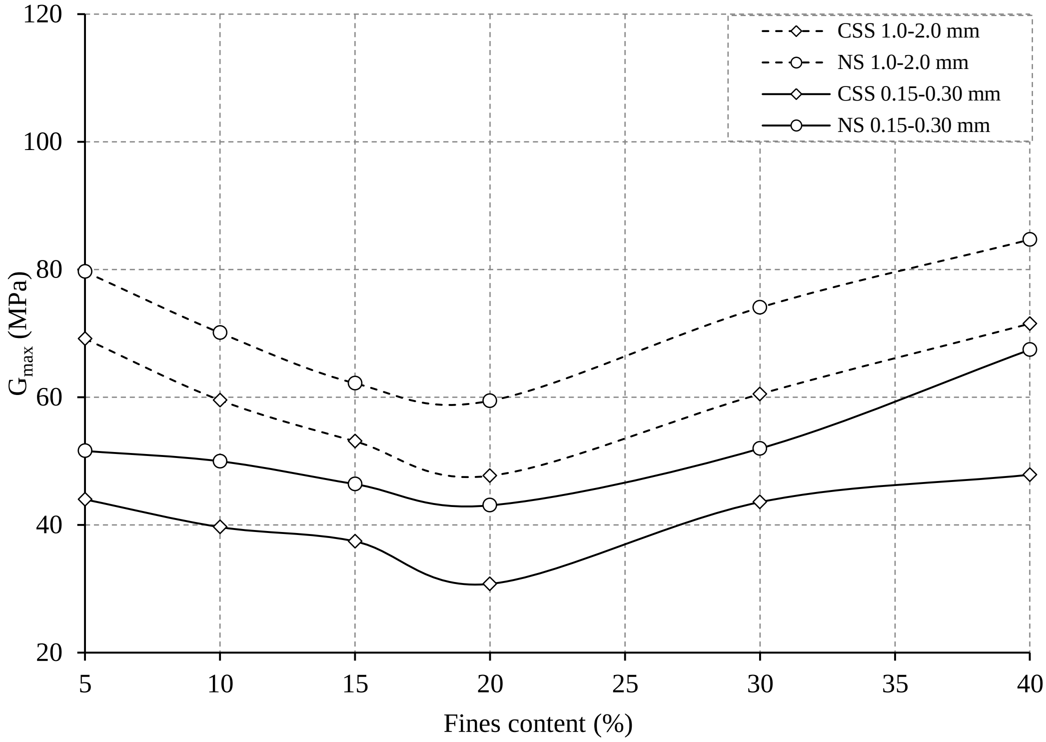


Figure 4.21 Varition of fines content with G_{max} for 17 frequency with different mixtures

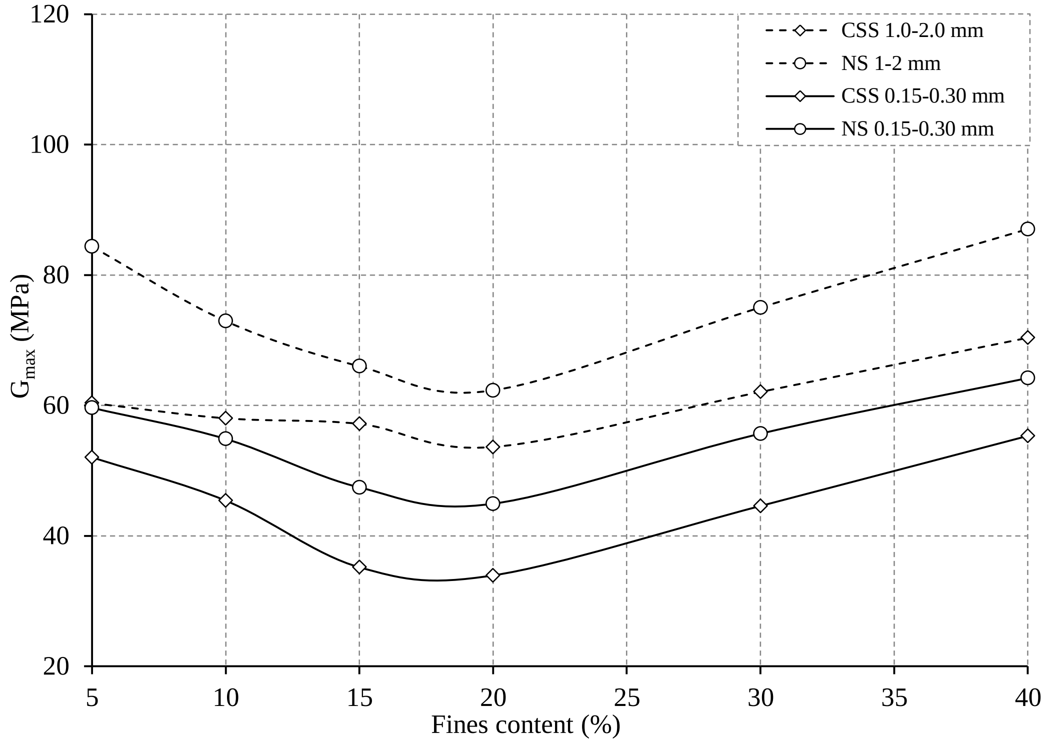


Figure 4.22 Varition of fines content with G_{max} for 20 frequency with different mixtures

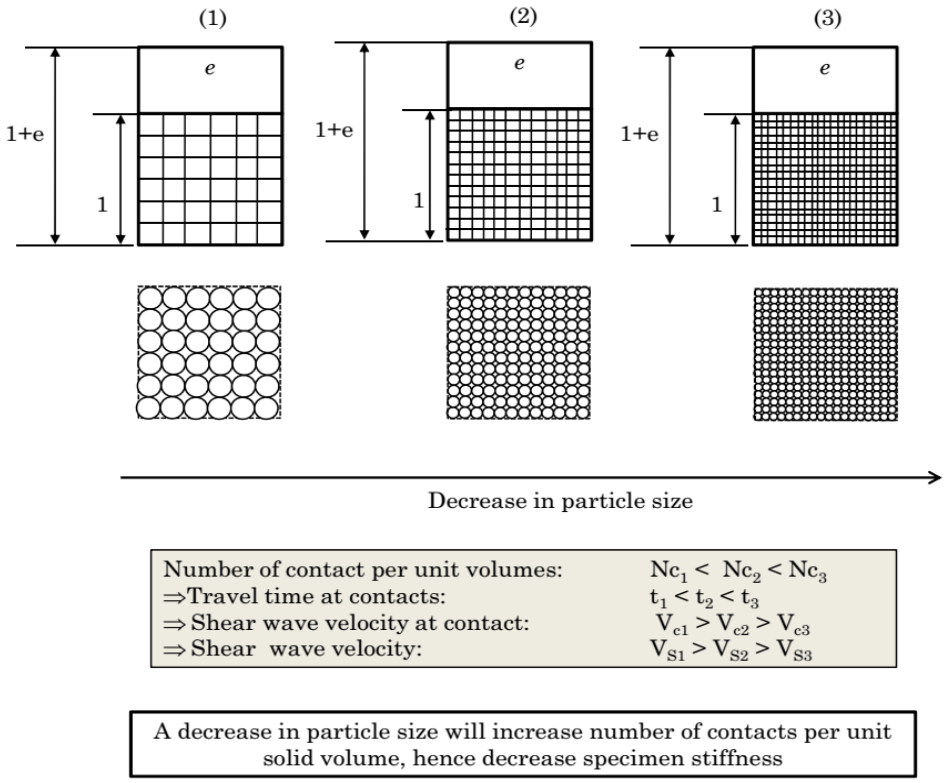


Figure 4.23 Hypothesis for the effect of particle size on G_{max} (Bui, 2007)

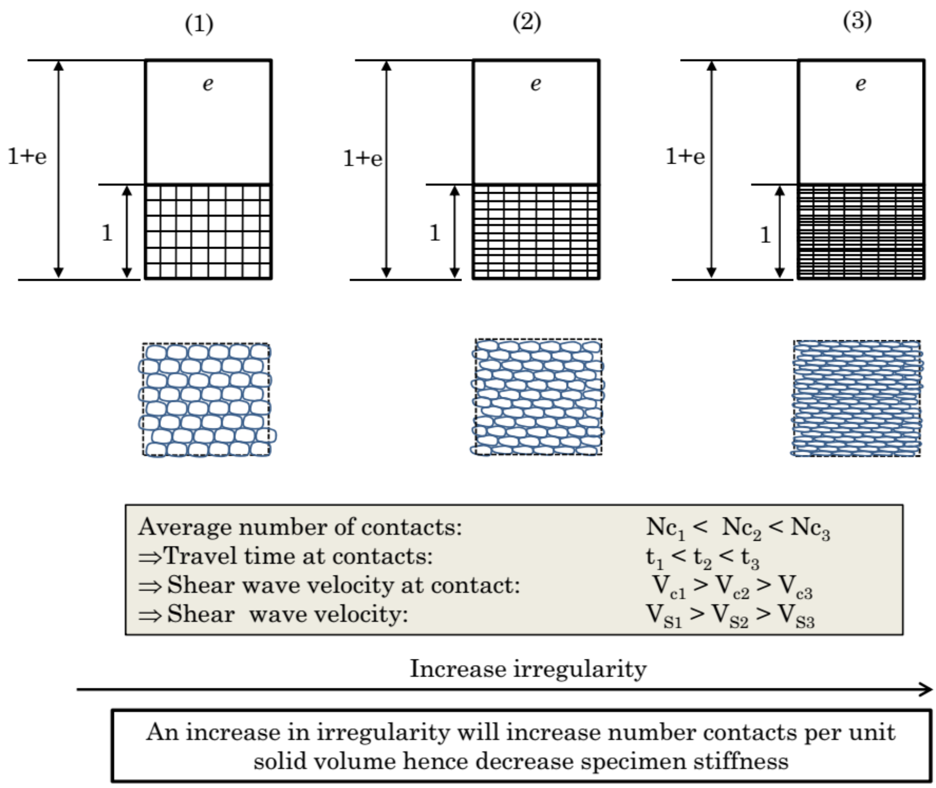


Figure 4.24 Hypothesis for the effect of particle shape on G_{max} (Bui, 2007)

4.4 Liquefaction Resistance

4.4.1 Effects of fines content

There are several methods in the literature to evaluate liquefaction potential (Boulanger et al., 1997). Numerous researchers evaluated liquefaction resistance using SPT values (Tokimatsu and Yoshimi, 1983; Seed et al., 1985), CPT (Robertson and Campella, 1985; Stark and Olson, 1985; Boulanger et al., 1997) and using Vs (Andrus and Stoke, 2000). In this study, a dynamic bender element test was carried out to evaluating liquefaction potential, and CRR values calculated as Yunmin et al. (2005) and Zhou et al. (2005) as described in equation 4.2.

$$CSR_{tx} = \frac{Kn^2 G_{max}{}^2}{F^2\left(e_{min}\right)\, r_m{}'} \tag{4.2}$$

where, Kn is a coefficient that the researchers were proposed a value of $1,22*10^{-4}$ Kpa $^{-0,5}$ as results of end result of comparison of bender element and triaxial experiments in the literature. Zhou et al. (2005) expressed Fe = $(2,97\text{-}e)^2/(1+e)$ for angular grains and Fe=$(2,17\text{-}e)^2/(1+e)$ for rounded grains. r_m' is confining stress and taken as equation 4.3.

$$r_m{}' = \frac{1+2K_0}{3} r_v \tag{4.3}$$

where, r_v is the vertical stress which all the experiments were carried out at 600 kPa r_v. K_0 was assuming 0.5 according to Tokimatsu and Uchida (1990). They suggested any value between 0.5 and 1.0 might be assumed for practical purposes. The CSR value at which the liquefaction initiated for each mixture formed according to these calculations are shown from Figure 4.25 to Figure 4.29. The CSR values decreased up to 20% with the fine content but increased after this ratio. In the literature, there are many studies as well as many comments as the effect of fines content on liquefaction. In recent years, many researchers concluded similar results. They found that the liquefaction resistance decreased to FC_t but increased after this ratio (Chang et al., 1982; Polito and Martin, 2000; Thevenagam et al., 2000; Das and Sitraham, 2001; Xenaki and Athanadpoulos, 2003; Ueng et al., 2004; Popodopulo and Tika, 2008; Karim and Alam, 2014). Thevenagam (2000) expressed as a reason for such behavior is that in the mixture the fine grains floats in voids in clean sand up to a certain ratio and fines cut the contact points between the sand grains and reduces the resistance to liquefaction. After this ratio, the liquefaction resistance is increased because the fine grains take the main behavior of soil matrix. In this study, FC_t was found to be 20%. FC_t found by Chang et al. (1982) to be 10%, by Thevanayagam et al. (2000) 25%, by Polito and Martin (2000) 35%, by Xenaki and Athanasopoulos (2003) 44%, by Ueng et al. (2004) 20%, by Popodopulo and Tika (2008) % 35, by Chang and Hong (2008) from 17 to 26%, by Dash and Sitharam (2011) 21%, by Karim and Alam (2014) 30% and by Karakan and Altun (2016) 40%. As a reason for the relatively small FC_t value of this study; the fines used in this study is of low plasticity. In the most of the reference study, non-plastic fine grains were used. As Grecevez (2006) pointed out, as the plasticity value of fines increased, the liquefaction resistance increased. For this reason, the FC_t found to be %20 in this study which is around 25-40% in most reference studies. However, FC_t in this study is consistent with Ueng et al (2004), Chang and Hong (2008) and Dash and Sitharam (2011) found the threshold ratios of 20%, 17% and 21%, respectively.

4.4.2 Effect of sand gradation and shape

Mixtures prepared with coarse sand gradation showed greater liquefaction resistance than those with fine gradation (from Figure 4.25 to Figure 4.29). This approach was also observed in study of Belkadit et al. 2011. Moreover Figueroa et al. (1999) found energy values to initiate liquefaction concluded that, courser sands grains require more energy to initiate liquefaction than fine sands. Polito (1999) found that as the value of D_{50} increased, liquefaction resistance will increase in clean sands particles. In addition, rounded particles showed greater liquefaction resistance than angular particles. Similar with found by Vaid et al. (1985) showed at low confining pressure values, angular particles showed more resistance to liquefaction, while at higher pressures, rounded particulars showed more resistance concluded that angular particles are susceptible to liquefaction.

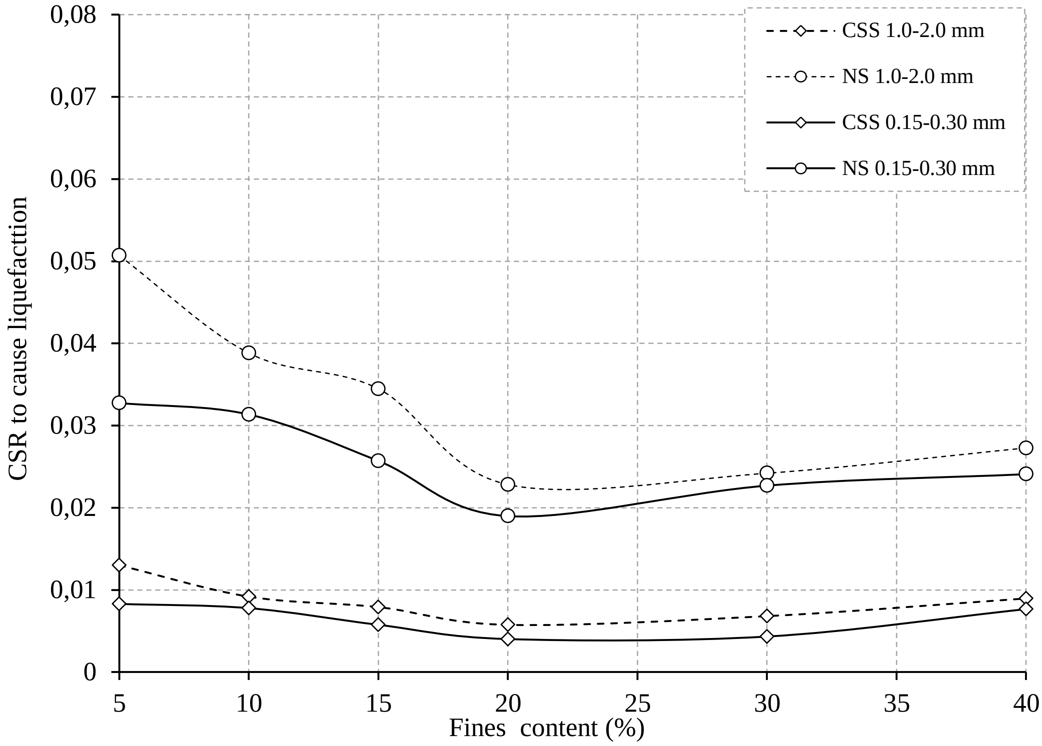


Figure 4.25 Varition of fines content with CSR for different mixtures at 10 frequency

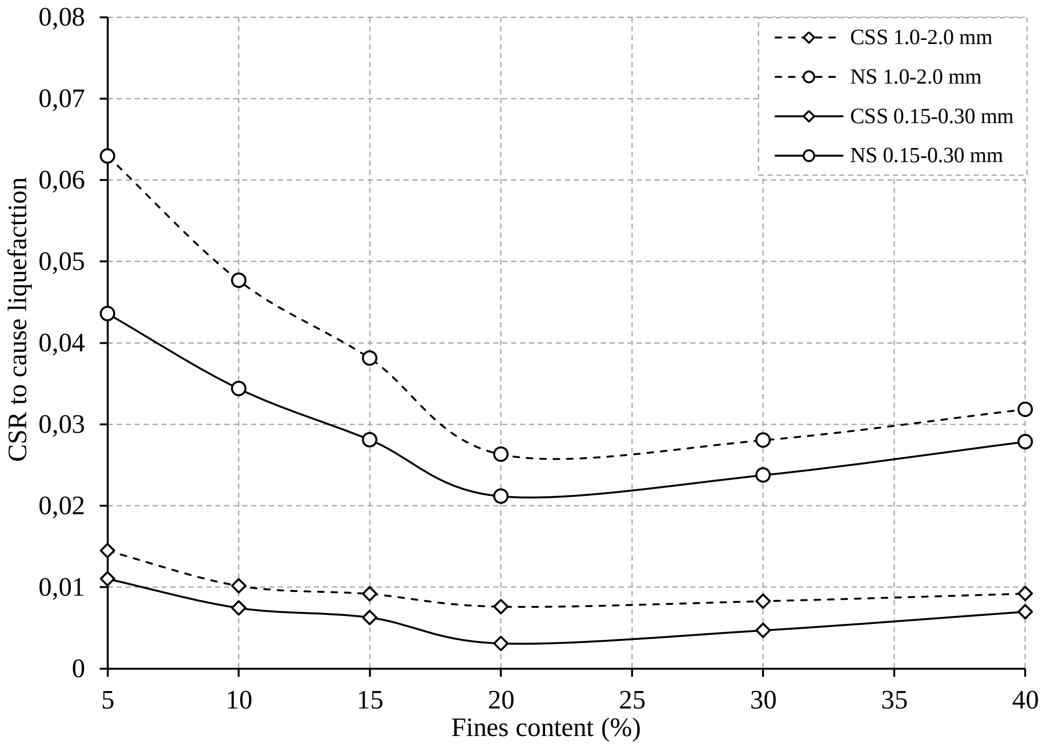


Figure 4.26 Varition of fines content with CSR for different mixtures at 12 frequency

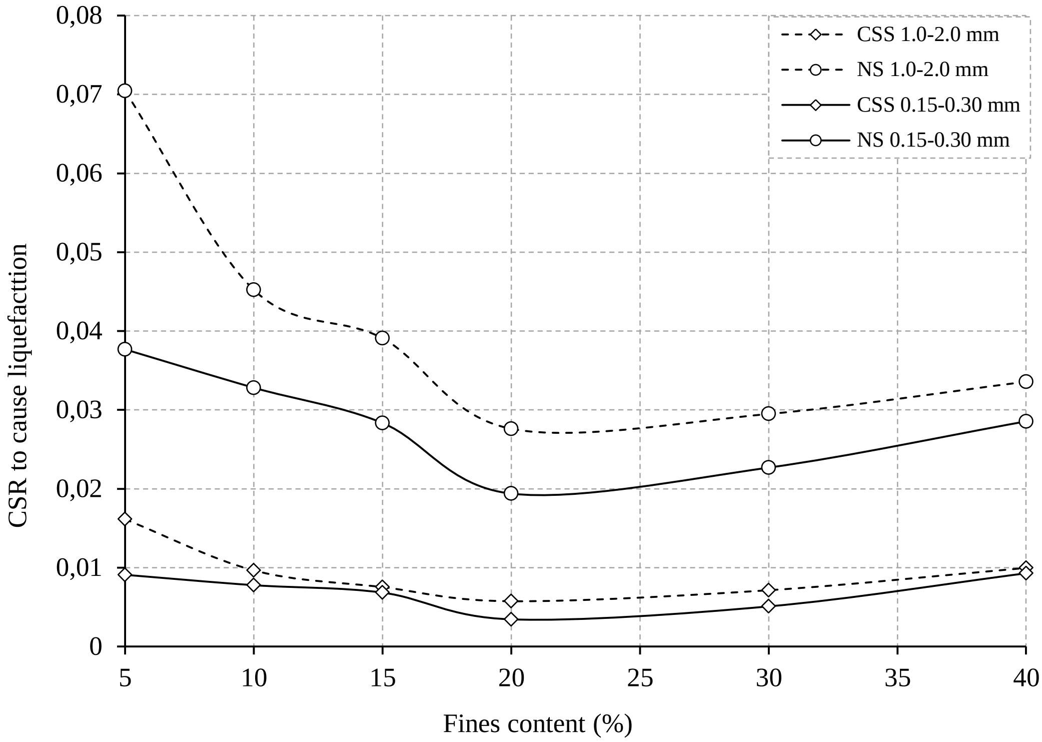


Figure 4.27 Varition of fines content with CSR for different mixtures at 15 frequency

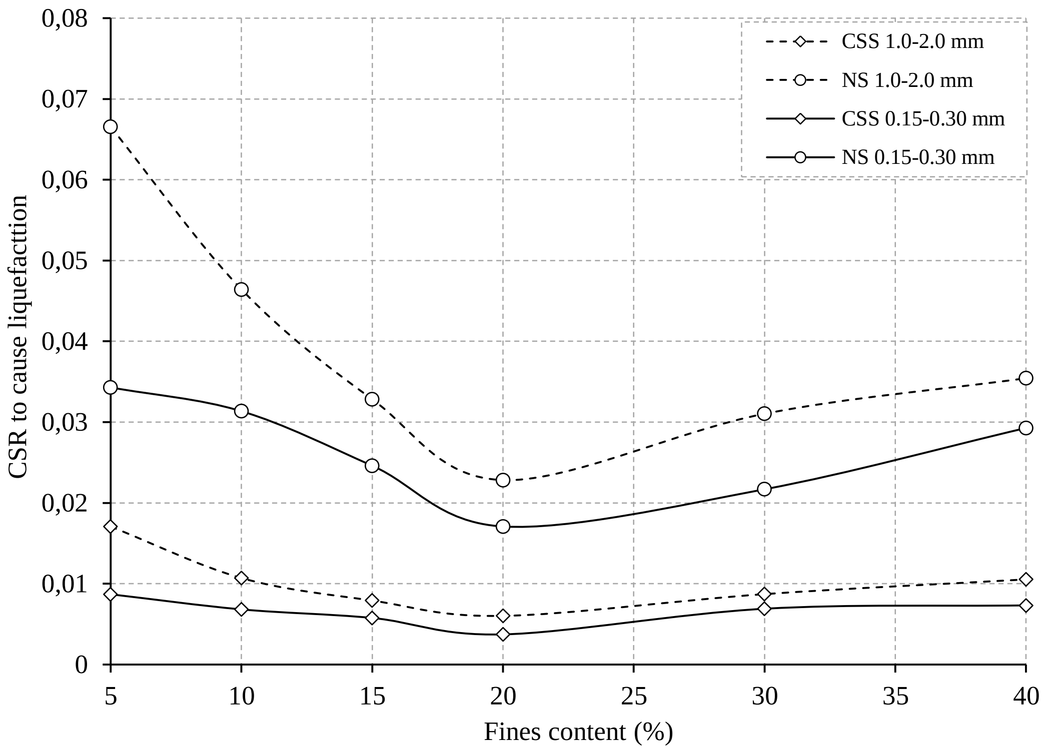


Figure 4.28 Varition of fines content with CSR for different mixtures at 17 frequency

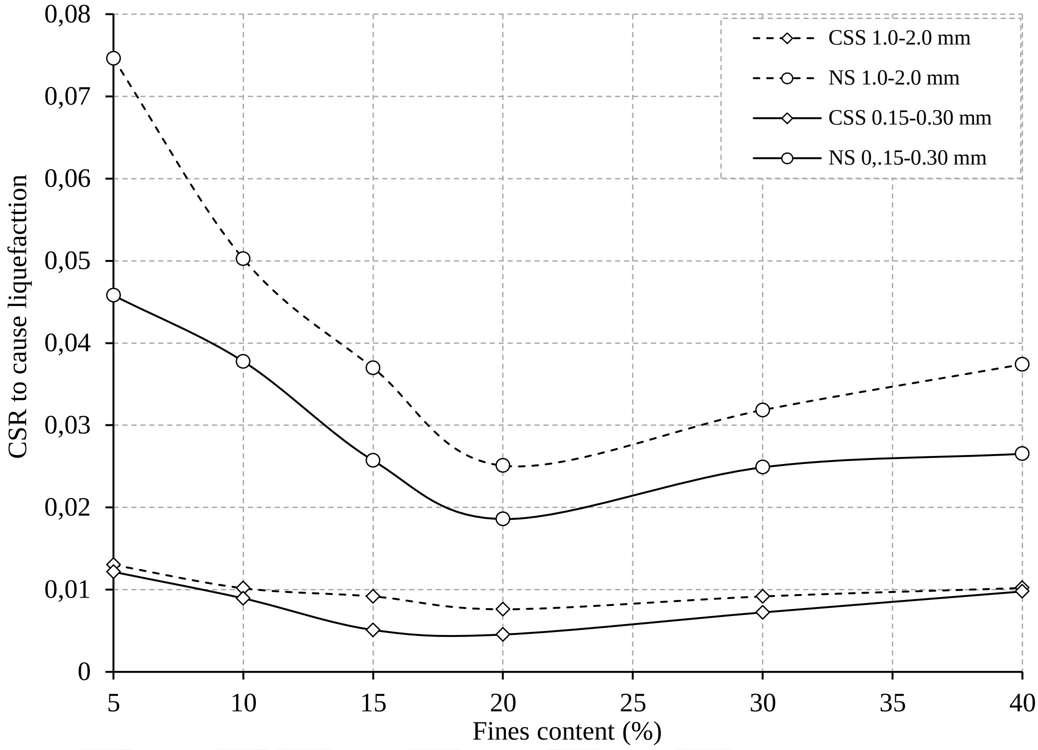


Figure 4.29 Varition of fines content with CSR for different mixtures at 20 frequency

CHAPTER 5

CONCLUSIONS

In this study, Bender element test was carried out for MDD and OMC of sand with various fines mixtures and liquefaction resistance was evaluated with G_{max} value. The fines (< 0.075mm) was added to two different particle gradations (0.15-0.30 mm, 1.0-2.0 mm) of clean sand having distinct shapes (rounded and angular) with mixture ratio of 5%, 10%, 15%, 20%, 30% and 40%. The following conclusions can be drawn in this study.

- As the FC increased, the MDD values increased and the OMC value decreased. It can be concluded that fine grains filled the voids into the sand. Mixtures with coarse graded (1.0-2.0) clean sand have greater MDD values than those with fine grain (0.15-0.30) and less OMC values than those with fine particles at a given FC. Rounded NS has greater MDD and less OMC than angular CSS at a special FC.
- G_{max} values decreased with the fines content up to 20% which after increased. Mixtures with coarse graded clean sand has greater G_{max} values than those with fine grain mixtures and less OMC values than those with fine particles at a given FC. Rounded NS has and greater MDD and less OMC than angular CSS at a special FC.
- CSR value to initiate liquefaction is calculated for each mixture as given by Yunmin et al. (2005) and Zhou et al. (2005). The FC_t found to be %20. The mixtures with coarse graded clean sand showed greater liquefaction resistance than those with fine sands at a given FC. Mixtures prepared with the rounded sand showed greater resistance to liquefaction than angular sandy soils at a special FC.

RECOMMENDATIONS

This study evaluate the effects of FC, gradation and shape effect of sand, frequency on compaction and liquefaction with bender element. Future works can focus on utilizing different experimental process such as dynamic triaxial, static triaxial, resonant column. Different clay types can be used. Effects of plasticity, size and shape of clay can be evaluated.

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