

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**DETERMINATION OF VOLUMETRIC STRAINS IN FULLY AND INDUCED
PARTIALLY SATURATED SANDS THROUGH DYNAMIC SIMPLE SHEAR
TEST DEVICE WITH CONFINING PRESSURE**

M. Sc. THESIS

Tuğba Bahar BAHŞI

Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Programme

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**TAM DOYGUN VE KISMİ DOYGUNLUĞA İNDİRGENMİŞ KUMLARIN
HÜCRE BASINÇLI DİNAMİK BASİT KESME DENEYLERİ İLE
HACİMSEL DEFORMASYONUN TAYİN EDİLMESİ**

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To whom I love,



FOREWORD

Liquefaction of saturated granular soils during earthquakes has been one of the most important problems in the field of geotechnical earthquake engineering. Previous laboratory studies on the effect of degree of saturation on volumetric strains under cyclic loading have not yet reached a consensus. This research presents an effort to find a relationship between the degree of saturation and volumetric strains under drained conditions.

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TABLE OF CONTENTS

	<u>Page</u>
FOREWORD	ix
TABLE OF CONTENTS.....	xi
ABBREVIATIONS	xiii
SYMBOLS	xv
LIST OF TABLES.....	xvii
LIST OF FIGURES	xix
SUMMARY	xxiii
ÖZET	xxv
1. INTRODUCTION.....	1
2. REVIEW OF LIQUEFACTION STUDIES.....	3
2.1 Liquefaction Definition	3
2.1.1 Flow liquefaction.....	6
2.1.2 Cyclic mobility.....	6
2.2 Evaluation of Liquefaction	6
2.2.1 Cyclic stress approach	6
2.2.1.1 Characterization of liquefaction resistance based on laboratory test... 8	
2.2.1.2 Characterization of liquefaction resistance based on in-situ tests	9
2.2.2 Cyclic strain approach	11
2.2.2.1 Characterization of liquefaction resistance	11
2.2.3 Effective stress-based response analysis approach	12
2.3 Cyclic Laboratory Tests for Liquefaction Studies	16
2.3.1 Small strain tests.....	18
2.3.2 Large strain tests.....	18
2.3.2.1 Cyclic triaxial test.....	18
2.3.2.2 Cyclic simple shear test.....	20
2.3.2.3 Cyclic torsional shear test	22
2.4 Partial Saturation as a Liquefaction Mitigation Method	23
3. STRAIN CONTROLLED CYCLIC TESTING PROGRAM.....	27
3.1 Dynamic Simple Shear System With Confining Pressure.....	27
3.2 Properties of Soil Tested.....	29
3.3 Sand Specimen Preparation	30
3.3.1 Fully saturated sand sample preparation.....	31
3.3.1.1 Moist tamping method	32
3.3.2 Partially saturated sand sample preparation.....	33
3.3.2.1 Wet pluvation method.....	35
3.4 Testing Procedure.....	37
3.4.1 Saturation	37
3.4.2 Consolidation	40
3.4.3 Undrained strain-controlled cyclic simple shear.....	41
3.4.4 Drained strain-controlled cyclic simple shear.....	41
4. DYNAMIC SIMPLE SHEAR TEST RESULTS	43
4.1 Undrained Cyclic Simple Shear Tests.....	43

4.1.1 Undrained strain-controlled tests on fully saturated sand samples.....	44
4.1.2 Undrained strain-controlled tests on partially saturated sand samples	47
4.2 Drained Cyclic Simple Shear Tests	50
4.2.1 Drained strain-controlled tests on fully saturated sand samples.....	50
4.2.2 Drained strain-controlled tests on partially saturated sand samples	53
4.3 Comparison of Undrained and Drained Strain-Controlled Tests in Partially Saturated Sand Samples	64
4.4 Estimation of Air-Water Bulk Modulus (K_{aw}).....	68
5. CONCLUSIONS AND RECOMMENDATIONS	71
REFERENCES	75
APPENDICES	79
APPENDIX A	80
CURRICULUM VITAE.....	85



ABBREVIATIONS

APC	: Automatic Pressure Controller
APCH	: Hydraulic Automatic Pressure Controller
ASTM	: American Society for Testing and Materials
CPR	: Cell Pressure Ramp
CSR	: Cyclic Stress Ratio
CSSLB	: Cyclic Simple Shear Liquefaction Box
DA	: Double Amplitude
DSS-C	: Dynamic Simple Shear Test Device with Confining Pressure
GCTS	: Geotechnical Consulting & Testing System
IPS	: Induced Partial Saturation
NGI	: Norwegian Geotechnical Institute
SGI	: Swedish Geotechnical Institute



SYMBOLS

d	: Depth
Dr	: Relative density
E_r	: Skeleton rebound modulus
K_{aw}	: Bulk modulus of air-water mixture
K_w	: Bulk modulus of water
K₀	: The coefficient of earth pressure at rest
N	: Number of cycles
n	: Porosity
N_L	: Number of cycles required to liquefaction
N_{max}	: Maximum number of cycles
P₀	: Atmospheric pressure
r_u	: Excess pore water pressure ratio
S	: Degree of saturation
u₀	: Pore water pressure
V	: Volume of soil
V_p	: P-wave velocity
W	: Weight of soil
ε_{vd}	: Volumetric strain
Δε_{vd}	: Change in volumetric strain in drained condition
Δu	: Excess pore water pressure
τ	: Shear stress
γ	: Shear strain
σ	: Vertical total stress
σ'_v	: Vertical effective stress
σ_d	: Deviator stress
ξ	: Damping ratio
G	: Shear modulus
G_{sec}	: Secant shear modulus
G_{max}	: Maximum shear modulus
ν	: Poisson ratio



LIST OF TABLES

	<u>Page</u>
Table 2.1 : Desaturation techniques as a liquefaction mitigation method.	24
Table 3.1 : The properties of Şile sand.	29
Table 3.2 : The measurements of a fully saturated sample.	32
Table 3.3 : The measurements of a partially saturated sample.	36
Table 3.4 : The measurements of a partially saturated sample during saturation.	40
Table 3.5 : The measurements of a partially saturated sample during consolidation.	41
Table 4.1 : The undrained strain-controlled tests of fully saturated sand samples.	44
Table 4.2 : The undrained strain-controlled tests of partially saturated sand samples.	48
Table 4.3 : The unreliable results of fully saturated sand samples obtained prior to using the recommended software settings.	51
Table 4.4 : The unreliable results of partially saturated sand samples obtained prior to using the recommended software settings.	53
Table 4.5 : The test results of partially saturated sand samples obtained after using the recommended $K_0=1.0$ and calibration settings.	56
Table 4.6 : Shear modulus values.	63
Table 4.7 : The comparison of K_{aw} values.	70
Table A.1 : Fully saturated specimen preparation calculations.	80
Table A.2 : Fully saturated specimen saturation and consolidation calculations.	81
Table A.3 : Partially saturated specimen preparation calculations.	82
Table A.4 : Partially saturated specimen saturation and consolidation calculations.	83
Table A.5 : Calculations of horizontal deformation.	84



LIST OF FIGURES

	<u>Page</u>
Figure 2.1 : Niteko earth dams complex, after the Kobe earthquake (1995). (Courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley).	4
Figure 2.2 : Transfer of state of soil skeleton during liquefaction process (Ishihara et al., 2004).	4
Figure 2.3 : Excess pore pressure ratio for different degrees of saturation and relative densities at 0.1 % shear strain (Eseller-Bayat et al., 2013).	5
Figure 2.4 : Zone of liquefaction (Kramer, 2016).	7
Figure 2.5 : The results of a simple shear test with $D_r=54\%$, $\sigma'_v=95$ kPa and $CSR=0.20$ (Wu, 2002).	8
Figure 2.6 : The results of a simple shear test with $D_r=77\%$, $\sigma'_v=77$ kPa and $CSR=0.36$ (Wu, 2002).	9
Figure 2.7 : Relationship between CSR causing liquefaction and $(N_1)_{60}$ (Idriss and Boulanger, 2010).	10
Figure 2.8 : Relationship between CSR causing liquefaction and CPT Tip Resistance (Boulanger and Idriss, 2016).	10
Figure 2.9 : Pore pressure ratio vs. shear strain after 10 loading cycles at a strain-controlled cyclic triaxial test with different sample preparation types (Dobry and Ladd, 1980).	12
Figure 2.10 : Volumetric strains at different shear strain amplitudes (Martin et al., 1975).	13
Figure 2.11 : Hysteresis loop and volumetric strain vs. shear strain under 100 kPa confining pressure, at 0.998 % shear strain amplitude (Kang, 2016)	15
Figure 2.12 : Volumetric strain vs. number of cycles of dense specimens at different cyclic shear strain amplitudes under confining pressure of 100 kPa (Kang, 2016).	15
Figure 2.13 : Normalized stiffness degradation curve (modified after, Atkinson and Sallfors (1991) and Mair (1993)).	16
Figure 2.14 : Secant and tangent shear modulus (Kramer, 2016).	17
Figure 2.15 : Hysteresis loop (Hayashi et al., 2018).	18
Figure 2.16 : Triaxial apparatus (ASTM D5311/D5311M-13).	19
Figure 2.17 : B-check vs. degree of saturation in partially saturated sands (Kamata et al., 2009).	20
Figure 2.18 : SGI- type simple shear apparatus (Kjellman, 1951).	21
Figure 2.19 : NGI-type simple shear apparatus (Bjerrum and Landva, 1966).	21
Figure 2.20 : Basic definitions of the simple shear apparatus (Corte et al., 2017).	22
Figure 2.21 : Cyclic torsional test apparatus (Ishihara, 1996).	22
Figure 2.22 : The change of the effective stress and pore water pressure (Ishihara et al., 2004).	23
Figure 2.23 : Volumetric strains vs. pore water pressure ratios with degrees of saturation (He et al., 2013).	25

Figure 2.24 : Excess pore pressure generations (r_u) vs. number of cycles in fully and partially saturated sand specimens at $\sigma'_v=100$ kPa and $\gamma=0.2$ % (Gülen and Eseller-Bayat, 2017).....	25
Figure 2.25 : Excess pore water pressure generations vs. different shear strain in fully and partially saturated sand specimens at $\sigma'_v=100$ kPa (Gülen, 2017).	26
Figure 3.1 : Dynamic simple shear with confining pressure test device (VJ-Tech Manual, 2016).....	28
Figure 3.2 : DSS-C test device at ITU Geotechnical Engineering Laboratory.....	29
Figure 3.3 : Grain size distribution of Şile Sand.	30
Figure 3.4 : (a)O-ring tool, (b) split mold and (c)base pedestal of the test device. ..	31
Figure 3.5 : Correlation between degree of saturation and time for different R values within 24 hours.	34
Figure 3.6 : Correlation between the sodium percarbonate to sand ratios versus the degrees of saturation.	34
Figure 3.7 : Specimens prepared to obtain degrees of saturation in terms of different ratios R.	35
Figure 3.8 : 24 hours after the prepared partially saturated sand sample.	36
Figure 3.9 : B-check 2 stage of a partially saturated test.....	37
Figure 3.10 : Partially saturated samples pictures during saturation stages.	38
Figure 3.11 : The reference length for measurement of the sample diameter.	39
Figure 3.12 : The variation between B-check 2 and degree of saturation.	39
Figure 4.1 : The typical results of undrained strain-controlled tests in a fully saturated sample (a) r_u vs. N, (b) γ vs N, (c)Shear stress vs N and (d)Shear stress-shear strain hysteresis loop.....	45
Figure 4.2 : The undrained strain-controlled tests in fully saturated sample under 50 kPa and 100 kPa vertical effective stresses at (a) shear strain amplitude of 0.3 %, (b) shear strain amplitude of 0.35 %.....	46
Figure 4.3 : The excess pore pressure ratio versus number of cycles at different relative densities.	47
Figure 4.4 : The typical results of undrained strain-controlled tests in a partially saturated sample (a) r_u vs. N, (b) γ vs N, (c)Shear stress vs. N and (d)Shear stress-shear strain hysteresis loop.....	49
Figure 4.5 : The result of a drained strain-controlled test on fully saturated sand specimen without the recommended $K_0=1.0$ setting.	51
Figure 4.6 : Samples that collapsed during the experiment with the recommended $K_0=1.0$ setting.	52
Figure 4.7 : Sudden changes of vertical effective stress in drained strain-controlled test on fully saturated sand specimen with the recommended $K_0=1.0$ setting.	52
Figure 4.8 : The result of a drained strain-controlled test on partially saturated sand specimen without the recommended software setting.	54
Figure 4.9 : Sudden changes of vertical effective stress in isotropic consolidated partially saturated sand specimen during drained strain-controlled test.	55
Figure 4.10 : The sensitivity calibration set up from 4 to 1 before the test.....	55
Figure 4.11 : Drained strain-controlled tests for $D_r=35$ %, $S=90-80-75$ % under shear strain amplitude of (a) 0.4 %, (b) 0.3 % and (c) 0.2 %.....	58
Figure 4.12 : Drained strain-controlled tests for $D_r=45$ %, $S=90-80-75$ % under shear strain amplitude of (a) 0.4 % , (b) 0.3 % and (c) 0.2 %.....	59

Figure 4.13 : Shear stress and volumetric strains related to cyclic shear strains with 0.4 % amplitude in a sample 90 % degree of saturation.	60
Figure 4.14 : Shear stress and volumetric strains related to cyclic shear strains with 0.3 % amplitude in a sample with 90 % degree of saturation.....	61
Figure 4.15 : Shear stress and volumetric strains related to cyclic shear strains with 0.2 % amplitude in a sample with for 90 % degree of saturation.	62
Figure 4.16 : Secant shear modulus behavior for the specimen with S=90 % at different shear strain amplitudes.	63
Figure 4.17 : The shear modulus degradation curves.	64
Figure 4.18 : Comparison of r_u and ε_{vd} under $\sigma'_v=50$ kPa, $\gamma=0.4$ % and $Dr=30-35$ % at different range of degrees of saturation.	65
Figure 4.19 : Comparison of r_u and ε_{vd} under $\sigma'_v=50$ kPa, $\gamma=0.3$ % and $Dr=30-35$ % at different range of degrees of saturation.	66
Figure 4.20 : Comparison of r_u and ε_{vd} under $\sigma'_v=50$ kPa, $\gamma=0.2$ % and $Dr=30-35$ % at different range of degrees of saturation.	67
Figure 4.21 : Volumetric strains vs. excess pore water pressure ratios.	68
Figure 4.22 : Bulk modulus of air-water mixture related to degree of saturation.	69
Figure A.1 : Partially saturated samples pictures before and after B-check 1	84



DETERMINATION OF VOLUMETRIC STRAINS IN FULLY AND INDUCED PARTIALLY SATURATED SANDS THROUGH DYNAMIC SIMPLE SHEAR TEST DEVICE WITH CONFINING PRESSURE

SUMMARY

The concept of liquefaction, which is one of the most common research topics in geotechnical engineering, is the loss of vertical effective stress of the soil due to the excess pore water pressure of loose and saturated sands under earthquake loads. The induced excess pore water pressure cannot dissipate due to effect of short-term cyclic loads. After a while, liquefaction occurs at the point where the excess pore water pressure is equal to the vertical effective stress. The liquefaction can cause settlements and loss of bearing capacity. The effects of liquefaction were firstly observed in the 1964 Alaska and Niigata earthquakes that caused extensive structural damage and loss of lives. In the literature, studies focused on saturated loose sands and various methods proposed against liquefaction.

Many techniques were investigated as a method of soil improvement against liquefaction. IPS, Induced Partial Saturation was one of these newly proposed techniques. The preliminary researches about IPS method was proposed by Eseller (2004) and Yegian et al. (2007). The objective of Induced Partial Saturation (IPS) method, was to reduce the saturation by entrapping air/gas bubbles in the soil to achieve partial saturation. The major advantage of IPS is that it can also be applied under existing structures. The studies published in the literature showed no clear findings on how the degree of saturation affected volumetric strains. It was important to evaluate the volumetric strains developed in partially saturated sands during earthquakes in terms of providing knowledge about the use of IPS as a mitigation technique. Within the scope of the thesis, VJ-Tech Dynamic Simple Shear Test Device with Confining Pressure was used to investigate the dynamic behavior of partially and fully saturated sand specimens. The testing program included; saturation, isotropic consolidation, undrained and drained strain-controlled cyclic simple shear stages.

In this study, Şile sand was used for fully and partially saturated sand samples. Moist tamping method was preferred to obtain loose specimens on fully saturated specimens whereas the partially saturated specimens were prepared with IPS technique. In the IPS, sodium percarbonate was selected to generate air/gas bubbles in the sample. In fully and partially saturated sand specimens, excess pore water pressure ratio (r_u) was investigated at different shear strain amplitudes under undrained cyclic simple shear tests. During the undrained cyclic simple shear stage, unlike the behavior in fully saturated specimens, the excess pore water pressure (r_u) in partially saturated specimens did not reach 1.0 with the increasing number of cycles.

After undrained cyclic simple shear tests, the drained cyclic tests were performed on fully and partially saturated sand samples. Some problems faced during the tests and a software setting was recommended by VJ-Tech. Although the recommended setting were applied, it did not work on fully saturated sand samples. The recommended

settings worked fine for partially saturated sand specimens. The isotropic consolidation ($K_0 = 1.0$) performed prior to drained cyclic simple shear tests.

The relationship between volumetric strains, degrees of saturation, relative densities and shear strain amplitudes were investigated by drained cyclic simple shear tests. The volumetric strains decreased with; decreasing degree of saturation, increasing relative density and decreasing shear strain amplitude. Secant shear modulus was evaluated and obtained from the shear stress-shear strain hysteresis loop. In drained cyclic simple shear tests, hardening behavior was observed on partially saturated specimens. The volumetric strains were also compared with Finn et al. (1976) and Byrne (1991) correlations. Experimental results and empirical correlations followed similar trends in samples with low to high degrees of saturation. The presented results showed that the volumetric strains depended on the degree of saturation.

The bulk modulus of air-water mixture (K_{aw}) was estimated by using Martin et al. (1975) effective stress model. The calculated K_{aw} values were compared with those obtained from the numerical model based predictions Seyed-Viand and Eseller-Bayat (2020) and Koning (1963) equation. It can be seen that the experimental-based and numerical-based estimations were in a good agreement unlike Koning (1963) predictions.

Finally, the findings in this study will provide data and knowledge base for the future research to evaluate the post-liquefaction deformations in specimens mitigated by Induced Partial Saturation (IPS).

TAM DOYGUN VE KISMİ DOYGUNLUĞA İNDİRGENMİŞ KUMLARIN HÜCRE BASINÇLI DİNAMİK BASİT KESME DENEYLERİ İLE HACİMSEL DEFORMASYONUN TAYİN EDİLMESİ

ÖZET

Geoteknik mühendisliğinde en yaygın araştırma konularından biri olan zeminlerin sıvılaşması, deprem yükleri altında gevşek ve suya tam doygun kumların ve siltlerin, oluşan aşırı boşluk suyu basıncının etkisiyle, zeminin düşey efektif gerilmesinin düşmesi ve mukavemetini kaybetmesi durumudur. Deprem süresince oluşan kısa süreli çevrimsel yüklerin etkisiyle su drene olamaz ve bu durum aşırı boşluk suyu basıncının artmasına sebep olur. Toplam gerilmenin değişmediği bu durumda oluşan aşırı boşluk suyu basıncının etkisiyle daneler birbirinden uzaklaşmaya başlar. Daneler arası temasın kaybolmasıyla düşey efektif gerilmedeki azalma bir süre sonra oluşan aşırı boşluk suyu basıncına eşit olur ve aşırı boşluk suyu basıncının deprem etkisine maruz kalmadan önceki düşey efektif gerilmeye eşit olduğu noktada sıvılaşma meydana gelir. Sıvılaşma sonrası zeminde oturmalar ve taşıma gücünde azalmalar gerçekleşir. Bu etkiler ilk olarak yapısal hasarların ve can kayıplarının yaşandığı 1964 Alaska ve Niigata depremlerinde gözlemlenmiştir. Literatür çalışmalarında bu doğrultuda suya doygun ve gevşek kumlar üzerine yoğunlaşmış ve sıvılaşmaya karşı zeminde direnç oluşturabilecek çeşitli yöntemler araştırılmıştır. Bu yöntemlerden biri de son yıllarda geliştirilen Kısmi Doygunluğa İndirgeme (Induced Partial Saturation, IPS) yöntemidir (Yegian et al. 2007, Eseller-Bayat, et al. 2013 (a) ve (b)).

Yapılan araştırmalar sonucunda kısmi doygun kumların sıvılaşmaya karşı tam doygun kumlarla kıyaslandığında daha dayanımlı olduğu gözlemlenmiştir. Tekrarlı çevrimsel yükler altında kısmi doygun kumlarda boşluk suyu basıncındaki değişim ve bu değişimin doygunluk derecesiyle olan ilişkisi hakkında çalışmalar gerçekleştirilmiş (Eseller-Bayat, et al. 2013 (a) ve (b), Eseller-Bayat and Gülen 2020) ve araştırmalar halen devam etmektedir.

Sıvılaşmaya karşı zemin iyileştirme metodu olarak birçok teknik araştırılmaktadır. Son yıllarda kısmi doygun kumların sıvılaşmaya karşı direnci göz önünde bulundurularak geliştirilen yöntemlerden Kısmi Doygunluğa İndirgeme (IPS) metodundaki amaç, zemin içerisindeki boşluklarda hava kabarcıkları oluşturarak zeminin doygunluk derecesinin düşürülmesi ve kısmi doygunluğun sağlanmasıdır. Kısmi Doygunluğa İndirgeme yönteminin en büyük avantajı mevcut yapıların altında da uygulanabilir olmasıdır.

Çevrimsel yükleme sonrasında kısmi doygun kumlarda meydana gelecek olan hacimsel deformasyonların tam doygun kumlarda oluşacak deformasyonlarla kıyaslanabilmesi IPS yönteminin zemin iyileştirme yöntemi olarak kullanılması hakkında bilgi vermesi açısından önemlidir. Literatür çalışmaları incelendiğinde hacimsel deformasyonların tam doygun kumlar üzerine yoğunlaştığı ve doygunluk derecesinin hacimsel deformasyonlar üzerindeki etkisinde çalışmaların devam ettiği görülmektedir. Bu kapsamda yapılan bu tez çalışmasıyla, Hücre Basıncı Dinamik

Basit Kesme Test Sistemi kullanılarak IPS yöntemiyle kısmi doygunluğa indirgenmiş kumların ve tam doygun kumların hacimsel deformasyonlarının belirlenmesi amaçlanmaktadır. Bu doğrultuda; kısmi ve tam doygun kum numunelerinin dinamik davranışlarını derinliğe bağlı olarak inceleme amacıyla VJ-Tech marka Hücre Basıncılı Dinamik Basit Kesme Test Sistemi ve Clisp Studio yazılımı kullanılmıştır.

Çalışmanın ilk aşamasında tam doygun numunelerin drenajsız deformasyon kontrollü çevrimsel yükleme altında farklı birim kayma deformasyon değerlerinde aşırı boşluk suyu basıncı oranı incelenmiştir. Tam doygun numunelerde, gevşek ve doygun numuneler elde etmek amacıyla ıslak tokmaklama yöntemi tercih edilmiştir. Deneylere ilk olarak doygunluk aşamasıyla başlanmış olup, numunenin içinde kalan havayı tamamen dışarıya atıp tam doygun hale getirebilmek amacıyla yüksek değerlerde hücre basıncı ve ters basınç uygulanmıştır. Numunenin doygunluk derecesi yapılan B değeri (Skempton katsayısı) kontrolü ile sağlanmış olup, B değeri en az 0.95 olacak şekilde hücre ve ters basınçlar artırılarak numune doygun hale getirilmiştir. Doygunluk aşamasından sonra sıvılaşma deneyine geçmeden önce numune 50 kPa ve 100 kPa efektif gerilme altında izotropik olarak konsolide edilmiştir. Belli bir efektif gerilme altında konsolide edilen numuneler deformasyon kontrollü sinusoidal dalga formuna sahip çevrimsel yüklemeye maruz bırakılmıştır. Bu aşamada gerilme-deformasyon eğrisi ve aşırı boşluk suyu basıncı oranları incelenmiştir. Tam doygun numunelerde aşırı boşluk suyu basıncı oranı 1.0'e ulaşmış olup, efektif gerilme ve çevrim sayısına bağlı ilişkiler irdelenmiştir.

Tam doygun numunelerin drenajsız çevrimsel yüklemelerinin tamamlanmasından sonra 50 kPa efektif gerilme ve tekrarlı birim deformasyonlar altında kısmi doygun numunelerin deneyleri gerçekleştirilmiştir. Kısmi doygun numuneler kısmi doygun indirgeme (IPS) yöntemi ile hazırlanmıştır. IPS yönteminde, numune içinde hava kabarcıkları oluşturmaları için sodyum perkarbonat kimyasal maddesi seçilmiştir. İlk olarak kum ağırlığına bağlı, farklı oranlarda sodyum perkarbonat kullanılarak cam beherlerde ıslak yağmurlama yöntemi ile deneme numuneleri hazırlanmıştır. Numunelerde kimyasal reaksiyonun 8 saatten sonra yavaşladığı ve 24 saat sonra tamamen durduğu belirlenmiştir. Kum ve kimyasal ağırlığını belirlemek için yapılan bu denemelerde doygunluk derecesine ve sodyum perkarbonat ağırlığının kum ağırlığına oranına bağlı bir korelasyon elde edilmiştir. Bulunan korelasyon sonucunda 0.15'den küçük oranlarda numunenin doygunluk derecesinde ciddi bir azalma görülmemiştir. 0.30'dan büyük oranlarda ise numune kısmi doygun sınırından doygun olmayan derecelere düşmüş ve boşluklarda oluşan büyük hava kabarcıklarından dolayı homojen yapısını kaybetmiştir. 0.15 ve 0.30 aralığında ise numunede küçük hava kabarcıkları homojen olarak dağılmış olup, bu tepkime sonucunda başlangıçta sıkı durumda olan numuneler gevşek duruma geçmiştir. Deneylerde hedeflenen doygunluk derecelerine ulaşabilmek için 0.15-0.30 aralığına sahip oranlarda numuneler kurulmuştur. Hazırlanan numuneler tam doygun numunelerde olduğu gibi doygunluk, konsolidasyon ve sıvılaşma aşamalarında incelenmiştir. Numuneler 50 kPa efektif gerilme altında izotropik olarak konsolide edilmiştir. Sıvılaşma esnasında çevrim sayısına bağlı olarak aşırı boşluk suyu basıncındaki artış gözlemlenmiştir. Aşırı boşluk suyu basıncı oranı tam doygun kumlarda gözlemlenen davranıştan farklı olarak, artan çevrim sayısına rağmen 1.0'e ulaşmamıştır.

Tez kapsamında drenajsız deformasyon kontrollü tam doygun ve kısmi doygun numunelere ait deneylerin tamamlanmasından sonra benzer özellik ve parametrelere sahip tam doygun ve kısmi doygun numunelerde drenajlı deformasyon kontrollü deneyler gerçekleştirilmiştir. Çalışmanın başlangıcında çevrimsel yükleme sonrası

oluşan aşırı boşluk suyu basıncının drene olmasıyla zeminde oluşacak hacimsel deformasyonu tayin edilmek istenmiş ancak deney aletinde çevrimsel yükleme sonrası drenaj aşaması yapılamadığı için, deney planı drenajlı çevrimsel yükleme esnasında oluşacak hacimsel deformasyonu bulmaya yönelik olmuştur. İlk olarak tam doygun kum numunelerde deneyler yapılmaya başlanmıştır. Deney prosedüründe belirtildiği üzere düşey efektif gerilmenin deney süresince sabit kalmadığı ve gittikçe azalmaya başladığı gözlemlenmiştir. Kısmi doygun kum numunelerde de aynı davranış gözlemlendiği için elde edilen değerlerin güvenilir olmadığı düşünülerek, Clisp programında efektif düşey gerilmeyi sabit tutabilmek için izotropik konsolidasyondan sonra VJ-Tech firması tarafından önerilen kısa süreli anizotropik konsolidasyon ayarı aşaması yapılmıştır. Ancak anizotropik ayarında hücre içindeki ortam koşullarını değiştirmemek için $K_0=1.0$ olarak alınmıştır. Yapılan bu ayarlardan sonra drenajlı çevrimsel yükleme aşamasında ise tam doygun kum numunelerde deney başlangıcında, kısmi doygun kum numunelerde ise deney tamamlanmadan numunede göçme meydana gelmiştir. Bu soruna çözüm olarak Clisp programında düşey yükü kontrol eden bölge için kalibrasyonda değişiklikler yapılmıştır. Yapılan değişiklik sonucu kısmi doygun numunelerde drenajlı çevrimsel yükleme aşaması yapılabilirken, tam doygun numunelerde bu uygulama bir çözüm olmamıştır. Kısmi doygun numunelerde hücre içerisindeki izotropik durumu değiştirmeyecek şekilde $K_0=1.0$ anizotropik ayarı yapılarak ve 1.0 Hz frekansa sahip tekrarlı birim deformasyonlar altında drenajlı çevrimsel yükleme deneyleri yapılmıştır. Farklı doygunluk derecesinde hazırlanan numunelerde doygunluk derecesi, sıklık derecesi ve kayma birim deformasyonların hacimsel birim deformasyonlar üzerindeki etkileşimi incelenmiştir.

Çevrimsel yükleme altında drenajlı deformasyon kontrollü deneylerde numunelerde genel olarak gevşek kum davranışı gözlemlenmiş ve hacimsel olarak sıkışma meydana gelmiştir. Artan doygunluk derecesiyle birlikte hacimsel deformasyonlarda artış görülürken, sıklık derecesi arttıkça hacimsel deformasyonda azalma görülmüştür. Artan çevrim sayısı ile kayma gerilmelerindeki artışa bağlı olarak numunelerde pekleşme davranışı gözlemlenmiştir. Kısmi doygun kum numuneye ait sekant kayma gerilmesi degradasyon eğrisi elde edilmiştir. Yapılan drenajsız deneyler sonucu elde edilen aşırı boşluk suyu basıncı oranı ile drenajlı deneyler sonucunda bulunan hacimsel deformasyonlar kıyaslanmış olup, yüksek aşırı boşluk suyu basıncı oranlarında daha fazla hacimsel deformasyonların olduğu görülmüştür. Hacimsel deformasyonların tayininden sonra, drenajsız deneylerden elde edilen boşluk suyu basıncındaki değişim ve drenajlı deneylerden elde edilen hacimsel deformasyondaki değişimden faydalanılarak Martin et al. (1975) tarafından önerilen efektif gerilme modeli ile su-hava karışımına ait hacimsel (sıkışma) modülü (K_{aw}) deneysel olarak tahmin edilmiştir. Bulunan sonuçlarda hacimsel deformasyon modülünün azalan doygunluk derecesiyle birlikte azaldığı gözlemlenmiştir. Elde edilen K_{aw} değerleri, hacimsel modülü doygunluk derecesine bağlı olarak tahmin eden Koning (1963) denklemi ve Seyedi-Viand and Eseller-Bayat (2020) nümerik sonuçları ile kıyaslanmıştır. Koning denkleminden hesaplanan değerlerin deneysel olarak elde edilen K_{aw} değerlerinden daha düşük geldiği belirlenirken, nümerik sonuçlar ile deneysel sonuçların uyumlu olduğu görülmüştür. Koning denklemi ile deneysel sonuçların birbiriyle uyuşmaması hacim modülü için yeni bir formülasyon geliştirilebileceği sonucunu çıkartmıştır. Nümerik analizlerle deneysel çalışmaların tutarlı bir şekilde değerlendirilebilmesi için doygunluk derecesine göre K_{aw} parametresi önemli bir rol oynamaktadır. Kısmi doygun kumların boşluklarında yer alan su-hava karışımının hacim modülünün doygunluk derecesine göre belirlenmesi, nümerik analizlerde kısmi doygun kumların

evrimsel ykleme altındaki davranışının gereki olarak tayin edilmesi aısından oldukça nemlidir.

Yapılan alıřmalar sonrasında drenajlı evrimsel yklemeler altında artan evrim sayısına baėlı olarak kısmi doygun numunelerde hacim azalması eėilimi grlmřtr. Hacimsel sıkıřmaya baėlı olarak numunelerde pekleřme meydana gelmiřtir. Gzlemlenen hacimsel deformasyonların birim kayma deformasyonundaki artıřla birlikte arttıėı belirlenmiřtir. Ayrıca numunelerin sıkılık derecesi de hacimsel deformasyonları etkileyen bir bařka nemli parametredir. Kısmi doygun kumlarda doygunluk derecesinin azalmasıyla daha dřk deėerlerde hacimsel deformasyonlar meydana gelmiřtir. Elde edilen bulgular, kısmi doygunluėa indirgeme (IPS) yntemiyle hazırlanan numunelerin evrimsel yklemeler sonrası meydana gelecek deformasyonlarının incelenmesi aısından gelecek literatr alıřmalarına katkı saėlayacaktır.



1. INTRODUCTION

Soil liquefaction is a phenomenon in which soil substantially loses its strength due to reduced vertical effective stress. There have been comprehensive studies to understand the mechanism underlying the destructive effects of liquefaction, as first seen in the 1964 Good Friday earthquake in Alaska, and then in the Niigata earthquake in Japan.

Liquefaction potential in the field can be measured via standard penetration, cone penetration, and seismic field tests. Additionally, various analysis methods have developed to determine the liquefaction potential of soil *in-situ*. These methods are highly remarkable in experimental studies in the laboratory for precise determination of dynamic soil parameters.

Studies have mainly focused on the liquefaction resistance of sands under cyclic loading at undrained conditions. However, cyclic deformation characteristics under drained conditions have not been investigated adequately. In this study, a dynamic simple shear test device with confining pressure (DSS-C) was used to predict volumetric strains of fully and partially saturated sand samples. Since the sample preparation method is essential on the cyclic strength of the reconstituted specimens, two sample preparation methods were selected in the tests. Depending on the desired relative density and degree of saturation, either moist tamping or wet pluviation method was used. The fully saturated and partially saturated sand samples were prepared by moist tamping and wet pluviation techniques, respectively.

The first part of the study included undrained strain-controlled tests on fully and partially saturated samples. The tests were performed at various shear strains ranging from 0.2 % to 0.4 % under two different initial vertical effective stresses. Excess pore water pressure generation was measured and evaluated in terms of the number of cycles. The evaluation was made for a range of degrees of saturation, relative densities, and shear strain amplitudes.

The second part of the study was aimed to obtain volumetric strains under drained strain-controlled tests on fully and partially saturated samples. Due to some problems

encountered in the fully saturated sand experiments, tests were continued with partially saturated sand samples. The tests were performed at shear strain levels ranging from 0.2 % to 0.4 % and the samples were consolidated under 50 kPa vertical effective stress. The effect of shear strain amplitudes and relative densities were also considered in drained tests. Drained and undrained strain-controlled test results were compared to investigate the relationship between volumetric strains and excess pore water pressure ratios. The shear modulus degradation curves evaluated in terms of number of cycles. The bulk modulus of the air-water mixture was estimated by benefit from literature.

In Chapter 1, the aim of the study and general information was given. In Chapter 2, literature studies about liquefaction were presented. Liquefaction types and evaluation approaches were mentioned, and laboratory and field experiments conducted to summarize the liquefaction potential. In Chapter 3, the DSS-C test device and testing procedure were explained and the sample preparation methods were discussed in detail. The problems encountered during the tests were mentioned along with the solutions developed. In Chapter 4, the drained and undrained strain-controlled test results of fully and partially saturated sand samples were presented. The drained shear stress-shear strain behavior and shear modulus degradation of partially saturated sands were studied. Also, the excess pore water pressures generated in partially saturated sands tested under undrained conditions were compared with volumetric strains developed in drained tests. The air-water bulk modulus in partially saturated sands was estimated using the experimental results and Martin et al. (1975) and Byrne (1991) analytical correlations. In the last chapter, all the experimental study findings were presented with an emphasis on the important points.

2. REVIEW OF LIQUEFACTION STUDIES

2.1 Liquefaction Definition

Liquefaction of saturated sands and silts is one of the major problems in geotechnical engineering. In 1964, Good Friday earthquake in Alaska ($M_w=9.2$) and Niigata earthquake in Japan ($M_w=7.5$) had devastating effects due to liquefaction. The other tremendous destructive Kobe earthquake in 1995 had also liquefaction-induced damages which can be seen in Figure 2.1. Many researchers focused on the factors that affect the liquefaction resistance after these major earthquakes.

During an earthquake the saturated cohesionless soils tend to densify. However, since the duration of cyclic stress application is so short as compared to the time required for drainage of water, the volume contraction cannot occur immediately. As shown in Figure 2.2, in order to keep the contracting loose sand at a constant volume, pore water pressure increases and vertical effective stress decreases. When the pore water pressure generation equal to the initially existing confining pressure, liquefaction occurs. After a while, excess pore water can be drained and settlements observed in the soil. The excess pore water pressure ratio is equal to 1.0 ($r_u=1.0$) is the main criteria for liquefaction. Initial liquefaction can be explained below ;

$$\sigma'_0 = \sigma - u_0 \quad (2.1)$$

where σ and u_0 are total stress and initial pore water pressure. The total vertical stress remains constant during earthquake loading. Due to the excess pore pressure generation Δu , final vertical effective stress is calculated by Eqs 2.2 and 2.3.

$$\sigma'_{\text{final}} = \sigma - (u_0 + \Delta u) = 0 \quad (2.2)$$

$$\sigma'_{\text{final}} = \sigma'_{v0} - \Delta u = 0 \quad (2.3)$$

Initial liquefaction occurs when

$$\sigma'_{\text{final}} = \sigma'_{v0} - (1.0 - r_u) = 0 \quad (2.4)$$

$$r_u = \Delta u / \sigma'_{v0} = 1 \quad (2.5)$$

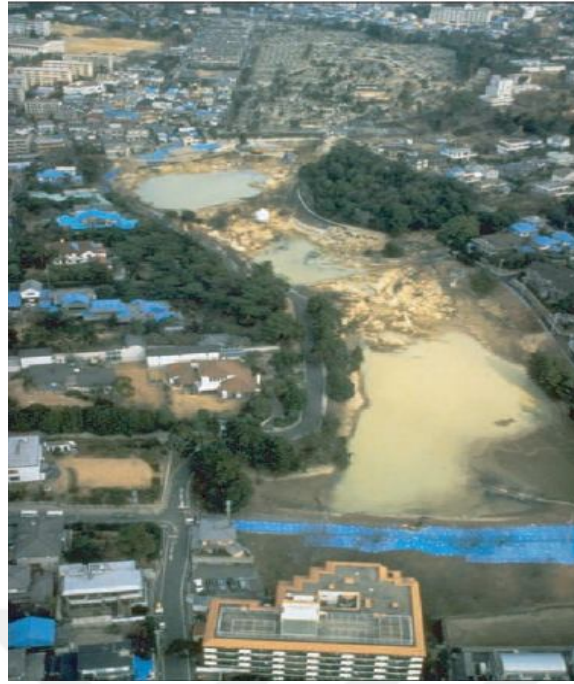


Figure 2.1 : Niteko earth dams complex, after the Kobe earthquake (1995).
(Courtesy of the National Information Service for Earthquake Engineering, EERC,
University of California, Berkeley).

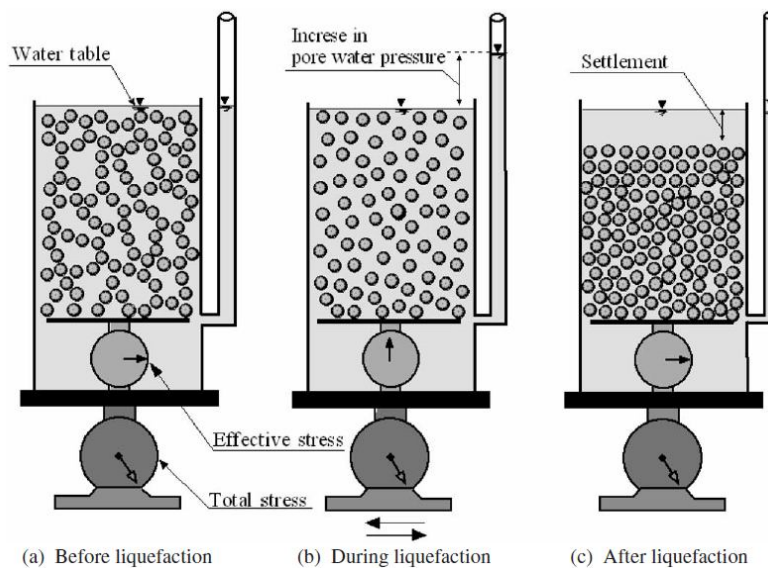


Figure 2.2 : Transfer of state of soil skeleton during liquefaction process (Ishihara et al., 2004).

Previous studies focused on the investigation of the noncohesive and fully saturated soils. Recent studies show that the partially saturated coarse-grained soil does not liquefy due to dynamic forces. However, even excess pore pressure ratio does not reach 1.0, the soil loses its strength with decreasing in vertical effective stress. Eseller-Bayat et al. (2013), performed cyclic shear strain tests in cyclic simple shear

liquefaction box (CSSLB) with partially saturated sands. The study showed that the excess pore pressure generation depends on degrees of saturation, relative densities, shear strain amplitudes and number of cycling loading. The excess pore pressure ratio vs. degree of saturation of the tested soil was displayed with different ranges of relative density. As shown in Figure 2.3, the degree of saturation was a major effect on maximum excess pore water pressure ratio. Furthermore, the maximum excess pore water pressure ratio was lower in relatively denser samples. As the soil in loose condition or the degree of saturation increased, the maximum excess pore water pressure ratio increased.

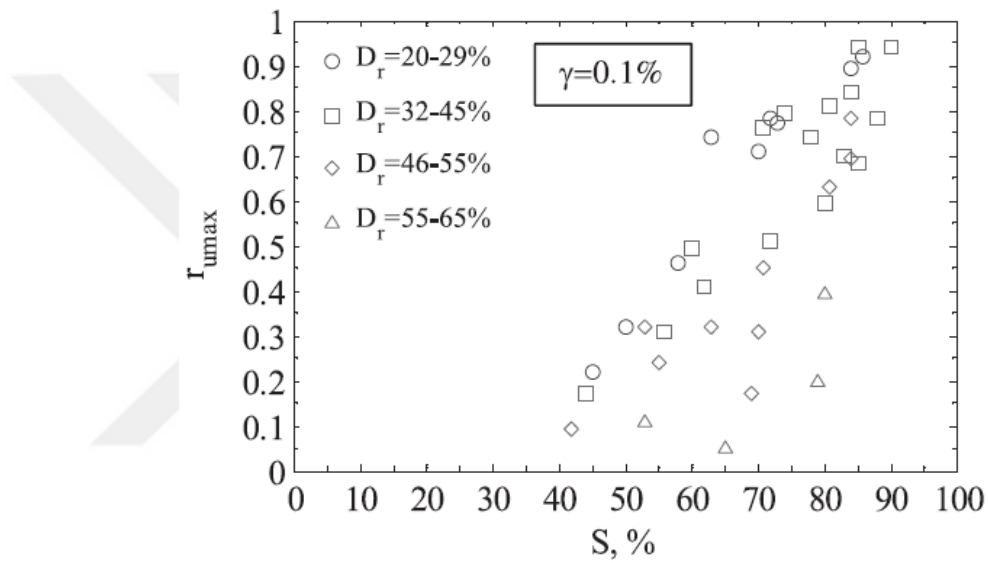


Figure 2.3 : Excess pore pressure ratio for different degrees of saturation and relative densities at 0.1 % shear strain (Eseller-Bayat et al., 2013).

In 1984, Ishihara proposed that nonplastic silts also have a significant effect on the liquefaction potential. In other words, plasticity properties of the soil are more important than the grain size. Liquefaction behavior of fine-grained soils has been studied by many researchers (Polito and Martin, 2001; Sancio, 2002; Hazırbaba, 2005; Idriss and Boulanger, 2008). Based on the data from sites where liquefaction was observed after earthquakes in China, some criteria established that any clayey soil containing less than 15% to 20% particles by weight smaller than 5 mm and having a water content to liquid limit ratio greater than 0.9 is susceptible to liquefaction. These are known as Chinese criteria and it has been modified by many researchers (Gratchev et al., 2006; Prakash and Puri, 2010).

2.1.1 Flow liquefaction

Under static or dynamic loading, when the static shear stress of the soil is greater than the shear strength of the liquefied soil, static equilibrium is destroyed and flow liquefaction can occur. These static shear stresses lead to large deformations which are called flow failures by bringing the soil to an unstable state. Flow liquefaction failures are characterized by the speed with which they develop and liquefied soils which are moved to a large distance (Kramer, 2016).

2.1.2 Cyclic mobility

Cyclic mobility is a phenomenon that can produce large deformations due to earthquake shaking. Cyclic mobility occurs when the static shear stress is less than the shear strength of the soil in its liquefied state. In contrast to flow liquefaction, the deformations are generated by static and cyclic shear stresses. Generally, these deformations are described as lateral spreading (Seed et al., 2003). When the fine content is higher in the sandy soil, it is more vulnerable to cyclic mobility.

2.2 Evaluation of Liquefaction

Several approaches were developed to understand the liquefaction potential. The evaluation methods are divided into two categories: cyclic stress-based and cyclic strain-based approaches. An alternative approach was developed to analyze drained and undrained conditions during cyclic loading. The effective stress model provides empirical relations between pore pressure generation and volumetric strain.

2.2.1 Cyclic stress approach

The description of the cyclic stress approach is the comparison of liquefaction resistance with the cyclic simple shear stress. Zone of liquefaction in terms of the depth which is shown in Figure 2.4. As shown in the figure, firstly the number of equivalent cycles should be determined using laboratory tests. The cyclic shear stress required to cause liquefaction is obtained in terms of depth. When the earthquake-induced loading exceeds the resistance, liquefaction is expected to occur. Liquefaction potential can be obtained from the factor of safety against liquefaction. The factor of safety is the ratio of cyclic shear stress required to cause liquefaction to equivalent cyclic shear stress induced by earthquake.

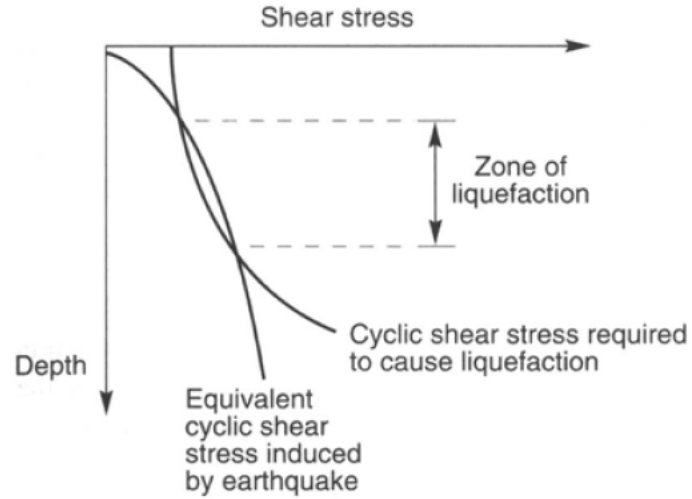


Figure 2.4 : Zone of liquefaction (Kramer, 2016).

The earthquake-induced shear stresses can be determined from the peak ground acceleration expected at the site or from site response analysis. The non-uniform stress cycles which are obtained from earthquake motion are converted to uniform cycles for liquefaction analysis. Since the earthquake motion cannot be generated by a maximum acceleration at any moment, uniform cycles are assumed to be 65 % of the maximum shear stress that is shown in Eqn. 2.6.

$$\tau_{cyc} = 0.65 \tau_{max} \quad (2.6)$$

where τ_{cyc} is amplitude of uniform cyclic shear stress, τ_{max} is maximum shear stress of ground motion. Seed and Idriss (1971) also presented a simplified equation that depends on vertical stress and depth. The reduction factor represents the flexibility of the soil at certain depth. Uniform cyclic shear stress can be calculated by Eqn. 2.7.

$$\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d \quad (2.7)$$

where a_{max} is the peak ground surface acceleration, g the acceleration of gravity, σ_v the total vertical stress, and r_d the value of a stress reduction factor at the depth of interest.

The liquefaction resistance of soil depends on how close the initial state of soil to the failure state. Failure state conditions that was mentioned Section 2.2 is related with the characterization of liquefaction resistance. Characterization of liquefaction resistance developed along two line: laboratory tests and in situ tests. In-situ conditions can be replicated stress or strain controlled laboratory tests.

2.2.1.1 Characterization of liquefaction resistance based on laboratory test

Most laboratory tests are performed to analyze the liquefaction resistance of the soil. In these tests, stress condition is important and it is conducted on isotropically consolidated triaxial specimens or K_0 -consolidated simple shear specimens. The criteria of liquefaction failure are defined as the point in which double strain amplitude is reached 5 % percent.

Laboratory tests show that the number of cycles which are required to liquefaction failure, N_L , depends on shear stress amplitude, vertical effective stress and the relative density of the soil (Figure 2.5 and Figure 2.6). As it can be seen in the both figures, in loose specimen, excess pore water pressure ratio reaches to 1.0 at 30th cycle and in denser specimen, r_u reaches to 1.0 at 25th cycle. N_L decreases with increasing shear stress amplitude. The shear strain increases with the increasing number of cycles. Laboratory cyclic strength curves show the relationship between relative density, cyclic stress amplitude, and number of cycles to liquefaction failure. Cyclic stress curves generally associated with initial effective confining pressure, because of this cyclic stress normalized by initial effective confining pressure which is called cyclic stress ratio (CSR).

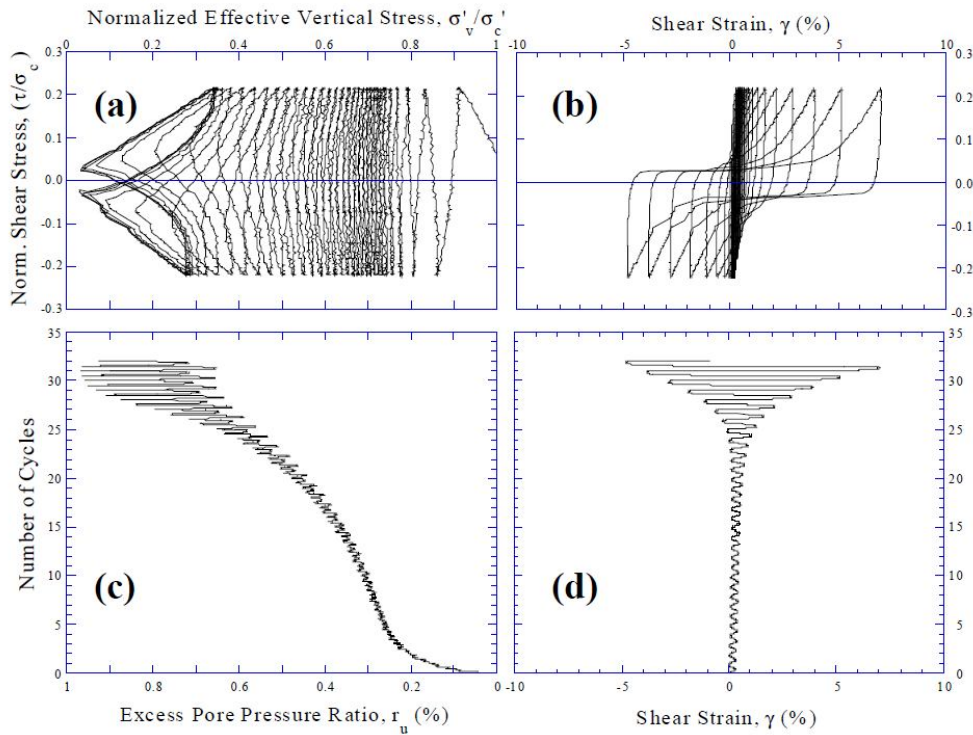


Figure 2.5 : The results of a simple shear test with $Dr=54$ %, $\sigma'_v=95$ kPa and $CSR=0.20$ (Wu, 2002).

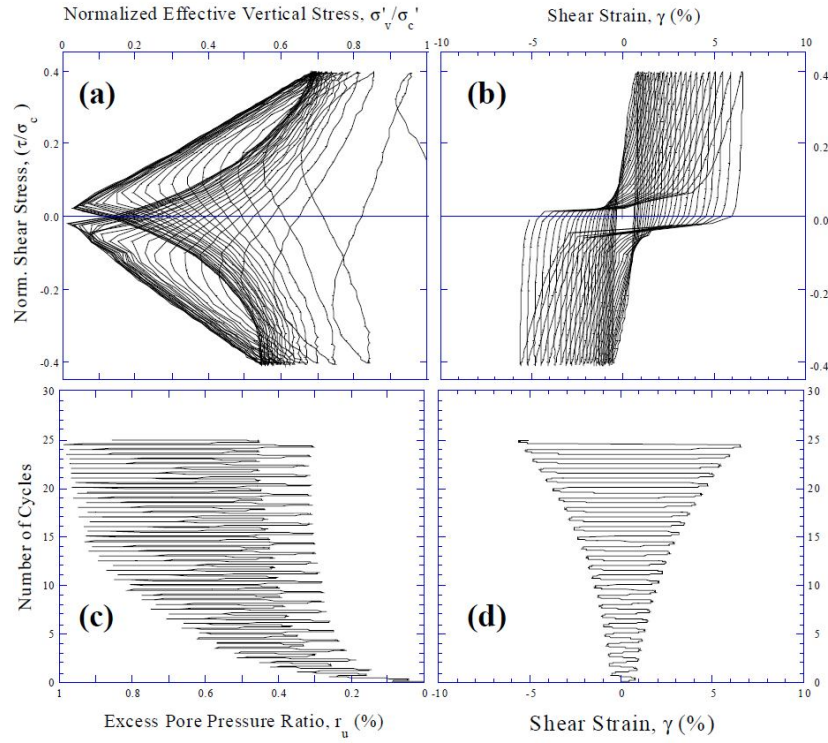


Figure 2.6 : The results of a simple shear test with $Dr=77\%$, $\sigma'_v=77$ kPa and $CSR=0.36$ (Wu, 2002).

Earthquake-induced shear stress is multidirectional and it generates the pore pressure rapidly than laboratory tests. The relationship between laboratory tests and liquefaction in field, Seed et al. (1985) suggested that CSR should be 10% less than that required in unidirectional simple shear tests. The equation can be seen in Eqn. 2.8.

$$CSR_{field} = 0.9 CSR_{ss} \quad (2.8)$$

where CSR_{field} is field cyclic stress ratio, CSR_{ss} is simple shear test cyclic stress ratio.

2.2.1.2 Characterization of liquefaction resistance based on in-situ tests

In-situ test parameters that reflect the density and pore pressure generation characteristics of the soil are used as liquefaction resistance parameters. The in-situ tests are listed in below;

Standard Penetration Test: The standard penetration resistance is correlated with properties of soil skeleton such as relative density, seismic straining, overconsolidation ratio. Seed et al. (1985) compared the corrected SPT resistance and CSR for sand with the percentage of fine content (Figure 2.7). As it can be seen in the figure, the liquefaction resistance increases with the increasing $N_{1,60}$.

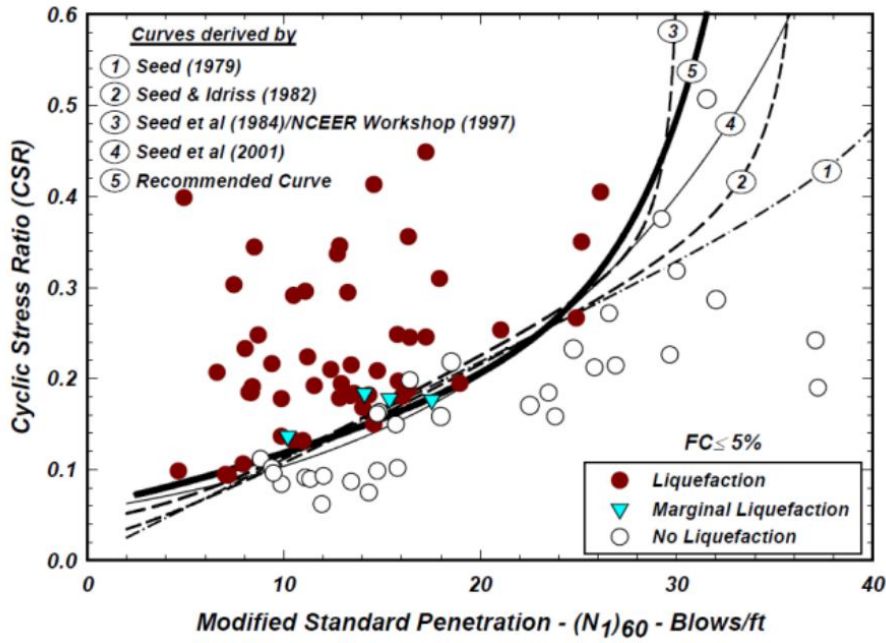


Figure 2.7 : Relationship between CSR causing liquefaction and $(N_1)_{60}$ (Idriss and Boulanger, 2010).

Cone Penetration Test: Cone penetration resistance also has a significant role to obtain liquefaction potential. Despite of SPT, cone penetration test (CPT) can be more reliable in loose soil to decide thin layers. The relation between CPT tip resistance and CSR was shown in Figure 2.8. As it can be seen in the figure, the liquefaction resistance increases with the increasing CPT tip resistance.

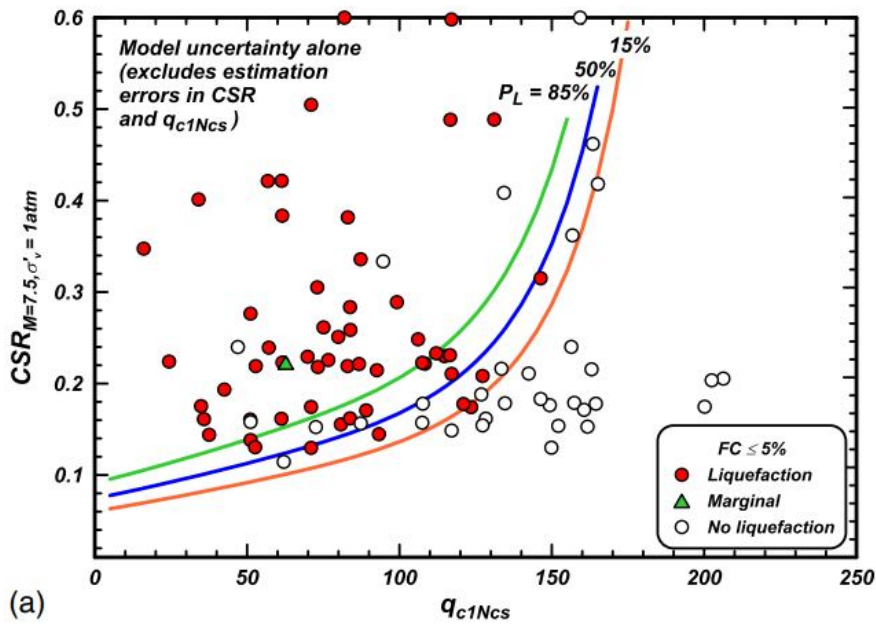


Figure 2.8 : Relationship between CSR causing liquefaction and CPT Tip Resistance (Boulanger and Idriss, 2016).

The cyclic stress approach based on the relationship between earthquake-induced cyclic stresses and uniform cyclic stress required to start liquefaction. In practice, this method must be used with laboratory tests, however, there are some disadvantages due to various parameters such as the relative density of the sample and effective stress. All these factors and certain shear strength may not be reliable. To solve these uncertainties, the cyclic strain approach must be evaluated.

2.2.2 Cyclic strain approach

The previous studies about the cyclic stress approach proved that there is a significant relationship between relative density and initial effective stresses. However, the studies also showed that the effect of density was incorporated cyclic stress and reconstituted samples and field density. Martin et al. (1975) associated cyclic loading under undrained conditions with the amplitude of given cyclic strain (γ_{cyc}) and pore water pressure ratio (r_u) rather than cyclic stress. Dobry et al. (1982) provided a simplified cyclic strain approaches (Eqs. 2.9 and 2.10).

$$\gamma_{cyc} = \frac{\tau_c}{G} \quad (2.9)$$

$$\gamma_{cyc} = 0.65 \left(\frac{a_{max}}{g} \right) \frac{\sigma_v r_d}{G_{max} (G / G_{max}) \gamma_{cyc}} \quad (2.10)$$

where G is the secant shear modulus of the soil, G_{max} , small-strain secant shear modulus of the soil; and $(G/G_{max})\gamma_{cyc}$, normalized shear modulus reduction ratio of the soil corresponding to γ_{cyc} . Dobry et al. (1982) showed that pore water pressure related to cyclic strain than cyclic stresses. Excess pore pressure generation does not increase at a certain cyclic strain which is called threshold strain (γ_t). If $\gamma_{cyc} < \gamma_t$ no pore pressure will be generated and, consequently, liquefaction cannot be initiated. If $\gamma_{cyc} > \gamma_t$, liquefaction is possible and the liquefaction resistance of the soil must be evaluated.

2.2.2.1 Characterization of liquefaction resistance

From laboratory tests, liquefaction resistance can be interpreted simply by the cyclic strain approach. The factors (e.g. density, soil skeleton, over consolidation ratio) that increase cyclic stresses also increase the shear modulus of soil (G). However, these factors affect the ratio of γ_{cyc} and shear modulus of soil less than the ratio of τ_{cyc}/G . The result of this effect provided by Dobry and Ladd (1980), their studies included

three types of sample preparation techniques at three initial effective stresses on two types of sand. As shown in Figure 2.9, pore pressure generation does not affected significantly by these different preparation methods and initial effective stresses at the same amplitude of strain.

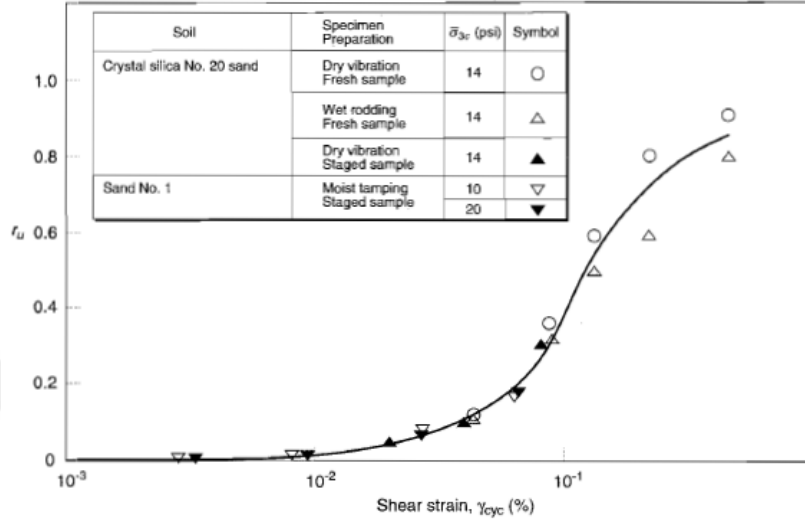


Figure 2.9 : Pore pressure ratio vs. shear strain after 10 loading cycles at a strain-controlled cyclic triaxial test with different sample preparation types (Dobry and Ladd, 1980).

2.2.3 Effective stress-based response analysis approach

An effective stress-based response analysis approach was developed to evaluate liquefaction potential as an alternative to the cyclic stress and cyclic strain approaches. Finn et al. (1976) accomplished an effective stress model that can predict a relationship between excess pore water pressure generation in undrained loading and induced volumetric strain in the same loading increment under drained conditions. The rise in pore water pressure can be computed by Eqn. 2.11;

$$\Delta u = \Delta \epsilon_{vd} \bar{E}_r \quad (2.11)$$

where $\bar{E}_r$ is the one-dimensional rebound modulus of the sand which is calculated at any effective stress was given in Eqn. 2.12.

$$\bar{E}_r = \frac{(\sigma'_v)^{1-m}}{mK_2(\sigma'_{v0})^{n-m}} \quad (2.12)$$

where σ'_v and σ'_{v0} are the current and vertical initial effective stresses and m,n and k₂ are the experimental constants.

The potential volumetric compaction strain is a function of the total volumetric strain and the shear strain amplitudes. The relationship was given for crystal silica sand with a relative density of 45 % (Eqn. 2.13).

$$\Delta \varepsilon_{vd} = C_1(\gamma - C_2 \varepsilon_{vd}) + \frac{C_3 \varepsilon_{vd}^2}{\gamma + C_4 \varepsilon_{vd}} \quad (2.13)$$

where C_1, C_2, C_3, C_4 are the experimental parameters determined from the results of drained cyclic simple shear tests. At the initial cycle, ε_{vd} is assumed zero and $\Delta \varepsilon_{vd}$ is computed by using shear strain and the experimental parameters. After the calculation of $\Delta \varepsilon_{vd}$ and $\bar{E}_r$ at the first cycle, the equations (2.12 and 2.13) should be repeated for each number of loading cycles.

The volume change at different relative densities can be obtained from;

$$(\varepsilon_{vd})_{Dr} = R (\varepsilon_{vd})_{45} \quad (2.14)$$

in which R is a parameter that varies with the relative density. It is calculated as below;

$$R = 0.00031(100 - Dr)^2 + 0.062 \quad (45 < Dr < 80) \quad (2.15)$$

According to Martin et al. (1975), volumetric strain curves for uniform shear strain amplitude cycles were shown in Figure 2.10. This study comprises accumulated volumetric strain versus the number of cycles from simple shear tests on crystal silica sand conducted at three different levels of shear strain and a relative density of 45 %.

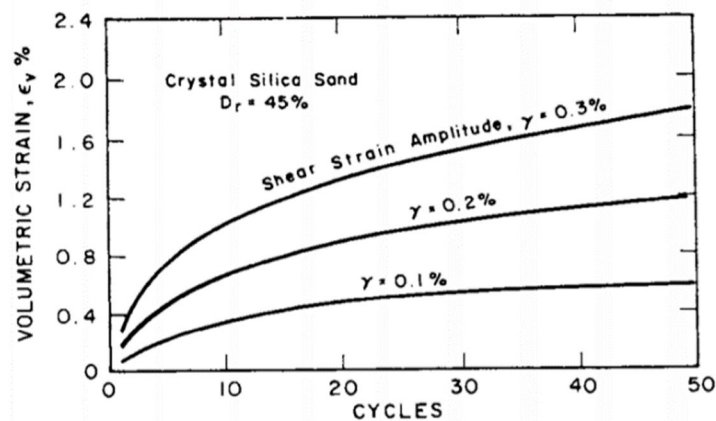


Figure 2.10 : Volumetric strains at different shear strain amplitudes (Martin et al., 1975).

The volumetric strain change ($\Delta \varepsilon_{vd}$) was also expressed by Byrne (1991) simplified method (Eqn. 2.16). Byrne (1991) proposed the following expression for computing

the volumetric strain that would occur under drained conditions in a soil having a given D_r subjected to a cycle of loading of amplitude γ :

$$\Delta\varepsilon_{vd} = \gamma C_1 \exp(-C_2(\frac{\varepsilon_{vd}}{\gamma})) \quad (2.16)$$

C_1 parameter can be calculated according to relative density. C_2 parameter can be computed from the C_1 value.

$$C_1 = 7600 (D_r)^{-2.5} \quad (2.17)$$

D_r should be entered as a percentage and,

$$C_2 = 0.4 / C_1 \quad (2.18)$$

A relationship between volumetric strains from drained cyclic tests and pore water pressure from undrained cyclic tests was developed (Eqn. 2.19). This relationship enables the build-up of pore water pressure during cyclic loading to be computed using basic effective stress parameters of the sand.

$$\Delta u = \frac{\Delta\varepsilon_{vd}}{\frac{1}{\bar{E}_r} + \frac{n}{K_w}} \quad (2.19)$$

where n is the porosity of the soil and K_w is the bulk modulus of water. For fully saturated samples, $K_w = 4 \times 10^7$ psf and $\bar{E}_r$ can be assumed 10^6 psf (1 psf $\approx$ 0.05 kPa). For partially saturated samples, K_w should not be used directly. In 1963, Koning calculated the compressibility of the air-water mixture and developed a relationship between the degree of saturation and pore water pressure (Eqn. 2.20).

$$\frac{1}{K_{aw}} = \frac{S}{K_w} + \frac{1-S}{u_w} \quad (2.20)$$

where K_{aw} is the bulk modulus of the air-water mixture, S is the degree of saturation and u_w is pore water pressure. The volumetric strain at drained simple shear tests was studied by Kang (2016). The tests were performed under strain-controlled cyclic loading and monotonic shear loading at constant K_0 ($K_0 = 0.5$) condition at GCTS test device. Figure 2.11 showed that both volumetric strain (ε_{vd}) and shear stress increased during cyclic loading. When the monotonic shear loading was applied, the shear stress decreased. As a result of dynamic compaction, the loose specimen became denser and it displayed dilation behavior.

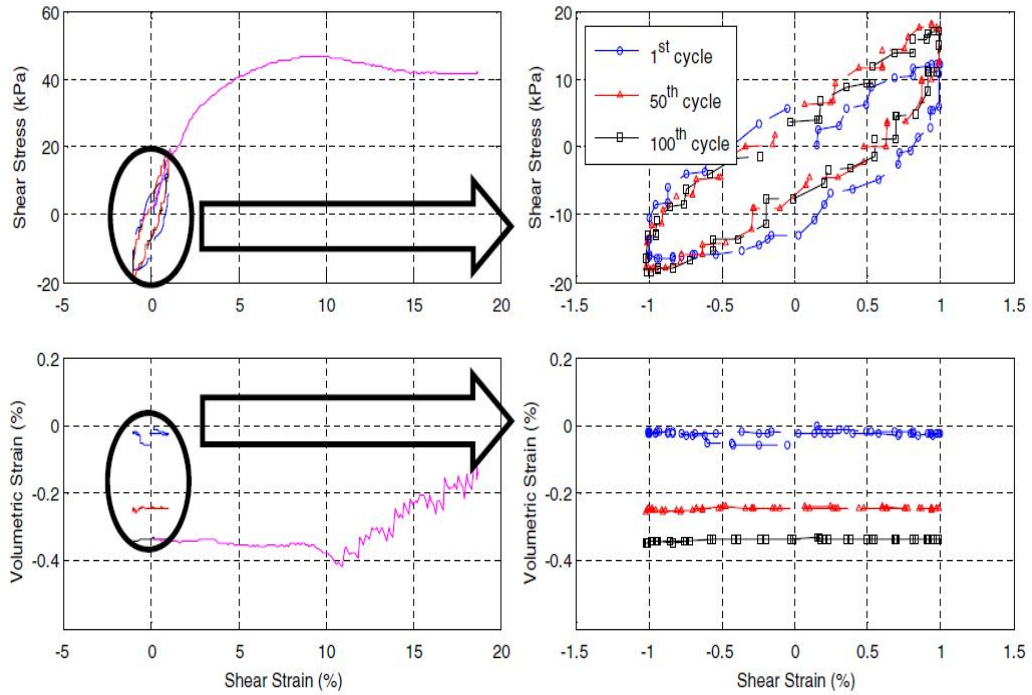


Figure 2.11 : Hysteresis loop and volumetric strain vs. shear strain under 100 kPa confining pressure, at 0.998 % shear strain amplitude (Kang, 2016)

Figure 2.12 demonstrated a relationship between volumetric strains and shear strains in terms of the number of cycles for dense specimens. The volumetric strain increased as the number of cycles and shear strains increased.

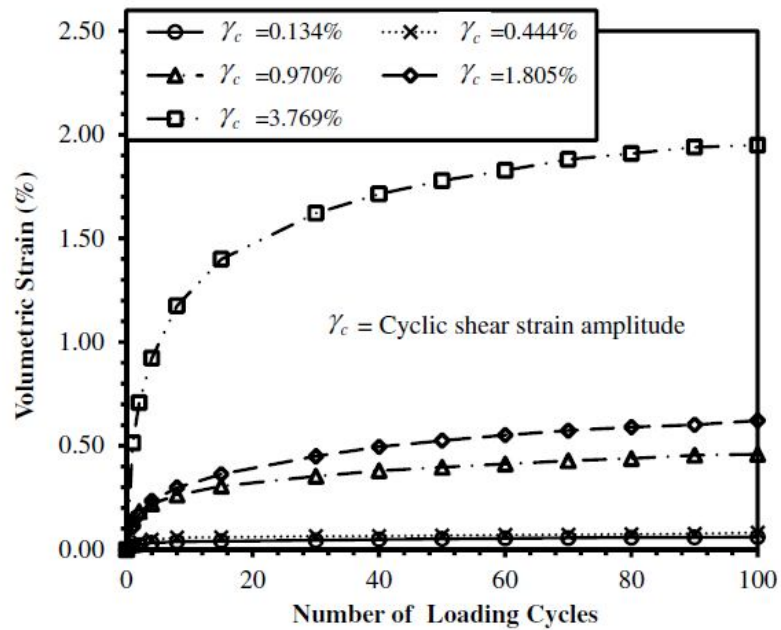


Figure 2.12 : Volumetric strain vs. number of cycles of dense specimens at different cyclic shear strain amplitudes under confining pressure of 100 kPa (Kang, 2016).

2.3 Cyclic Laboratory Tests for Liquefaction Studies

Laboratory tests are performed on undisturbed or reconstituted specimens. In-situ soil conditions such as stress conditions, void ratio, density and consistency are tried to be reflected in the laboratory experiments. Dynamic properties of soils depend on the shear strain amplitudes hence laboratory experiments fall into two categories those that small strain and large strain experiments to measure the dynamic properties.

The maximum shear modulus ($G_{\max}$) and the damping ratio (ξ) are important dynamic parameters that can be obtained from shear stress-shear strain hysteresis loop. The normalized stiffness degradation curve by comparing the ground response from geotechnical construction and the measurement accuracy from laboratory tests were first presented by Atkinson and Sallfors (1991) and Mair (1993) as shown in Figure 2.13. The small strain tests include resonant column, ultrasonic pulse and bender element test. The most commonly used large strain tests are cyclic triaxial, cyclic simple shear and cyclic torsional test.

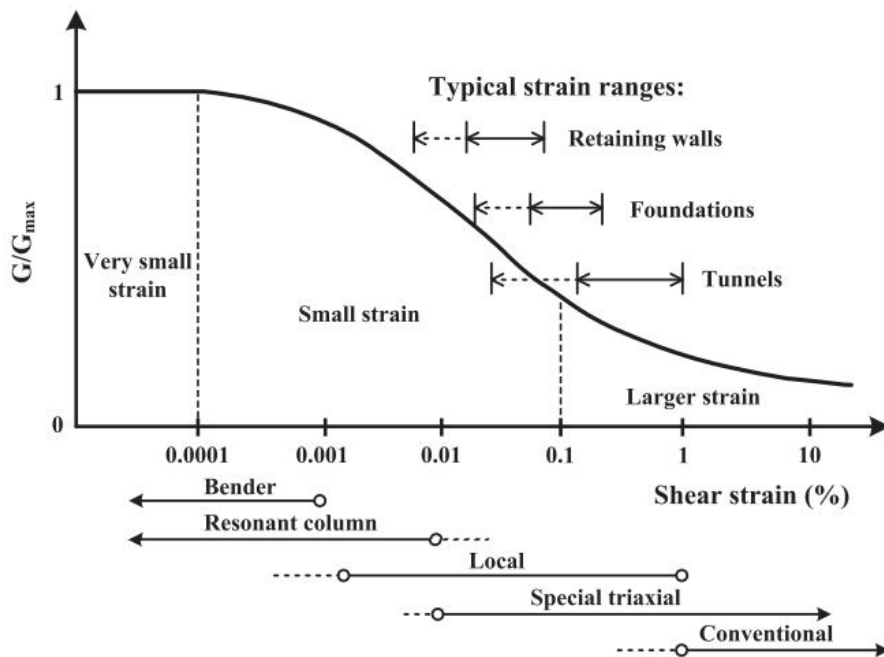


Figure 2.13 : Normalized stiffness degradation curve (modified after, Atkinson and Sallfors (1991) and Mair (1993)).

A stress-strain behavior of soils under cyclic loading can be observed from hysteresis loop (Figure 2.14). The inclination and breadth are the most important characteristics of the shape of a hysteresis loop. The inclination is identified as the tangent shear modulus which depends on the stiffness of the soil. On the other hand, to describe the

equivalent linear behavior of the soil, the average value of tangent shear modulus is expressed by secant shear modulus (G_{sec}) (Eqn. 2.21).

$$G_{\text{sec}} = \frac{\tau_c}{\gamma_c} \quad (2.21)$$

where τ_c and γ_c are the shear stress and shear strain amplitudes, respectively. Under cycling loading, soil behaves linearly at lower shear strain and the maximum shear modulus is obtained from the linear part of the loop.

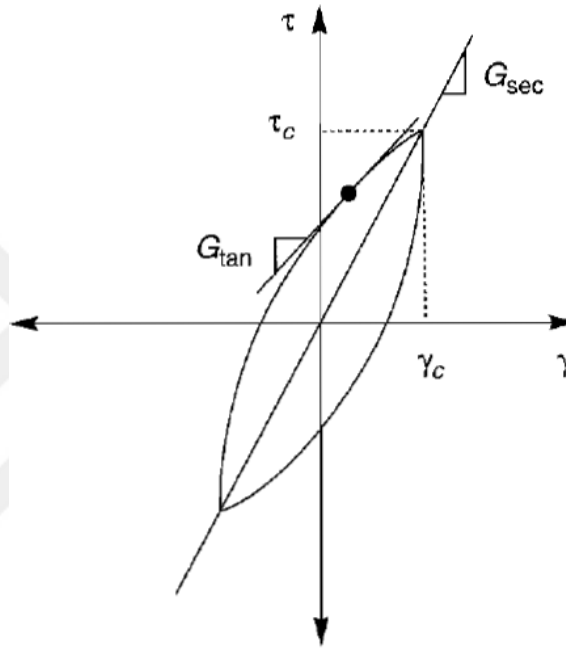


Figure 2.14 : Secant and tangent shear modulus (Kramer, 2016).

The breadth of the hysteresis loop is related to the area that can be described by the damping ratio (Eqn. 2.22). The damping ratio measures the energy dissipation during the cyclic loading.

$$\xi = \frac{\Delta W}{4\pi W} = \frac{1}{2\pi} \frac{A_{\text{loop}}}{G_{\text{sec}} \gamma_c^2} \quad (2.22)$$

where ΔW is the dissipated energy in one cycle, W is the maximum strain energy and A_{loop} the area of the hysteresis loop. A_{loop} can be seen in Figure 2.15. The damping ratio is independent of frequency.

In situ, the damping ratio is determined by analysing of shear waves propagating along the horizontal direction with particle motion. Crosshole and Downhole tests can be used to generate shear waves.

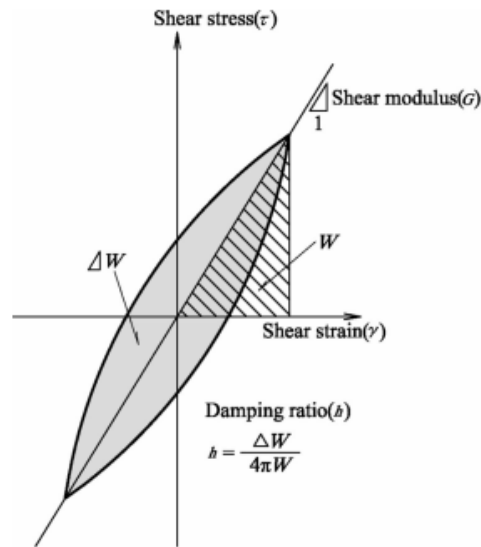


Figure 2.15 : Hysteresis loop (Hayashi et al., 2018).

2.3.1 Small strain tests

The small strain level is where the stiffness modulus varies non-linearly with the strain. Small strain behavior can be seen at offshore soil, machine foundations, compaction, densification and construction activities. The well-known small strain test is the resonant column test. The test is performed under increasing frequency until shear strain amplitude reaches a maximum. Ultrasonic pulse and bender element tests are also small strain tests in which wave propagation velocities can be measured (Kramer, 2016).

2.3.2 Large strain tests

At large strain levels, soils tend to volume change. Under drained conditions, the tendency of volume change showed up as volumetric strain, but under undrained conditions excess pore pressure generated due to volume change. The aim of large strain tests is measuring volumetric strains or pore pressure.

2.3.2.1 Cyclic triaxial test

Cyclic triaxial testing was firstly performed by Seed and Lee (1966). In the triaxial test, a cylindrical specimen is placed between the top and bottom loading platens and formed in a latex membrane which can be seen in Figure 2.16. The cyclic triaxial test can be performed under isotropically or anisotropically consolidated conditions. A servo-system is used to apply cycles for strain or stress-controlled tests (Wang et al., 2016; Yang et al., 2018).

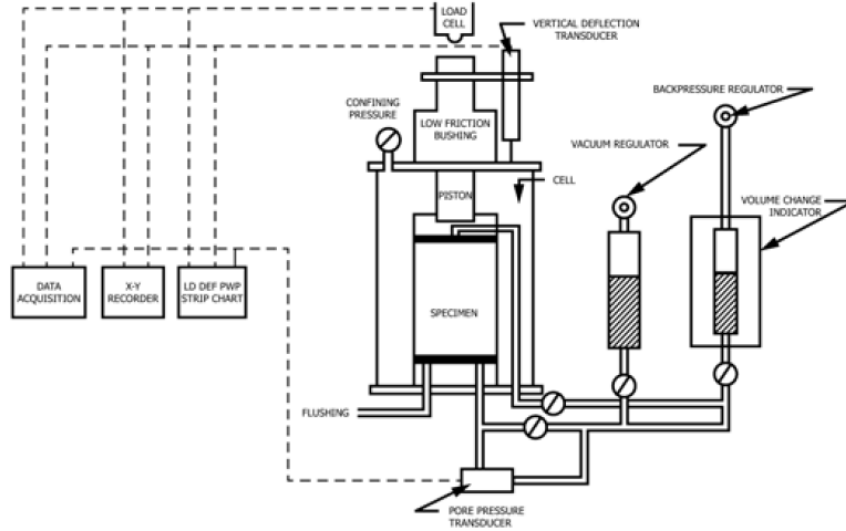


Figure 2.16 : Triaxial apparatus (ASTM D5311/D5311M-13).

The specimen height to diameter ratio should be between 2.0 and 2.5. Intact or reconstituted specimens can be used at the tests. The intact specimens should be trimmed by any method that does not affect the relative density and the water content. For the sampling, the split tube is preferred to facilitate the removal of the sample with minimum disturbance. Reconstituted specimens can be prepared by different techniques. These are pluviation, dry or moist vibration and moist tamping methods. After preparation of the specimen in the split mold, vacuum is applied to prevent disturbance during removing the mold (ASTM D5311/D5311M-13).

The testing stages are followed as saturation, consolidation and cyclic loading. At the saturation stage, the sample is saturated until the desired degree of saturation. The degree of saturation is evaluated by Skempton's coefficient (B). The B-value is the ratio of the increment in pore water pressure, Δu to the increment in isotropic confining pressure, $\Delta \sigma$ and can be expressed by the following equation:

$$B = \frac{\Delta u}{\Delta \sigma} = \frac{1}{1 + n \frac{K_s}{K_{aw}}} \quad (2.23)$$

where, K_s and K_{aw} are the bulk modulus of the solid skeleton and air-water mixture, respectively; n is the porosity of the soil. B value can be also expressed in terms of the degree of saturation (Eqn. 2.24).

$$B = \frac{1}{1 + n \frac{K_s}{K_w} + n K_s \frac{(1-S)}{u}} \quad (2.24)$$

For fully saturated soil, B value should be reached at least 0.95. Chaney (1978) specified that coefficient B should be higher than 0.96 so that the soil is well saturated. However, in partially saturated soil, B value is lower than 1.0 due to the surface tension. In Figure 2.17, partially saturated zone vs. B value was shown for Toyoura sand with different poisson ratios.

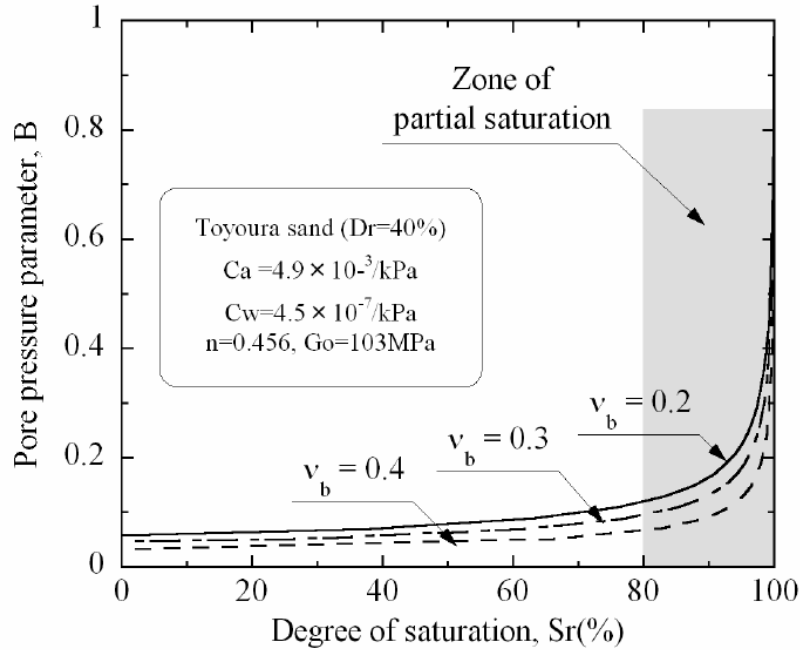


Figure 2.17 : B-check vs. degree of saturation in partially saturated sands (Kamata et al., 2009).

After the saturation stage, the sample is consolidated at the desired consolidation pressure to reflect in-situ behavior. The final degree of saturation can be calculated after consolidation. B-value should be rechecked before cyclic loading.

Deviatoric stress is applied cyclically for both strain and stress-controlled tests. During the test, radial stress remains constant and axial stress cycled at a desired frequency. The elasticity and shear modulus can be obtained from the test.

2.3.2.2 Cyclic simple shear test

The first cyclic simple shear test device was developed by the Royal Swedish Geotechnical Institute (SGI) in 1936 (Kjellman, 1951). In this method, a short cylindrical soil specimen was prepared in a rubber membrane and surrounded by aluminum rings (Figure 2.18). In 1961, SGI-type device was enhanced by Norwegian Geotechnical Institute (NGI) by Landva (Bjerrum and Landva, 1966). The cyclic simple shear test is an ideal duplication of the in-situ stress conditions much better than

the cyclic triaxial test. By applying cyclic horizontal shear stresses to the top or bottom of the specimen, the test specimen is deformed in much the same way as an element of soil subjected to vertically propagating S-waves.

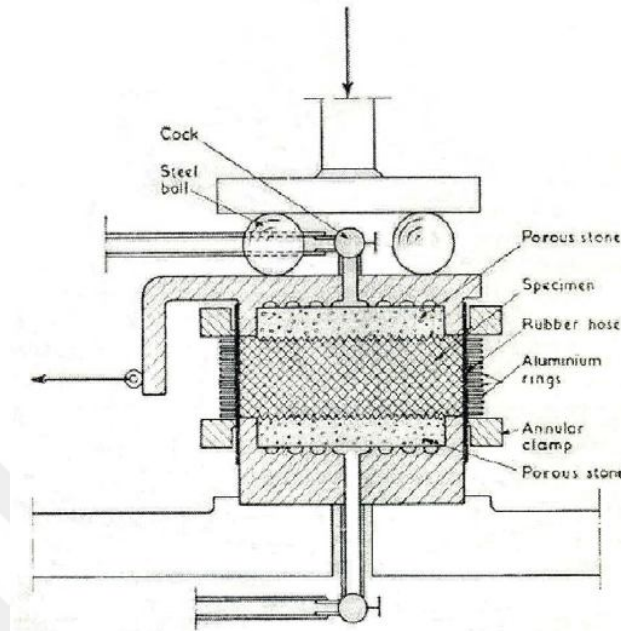


Figure 2.18 : SGI- type simple shear apparatus (Kjellman, 1951).

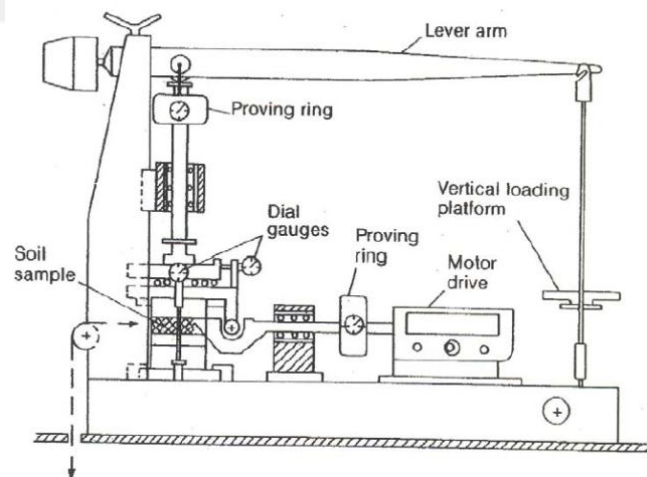


Figure 2.19 : NGI-type simple shear apparatus (Bjerrum and Landva, 1966).

NGI-type of device can be seen in Figure 2.19. The wire-reinforced membrane is used to confine the specimen at NGI type simple shear test. The reinforced membrane limits lateral boundaries and the volume is kept constant during shearing. Other developments on the direct simple shear device were improved by Roscoe in 1953 at Cambridge University and it was called Cambridge type simple shear test device. In this type of device, a cubical specimen is supported by rigid lateral plates.

Figure 2.20 presented the basic definitions of the simple shear mechanism. Shear stress is applied which results in shear strain. The shear strains assume that the tangent of the angle which occurs between initial form and reformed shape.

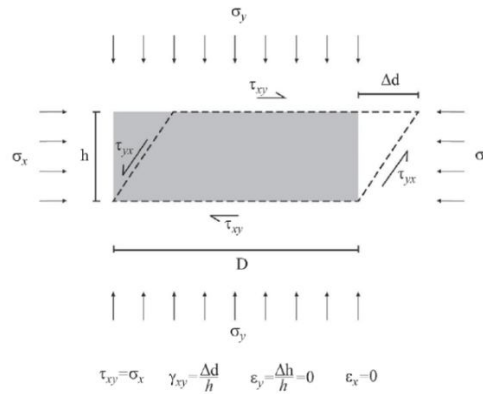


Figure 2.20 : Basic definitions of the simple shear apparatus (Corte et al., 2017).

2.3.2.3 Cyclic torsional shear test

In cyclic torsional shear testing, a solid cylindrical or hollow cylindrical specimen is confined by cell pressure through a latex membrane. A hollow cylinder configuration is shown in Figure 2.21.

In this test, hollow cylindrical samples are used to prevent stress accumulation in the top of the sample. Hollow cylinder samples offer uniformity and control over stresses (Yamada et al., 2008). The torque with a certain frequency is applied to the top of the sample and shear stresses are induced to obtain shear strain in the sample (Ishihara, 1996). The drainage can be controlled well but it is not a widely used test device since the sample preparation is quite difficult.

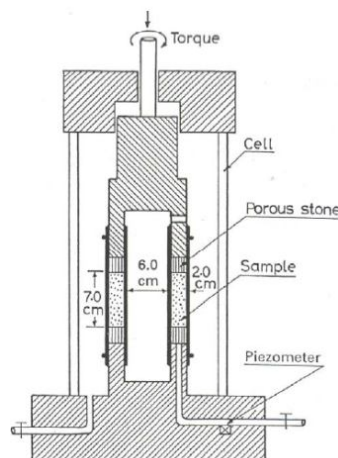


Figure 2.21 : Cyclic torsional test apparatus (Ishihara, 1996).

2.4 Partial Saturation as a Liquefaction Mitigation Method

The liquefaction of saturated loose sands under the cyclic loading is a typical example of the poor performance of cohesionless soils. Since the liquefaction is known to occur in sand and silts below the groundwater table, a vast majority of the laboratory tests have been conducted on fully saturated sands. Laboratory tests were shown that the liquefaction potential of sand tends to increase with an increment in the degree of saturation (Ishihara et al., 2004).

Partially saturated soils are considered a relatively new field of study in the geotechnical engineering. In partially saturated soils, the soil skeleton consists of three phases comprised in a soil-water-air interface at equilibrium as illustrated in Figure 2.22. The air-water mixture in the voids of the soil absorbs induced cyclic shear stresses thus effective stress does not decrease dramatically.

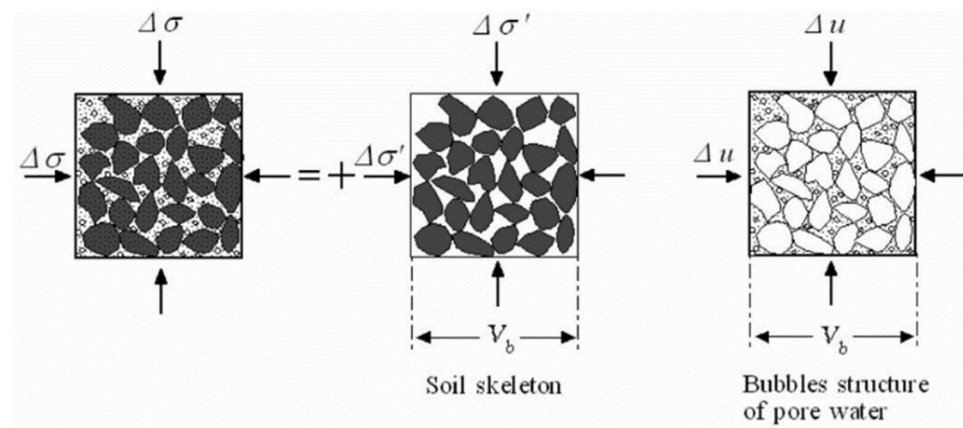


Figure 2.22 : The change of the effective stress and pore water pressure (Ishihara et al, 2004).

Liquefaction resistance of partially saturated sands is higher than fully saturated soils. During the cyclic loading excess pore water pressure generation was prevented due to the compressibility of air-water mixture. Martin et al. (1975) presented that a small reduction in the degree of saturation of a saturated sand specimen caused decrement in the bulk modulus of water and it led to a reduction in the pore water pressure increased per cycle.

Intensive efforts have been made to develop the desaturation technique as an liquefaction mitigation (Table 2.1). These include air injection, water electrolysis, sodium perborate and sodium percarbonate, microbial denitrification. The chemical

and biological methods are most preferred due to the generation of homogeneous distribution of gas bubbles.

Table 2.1 : Desaturation techniques as a liquefaction mitigation method.

Reference	Experimental Setup	Specimen Preparation Method	Mitigation method
Nagao et al. (2007)	Cyclic triaxial tests	Air-pluviation Toyoura sand	Injection of air bubbles
Yegian et al. (2007)	Shaking table	Wet pluviation	Water electrolysis method
Eseller-Bayat (2013)	Shaking table with cyclic simple shear liquefaction box	Wet pluviation with chemical powder	Sodium perborate method
He et al. (2013)	Cyclic triaxial and shaking table with laminar box	Moist tamping and wet pluviation	Biogas method
Marasini and Okamura (2015)	Centrifuge test	Tamping method Toyoura sand	Injection of air bubbles
Gülen and Eseller-Bayat (2017)	Dynamic simple shear with confining pressure test	Wet pluviation with chemical powder	Sodium percarbonate method

He et al. (2013) performed cyclic triaxial and shaking table with laminar box tests. Partially saturation was provided by microbial denitrification. Specimens were prepared by moist tamping and wet pluviation. In this study, a relationship between excess pore water pressure ratios and volumetric strains was observed at different degrees of saturation. It was observed that volumetric strains in partially saturated sands were lower than fully saturated sands (Figure 2.23).

The study also showed that there was a relationship between excess pore water pressure ratios and volumetric strains. The lower volumetric strains were obtained at the lower excess pore water pressure ratios. When the samples liquefy ($r_u=1.0$), the higher volumetric strains were observed.

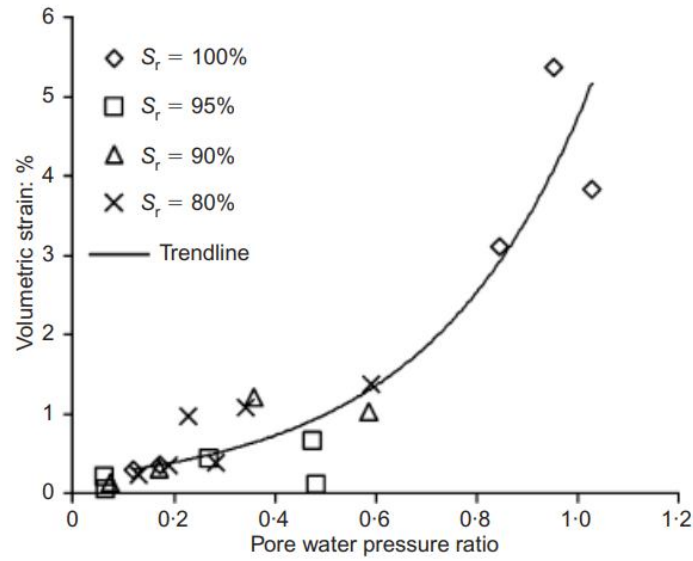


Figure 2.23 : Volumetric strains vs. pore water pressure ratios with degrees of saturation (He et al., 2013).

Gülen and Eseller-Bayat (2017) compared the responses of fully and partially saturated sand samples under different vertical effective stresses and shear strain amplitudes through undrained strain-controlled tests in dynamic simple shear test device with confining pressure. The specimens were prepared by different methods depending on the degree of saturation. For fully saturated sand specimens, moist tamping method was selected to achieve a loose soil skeleton. The induced partial saturation method was preferred to obtain partially saturated sand specimens by wet pluviation technique. Figure 2.24 shows that lower excess pore pressure ratio (r_u) generations were observed in partially saturated sand samples compared to the fully saturated sand samples.

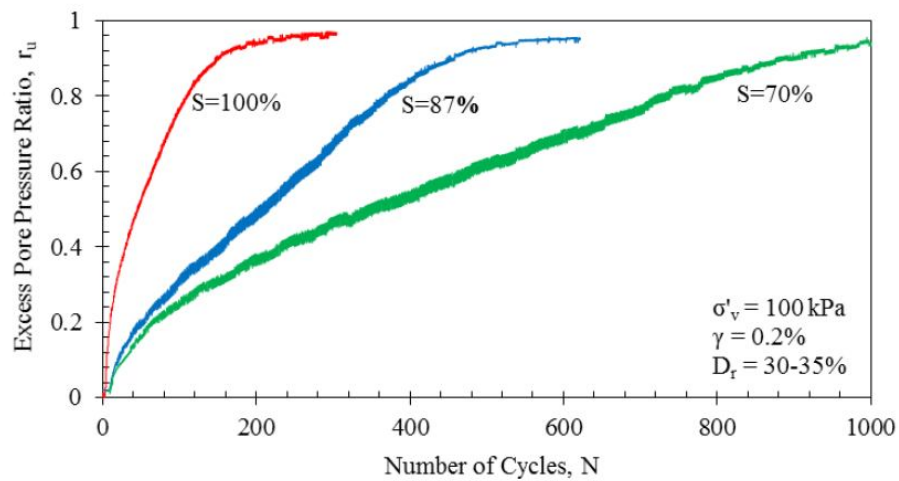


Figure 2.24 : Excess pore pressure generations (r_u) vs. number of cycles in fully and partially saturated sand specimens at $\sigma'_v=100$ kPa and $\gamma=0.2$ % (Gülen and Eseller-Bayat, 2017).

As a result of the study, liquefaction resistance of partially saturated sands was found to be higher than that of fully saturated sands. At the same shear strain levels, excess pore pressure ratio reduces as the degree of saturation decreases as seen in Figure 2.25.

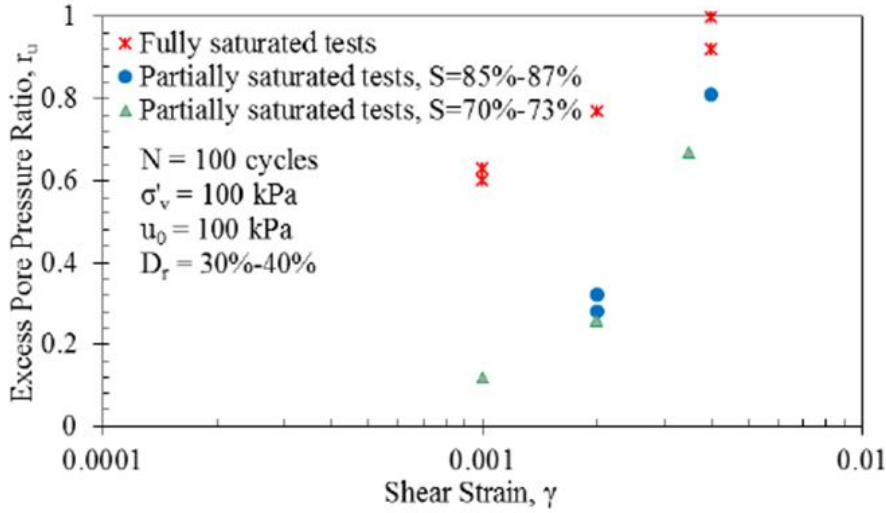


Figure 2.25 : Excess pore water pressure generations vs. different shear strain in fully and partially saturated sand specimens at $\sigma'_v=100$ kPa (Gülen, 2017).

In summary, the studies was evaluated to understand liquefaction mechanism in fully and partially saturated sands. This study aimed to investigate the drained behavior of the partially saturated sands prepared by IPS under cyclic loading and to observe volumetric strains in terms of degrees of saturation, shear strain amplitudes and relative densities. The primary goals of the study was to evaluate volumetric strains that obtained from drained cyclic simple shear tests. It was expected to observe lower volumetric strains when the degree of saturation decreased. The results were also compared with some other studies reported in the literature. Another goal was to investigate whether there was a relationship between excess pore water pressure ratios and volumetric strains. Finally, air-water bulk modulus was estimated by using undrained and drained test results.

In this study, the undrained and drained tests were performed by dynamic simple shear test device with confining pressure (DSS-C). To reproduce earthquake stress conditions more accurately, strain-controlled tests was preferred in DSS-C.

3. STRAIN CONTROLLED CYCLIC TESTING PROGRAM

This chapter was presented the details of the testing program. First, the testing set-up of DSS-C was introduced. The soil tested were presented with grain size distribution and index properties. Specimen preparation techniques were explained in detail for fully and partially saturated sands. Then, saturation, consolidation and strain-controlled cyclic simple shear procedures were outlined.

3.1 Dynamic Simple Shear System With Confining Pressure

In this study, VJ-Tech dynamic simple shear test device was used to investigate dynamic properties of the noncohesive soils under the effect of vertical effective stress. The device is controlled by Clisp Studio program. The controlling software enables all stages of a test Saturation, Consolidation (Isotropic, Anisotropic or K_0), Static Loading, Cyclic Shear (Stress or Strain) and Liquefaction. Plain membrane was preferred because the cell pressure cannot be applied in this device when wire reinforced membrane is used. The principle of the test device is focused on keeping the specimen height constant. The mechanism of the device can be listed as below (VJ-Tech Manual, 2016):

- A simple shear test with the height of the sample kept constant using active height control at undrained conditions
- A cyclic shear test with a cyclic horizontal force applied to the specimen whilst the height of the specimen is maintained at undrained conditions
- A cyclic shear test with a cyclic horizontal displacement applied to the specimen whilst the height of the specimen is maintained at undrained conditions
- A cyclic shear test with a cyclic horizontal force applied to the specimen whilst the vertical stress on the specimen is maintained at drained conditions
- A cyclic shear test with a cyclic horizontal displacement applied to the specimen whilst the vertical stress on the specimen is maintained at drained conditions

The DSS-C device is shown in Figure 3.1 and comprises of:

- A horizontal and a vertical force actuator
- Strain rods with an adjustable height horizontal cross beam
- A horizontal and a vertical internal submersible load cells
- A base plate with adjustable feet
- An adapted triaxial type cell with the sample containing unit

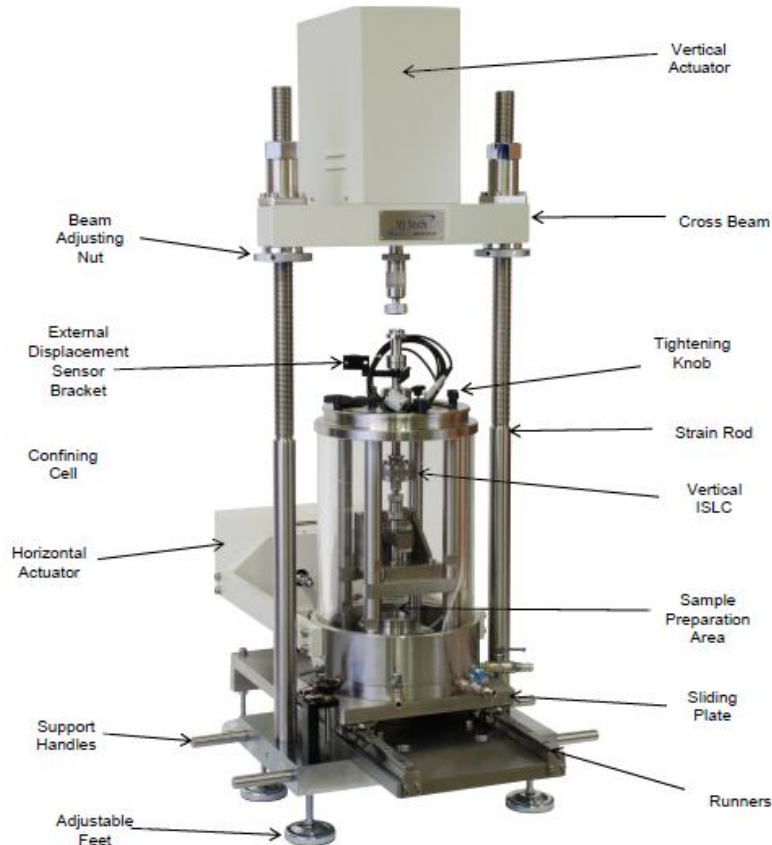


Figure 3.1 : Dynamic simple shear with confining pressure test device (VJ-Tech Manual, 2016).

The test device has five basic system components which are DSC, DSS-C, air-water cylinder, pneumatic APC and hydraulic APC. DSC and DSS-C are controlled by Clisp Studio software. The horizontal actuator provides the horizontal displacement during dynamic tests. The pneumatic APC transfer the air pressure to water pressure in the cell. The compressor procures air pressure and the conversion occurred in air/water cylinder. Hydraulic APC (APCH) is controlled back pressure via water pressure. The water volume capacity of APCH is 200 cm³ and it should not exceed. The experimental setup in ITU Geotechnical Engineering Laboratory can be seen in Figure 3.2.

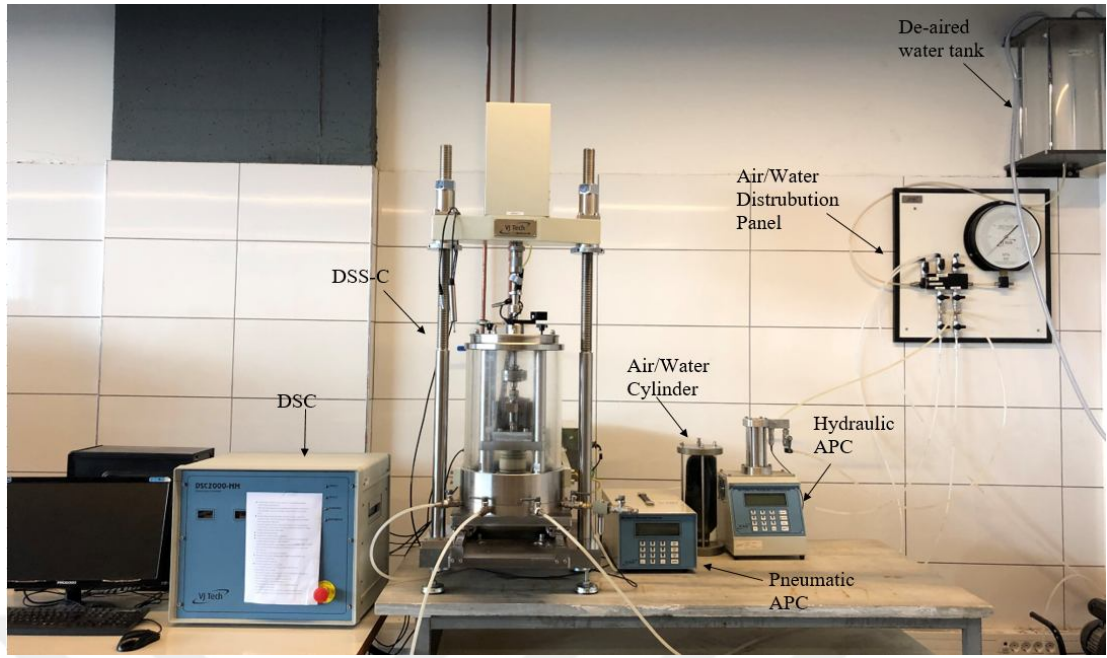


Figure 3.2 : DSS-C test device at ITU Geotechnical Engineering Laboratory

3.2 Properties of Soil Tested

Şile sand (AFS 40-45) was used to investigate the dynamic characteristics of clean sands. The grain size distribution of Şile sand was shown in Figure 3.3. The index properties were listed in Table 3.1. Sieve analysis and hydrometer tests were determined in accordance with ASTM D6913-17 and ASTM D7928-17. The maximum and minimum void ratios were determined in accordance with ASTM D4253 and ASTM D4254, respectively.

Table 3.1 : The properties of Şile sand.

Properties of Şile Sand	Values
Soil type	SP
$e_{\max}$	0.890
$e_{\min}$	0.570
γ_s (g/cm ³)	2.65
D ₁₀	0.22
D ₃₀	0.28
D ₆₀	0.35
C _u	1.59
C _c	1.02

The relative density is a significant role on liquefaction potential. It was calculated during preparation and after the consolidation stage.

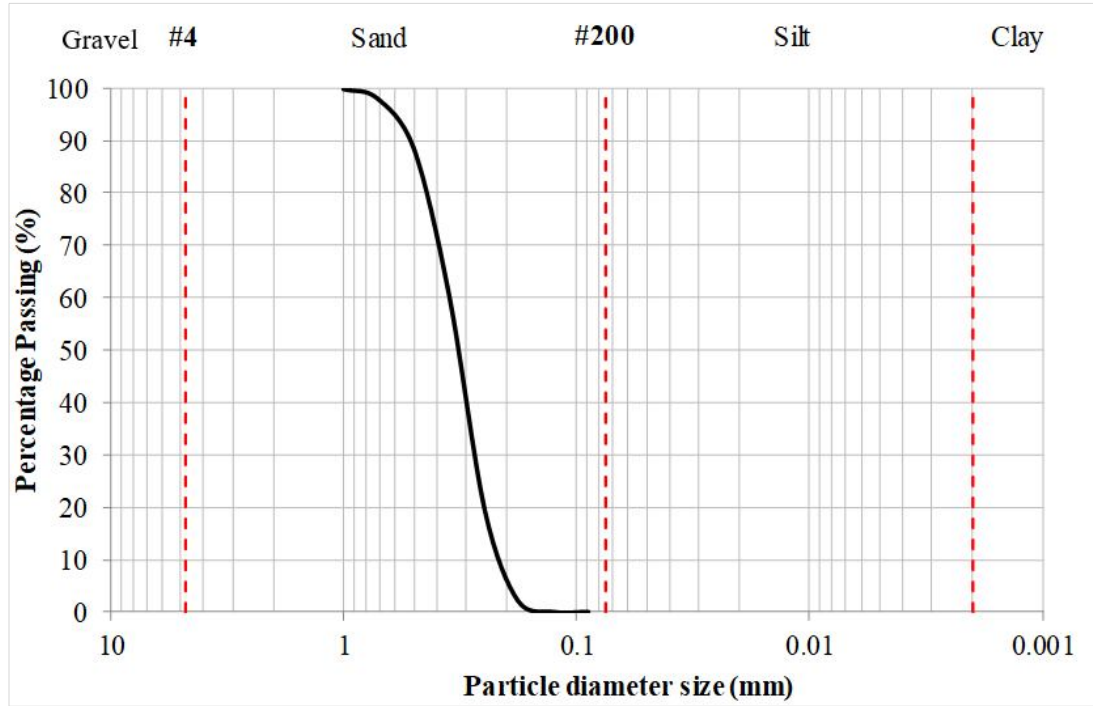


Figure 3.3 : Grain size distribution of Şile Sand.

3.3 Sand Specimen Preparation

Sample preparation methods were chosen according to target relative density and degree of saturation. Fully saturated samples were prepared by moist tamping whereas partially saturated samples were prepared by wet pluviation. Each of these techniques was discussed in more detail in the sections below. The general installation in the sample preparation area of DSS-C device was the same for both fully and partially saturated samples.

As the first step, full saturation was achieved by dipping porous disks in water for at least a few hours. The discs were regularly cleaned to determine whether they were clogged. The saturated porous disc and filter paper were placed onto the base pedestal of the cell and the base pedestal was greased to seal the membrane to avoid any leakage. Another disc and filter paper were used for the top cap.

The filter papers prevented soil particles escaping into the porous discs. The plain membrane was placed over the base pedestal and sealed with rubber O-rings. The split mold jointed the other two O-rings and placed over the membrane. The rubber membrane was folded over the mold. Figure 3.4 showed that the split mold, base pedestal and O-rings with O-ring tool.



Figure 3.4 : (a)O-ring tool, (b) split mold and (c)base pedestal of the test device.

In the second step, the prepared base pedestal was placed into the sample preparation area of DSS-C. Then, a partial vacuum was applied to stretch the membrane tightly. Then, a fully or partially saturated sample prepared inside the mold. The saturated drainage tubes were attached to the base pedestal and the top cap. The top cap was placed on top of the sample carefully. The membrane was folded up on the mold and then O-rings are slid off the split to the top cap. The split mold was removed and the diameter of the sample was measured.

In the third step, the top platform was placed onto the top cap and secured the four bolts. Thus, the connection of vertical ram was completed and the vertical load can be applied during the test. In the final step, the acrylic cell tube was placed and the clamp plate was located on it. Before the test, dynamic vertical and horizontal mechanism were checked to ensure both of them were aligned. In the end, the horizontal and vertical displacement transducers were fitted in their bracket.

3.3.1 Fully saturated sand sample preparation

There were different types of sample preparation methods for fully saturated sands. In literature, three types of preparation methods were checked and the advantages and disadvantages were evaluated. These methods were dry pluviation, wet pluviation and moist tamping. Gülen and Eseller-Bayat (2017) was investigated that the most suitable sample preparation of fully saturated samples to be tested in DSS-C device. Dry pluviation and wet pluviation methods were eliminated due to obtaining relatively higher relative densities. The relative density was one of the major parameters which affect liquefaction experiment results. Thus, in this study, the moist tamping method was preferred.

3.3.1.1 Moist tamping method

In the moist tamping method, the soil was compacted by applying a static load to layers in a membrane-lined mold using a tamper (Ladd, 1974). The main advantage of this method was the ability to predetermine the relative density and to control each layer by adjusting the dry weight of sand and the height of the sample. The disadvantage was the possibility to obtain nonhomogeneous soil fabric due to multi-layers. To avoid the nonhomogeneity, here, only three layers were used with the same amount of dry sand. The initial experiments were conducted with 15% initial moisture content, however, the sand particles were placed nonhomogeneously and major gaps have occurred in the soil fabric. Therefore, initial moisture content was decreased to 10 % and homogeneous and loose soil fabric was obtained.

Predetermined weights of dry sand and water for each layer were prepared in lidded containers to prevent evaporation before mixing. Each layer composed of 160 grams of the sand-water mixture (~53 g sand/layer) and placed into the mold. Then, each layer was compacted using a tamper by incremental forces. Prior to the beginning of compaction, the soil was in a very loose condition because it was placed into the mold by spooning, and the capillary forces due to the added moisture prevented the collapse of the soil when the mold was split (Frost and Park, 2003).

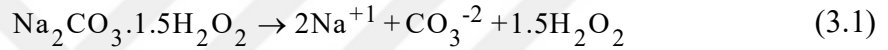
The soil parameters of a fully saturated sample were listed in Table 3.2. At the initial condition, the degree of saturation was lower ($S=33.6\%$) and it was almost in unsaturated condition. Then, the sample was saturated to achieve full saturation during saturation stage with back pressure. The details were given in Chapter 3.4. After saturation stage, the degree of the samples reached around 95-100 %. The calculation of degree of saturation can be examined in Appendix A.

Table 3.2 : The measurements of a fully saturated sample.

	Initial condition
The weight of sand, W_s (g)	160.0
The weight of water, W_w (g)	16.0
The height of the sample, H_s (mm)	29.3
Diameter, D (mm)	68.5
Degree of saturation, S (%)	33.6
Relative density, D_r (%)	31.7

3.3.2 Partially saturated sand sample preparation

In this study, the induced partial saturation (IPS) method was preferred as a partially saturated sand sample technique. Eseller (2004) and Yegian et al. (2007) improved the IPS method by generating gas bubbles inside the sand specimen. The IPS method aims that the water in the voids of the sand replaced with generated gas bubbles. The induction of gas bubbles reduced the degree of saturation and hence, increased the compressibility of the soil due to air-water mixture in the voids. Here, sodium percarbonate ($\text{Na}_2\text{CO}_3 \cdot 1.5\text{H}_2\text{O}_2$) was used as air/gas bubbles generating chemical when contacted with water. Depending on the chemical ratio, the degree of saturation can be controlled. Once sodium percarbonate dissolved in water and injected into fully saturated soils, it could transform them to partially saturated soils. The chemical reaction was shown in equations 3.1 and 3.2. (Kazemiroodsari, 2016).



In the series of experiments were conducted to investigate the effect of sodium percarbonate and sand ratios on the degree of saturation. To find a correlation, the amounts of the chemical and dry sand were changed in the ratio of R which was expressed in equation 3.3.

$$R = \frac{\text{Weight of sodium percarbonate (g)}}{\text{Weight of dry sand}} \times 100 \quad (3.3)$$

The samples were prepared in different concentrations varying in the range of 0.05 % to 0.45 % in glass beakers by the wet pluviation methods. In those series of experiments, the reaction was observed to start in 15 minutes and lasted for 8 hours. Figure 3.5 indicated that the reaction was faster during the first 4 hours and slowed down with time. The reaction rate also depended on the ratio of R. After 24 hours, the height of sample and the level of free water that collected top of the sample was measured. The final relative density and degree of saturation was calculated. The correlation between the ratio R and the degree of saturation was plotted in Figure 3.5. Therefore, in each test, the ratio needed for the target degree of saturation was determined from the correlation in Figure 3.6. The calculation of degree of saturation was given in Appendix A.

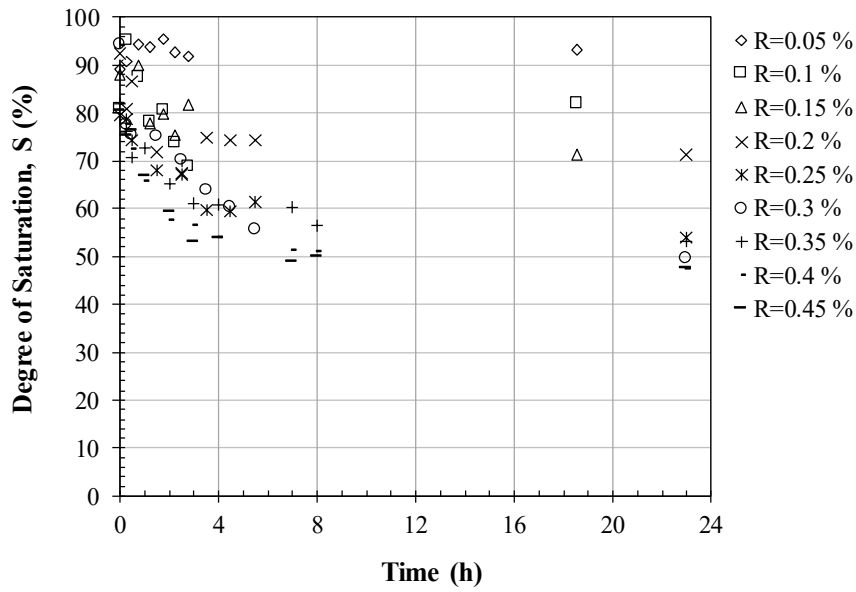


Figure 3.5 : Correlation between degree of saturation and time for different R values within 24 hours.

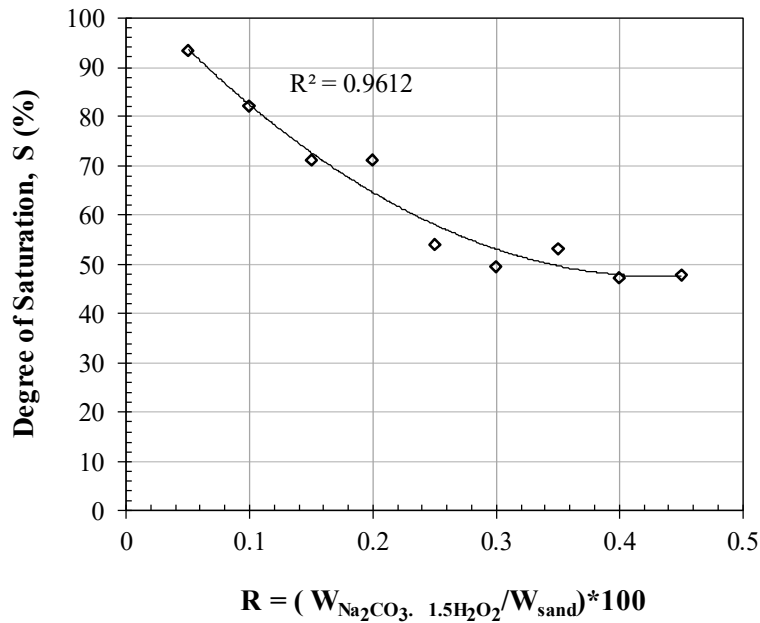


Figure 3.6 : Correlation between the sodium percarbonate to sand ratios versus the degrees of saturation.

The glass beakers allowed us to observe the generated air bubbles in the voids of the samples, hence facilitated the evaluation of the soil fabric homogeneity (Figure 3.7). The decrease in the degree of saturation and the change in the dense form of the samples were negligibly low at 0.05-0.15 % ratios. Larger air bubbles, which cause

nonhomogenous soil skeleton and unsaturated conditions, were observed at 0.35-0.45 % ratios. In the experiments, to obtain homogeneous and loose samples, we chose 0.2-0.3 % ratios. The wet pluviation technique was explained in detail in the next section.



Figure 3.7 : Specimens prepared to obtain degrees of saturation in terms of different ratios R .

3.3.2.1 Wet pluviation method

In this study, partially saturated sand samples were prepared by wet pluviation method. The wet pluviation was known to create denser soil fabric. However, this disadvantage of the method was overcome by entrapping air/gas in the sand voids with IPS technique. Entrapped gas molecules decreased the degree of saturation and relative density of samples. Gülen and Eseller Bayat (2017) were discussed wet pluviation technique with sodium percarbonate-sand and sodium percarbonate-water mixture.

As expressed in section 3.3, the mold was set and placed into the sample preparation area of DSS-C. The amount of sodium percarbonate was determined using the correlation shown in the previous section in Figure 3.6. The mixture of water-sodium percarbonate was prepared into the mold. Then, the pre-weighted sand was poured through water into a water-filled membrane placed on a forming mold. The top of the sample was leveled without disturbance. The height of the sample and free water level was measured to calculate its relative density and degree of saturation. In initial conditions, the sample had dense soil fabric and it was almost fully saturated. The top of the sample was covered with stretch film to prevent evaporation during 24 hours of the chemical reaction. The following day, the measurements were repeated and the air/gas bubbles generation in the voids of the sample was observed. Entrapped air/gas bubbles caused a reduction in the degree of saturation, S which was around 75-90 % depending on the sodium percarbonate to sand ratios (R). The free water collected on the surface and water level rise was measured to obtain the degree of saturation and

relative density of the sample. The free water above sand can be seen in Figure 3.8. The sample heaved due to air/gas bubbles and the height of the sample increased. After the chemical reaction, the sample became looser and partially saturated. The measurements and calculated soil parameters before the experiment were listed in Table 3.3. The degree of saturation and relative density was recalculated after saturation and consolidation stages. The calculations were given in Appendix A.



Figure 3.8 : 24 hours after the prepared partially saturated sand sample.

Table 3.3 : The measurements of a partially saturated sample.

	Initial condition	After 24 hours
The weight of sand, W_s (g)	160	160
The weight of water, W_w (g)	50	50
The height of the sample, H_s (mm)	28.2	29.5
Free water level, H_{fw} (mm)	30.3	32.7
Diameter, D (mm)	68.3	68.3
Degree of saturation, S (%)	95.9	76.7
Relative density, D_r (%)	55.9	31.2

3.4 Testing Procedure

The testing process can be broken down into three general stages: saturation, consolidation and cyclic simple shear. Each of these stages was discussed in the following sections.

3.4.1 Saturation

The testing program started with a saturation stage. The stage aimed to saturate the sample. The degree of saturation can be controlled by B-check (Skempton) parameters.

The saturation procedure was followed: B-check 1, Cell Pressure Ramp (CPR) and B-check 2. The initial degree of saturation can be estimated with B-check 1. As expected, B-check 1 had a lower value than 0.95 because the sample was not completely saturated. However, B-check 1 values were not considered for partially saturated samples. During the preparation process of partially saturated samples, free water was collected between membrane and sand particles. When cell pressure was applied, free water and water in the voids were compressed at the same time. Because of this reason, B-check 1 measured higher than it should be. The results were not considered reliable. The degree of saturation could be predicted by B-check 2. Figure 3.9 showed that the result of B-check 2 stage of a partially saturated test.

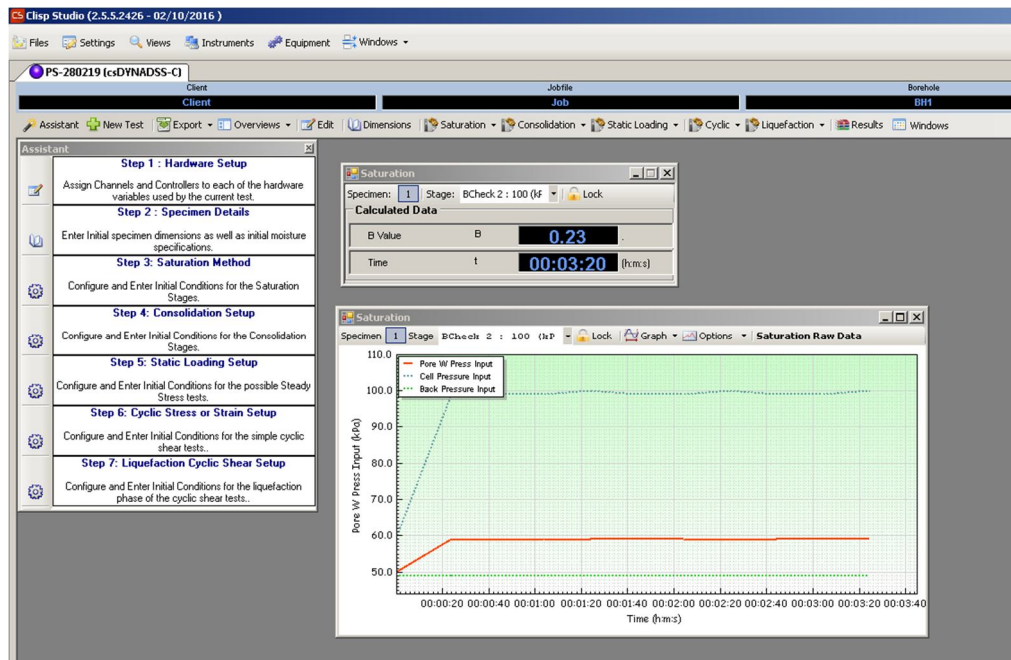


Figure 3.9 : B-check 2 stage of a partially saturated test.

The second step of saturation was the ramping of cell pressure (CPR). The back pressure valve was opened and both back and cell pressure started to rise under effective cell pressure of 10 kPa. The saturation of a sample was improved with increasing back pressure. CPR stage was performed for fully saturated samples under higher cell and back pressures. The cell pressure was increased around 450 kPa and to kept effective cell pressure of 10 kPa, back pressure was increased to 440 kPa. Higher back pressure dissipated the air in the sample and hereby degree of saturation was increased to almost 95-100 %. The degree of saturation was assured using B-check stages. CPR and B-check stages were repeated more than once when necessary.

Unexpectedly, upon cell pressure, the partially saturated samples were compressed laterally due to their loose form. The contraction affected the diameter of the sample. The change in the sample diameter was measured from high quality digital photos (Figure 3.10). Even though the sample diameter visibly changed after B-check 1 stage, the acrylic cell and water prevented the actual measurement. Therefore, the bottom cap length was measured out of the cell and selected as a reference length to get proportion inside the cell with water (Figure 3.11). The relative density was calculated with the rearranged diameter. The cell pressure did not have a major impact on fully saturated samples. The contraction in these samples was negligible low. The calculations were explained in details in Appendix A.

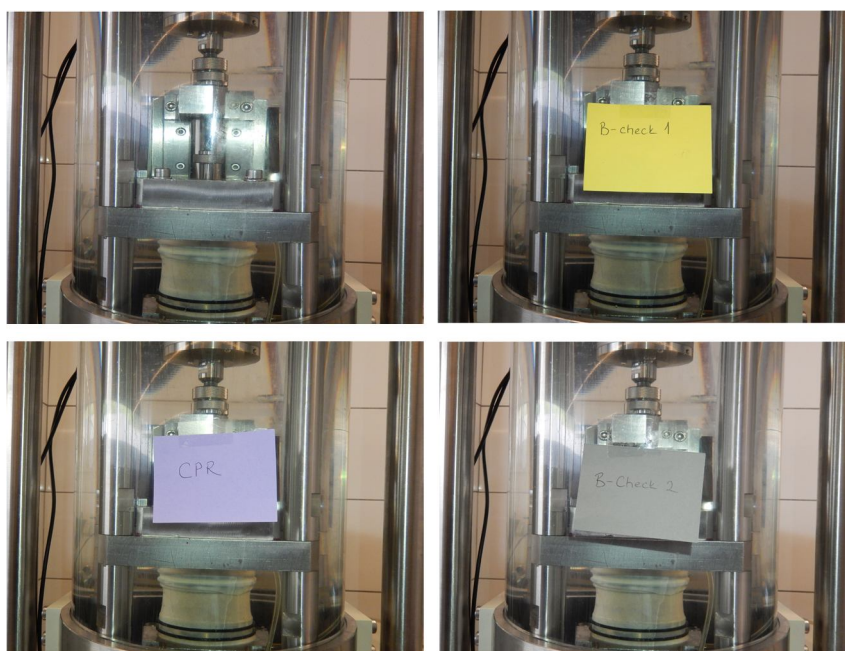


Figure 3.10 : Partially saturated samples pictures during saturation stages.

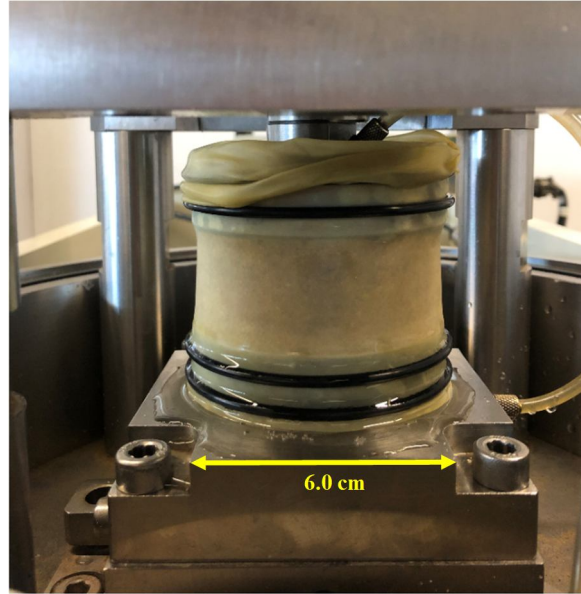


Figure 3.11 : The reference length for measurement of the sample diameter.

Additionally, during the CPR stage, controlling the degree of saturation of the partially saturated samples was difficult in practice. In some experiments, CPR was skipped and the consolidation stage was started. As a result, pore water pressure could not be obtained and the liquefaction stage could not be run. Therefore, CPR was performed in partially saturated samples under lower cell and back pressures to avoid full saturation. Since the samples were not fully saturated, the B-check value of 0.95 was not sought. The relation between B-check 2 value and the degree of saturation was investigated on partially saturated samples (Figure 3.12).

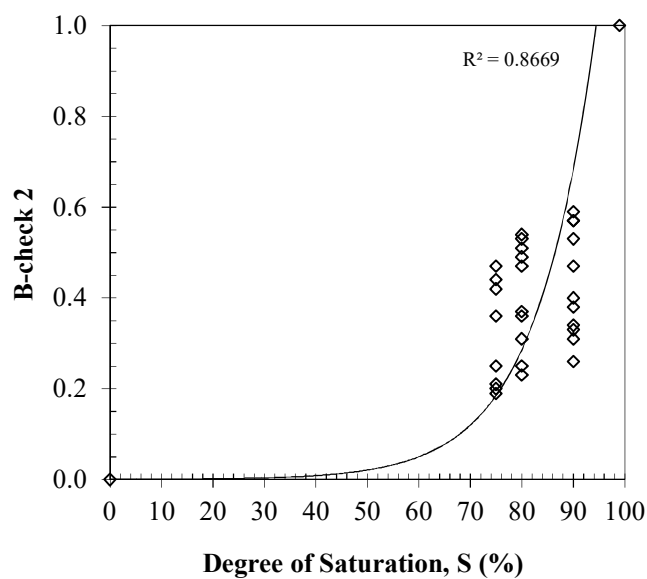


Figure 3.12 : The variation between B-check 2 and degree of saturation.

Upon examination of the CPR stage output data, it was noticed that the back volume of hydraulic APC increased before the stage and this increment was not recorded in the Clisp program. The reason for this increment was the water escaping from the specimen in response to the opened back pressure valve. The volume change in the specimen and hydraulic APC was corrected on the calculation of the degree of saturation (Table 3.4). The calculations were explained in details in Appendix A.

Table 3.4 : The measurements of a partially saturated sample during saturation.

	Before saturation	B-check 1	CPR
APCH Initial back volume (cm ³)	163.867	163.867	165.067
APCH Back volume change (cm ³)	0	0	+1.2*
Volume change (cm ³)	0	0	-6.9**
Total volume change (cm ³)	0	0	-5.7**
Degree of saturation, S (%)	76.7	78.1	90.4
Diameter, D (mm)	68.3	68.15	68.15
Vertical displacement, LVDT (mm)	0	0.102	0.025
Relative density, D _r (%)	31.2	35.6	36.1

* (+) corresponded to water that escaping from the specimen to APCH.

** (-) corresponded to water that went into the specimen.

3.4.2 Consolidation

The consolidation procedure was the second stage of the experiment. The software offered three following options as: isotropic, anisotropic and K_0 consolidation. In this study, isotropic consolidation was performed. To consolidate the specimen, the cell pressure was increased until the difference between the cell and back pressures equals the required effective pressure. For partially saturated specimens, the same back and effective pressures were chosen such as 50-50 kPa or 100-100 kPa. Otherwise, the degree of saturation of the sample would raise due to higher back pressures. When the back pressure was 50 kPa, cell pressure of 100 kPa applied to kept the effective pressure of 50 kPa. If the effective pressure was 100 kPa, the cell pressure was increased to 200 kPa. When the effective pressure was established, the consolidation started and the volume of water in the sample decreased due to dissipation. Since it was known that the consolidation was not a major impact in sands, the consolidation duration was kept for a short time. When pore water pressure dissipation exceeded

almost 100 %, the volume change stabilized at the end of the stage. The final relative density and degree of saturation were calculated after consolidation (Table 3.5).

Table 3.5 : The measurements of a partially saturated sample during consolidation.

	Before test	CPR	Consolidation
APCH Final Back volume (cm ³)	163.867	158.167	159.367
Volume change (cm ³)	0	0	+1.2*
Degree of saturation, S (%)	76.7	90.4	87.9
Diameter, D (mm)	68.3	68.15	68.15
Vertical displacement, LVDT (mm)	0	0.127	0.011
Relative density, D _r (%)	31.2	36.1	36.3

* (+) corresponded to water that went out from the specimen.

3.4.3 Undrained strain-controlled cyclic simple shear

Undrained cyclic tests were carried out strain-controlled with a selected sinusoidal waveform. At this stage, vertical control maintained constant in the vertical position. Applied horizontal displacement and undrained conditions caused excess pore water pressure generation and thus vertical effective stress reduction. Excess pore water pressure ratio was obtained in a range of shear strain amplitudes (0.2-0.4 %), vertical effective stresses and relative densities for fully and partially saturated samples. Samples were subjected to 50 cycles of loading at 1 Hz.

3.4.4 Drained strain-controlled cyclic simple shear

The strain-controlled drained tests were performed after the completion of undrained cyclic tests. The tests were conducted under the vertical effective stress of 50 kPa. The shear strain amplitudes were 0.2 % to 0.4 %. During the loading phase, volumetric strains were recorded. The cell pressure was expected to remain constant, however, it decreased as the number of loading cycles increased. Due to unreliable results, the experiments were repeated with an arrangement of software settings as recommended by VJ-Tech. The details were explained in Chapter 4.



4. DYNAMIC SIMPLE SHEAR TEST RESULTS

The experimental program aimed to find the relationship between the excess pore water pressure generated in undrained conditions and the volumetric strains occurring in drained conditions in partially and fully saturated sand samples.

First attempt in the testing program was to provide dissipation of excess pore water pressures generated in the samples during undrained conditions and evaluate the post-cyclic settlements. Since the DSS-C test device limited dissipation of excess pore water pressures up to $(r_u) = 0.67$, post-cyclic settlements couldn't be obtained by dissipation in fully saturated specimens. In partially saturated sands, the vertical load cell connection was separated when the dissipation phase started. The problem persisted in the following experiments. Therefore, the dissipation experiments were abandoned to avoid damaging the vertical actuator. The drained strain-controlled tests were preferred in case of dissipation tests and the undrained and drained dynamic simple shear tests in partially and fully saturated sand samples were conducted separately.

4.1 Undrained Cyclic Simple Shear Tests

The desired degree of saturation and vertical effective stress were achieved at saturation and consolidation stages. Following these established parameters, undrained cyclic simple shear tests were performed on fully and partially saturated samples. The purpose of this stage was to examine the excess pore water pressure ratio that occurred under conditions where drainage was not allowed during cyclic simple shear depending on the degree of saturation of the specimen. All specimens were isotropically consolidated under the vertical effective stress of 50 kPa or 100 kPa before cyclic simple shear. Anisotropic consolidation setting with $K_0=1.0$ on fully and partially saturated samples was also tested. As expected, both isotropic and anisotropic consolidation setting with $K_0=1.0$ yielded similar results due to the maintained isotropic state inside the cell. These results verified the use of the $K_0=1.0$ settings in drained cyclic simple shear tests. The reasons for this setting would be explained in

detail in section 4.2. The effect of shear strain amplitudes and vertical effective stresses were investigated. Samples were subjected to 50 cycles of loading at a frequency of 1 Hz. At this stage, the excess pore water pressure generation depending on the degrees of saturation and relative densities were measured.

4.1.1 Undrained strain-controlled tests on fully saturated sand samples

Undrained cyclic tests were performed using uniform cycles of shear strain to investigate the effect of the relative density (D_r), vertical effective stress (σ'_v) and shear strain amplitude (γ) on the generation of excess pore water pressure. The specimens were consolidated under vertical effective stress of 50 kPa or 100 kPa. Then, cyclic simple shear tests were performed at shear strain levels ranged from 0.2 % to 0.4 % and at 1 Hz. The tests performed along with their testing conditions were summarized in Table 4.1. The excess pore water pressure ratio (r_u) increased with increasing shear strain amplitudes. On the contrary, it decreased with higher relative density and vertical effective stresses. Seed and Idriss (1982) stated that 15 cycles of strain correspond to a magnitude of 7.5 earthquakes. Thus, the number of cycles of 15, 50 and 100 were taken into consideration in the experiments.

Table 4.1 : The undrained strain-controlled tests of fully saturated sand samples.

Test No	S (%)	D_r (%)	σ'_v (kPa)	γ (%)	r_u @N=15	r_u @N=50	r_u @N=100
1	100	30-35	50	0.4	0.70	0.93	1.00
2	100	30-35	50	0.35	0.60	1.00	1.01
3	100	30-35	50	0.3	0.48	0.85	1.00
4	100	30-35	50	0.2	0.48	0.78	0.90
5	100	40-45	50	0.35	0.39	0.79	1.01
6	100	40-45	50	0.3	0.38	0.72	1.03
7	100	40-45	50	0.2	0.63	0.84	0.89
8	100	30-35	100	0.4	0.35	0.82	0.98
9	100	30-35	100	0.35	0.32	0.84	1.00
10	100	30-35	100	0.3	0.32	0.60	1.04
11	100	30-35	100	0.2	0.29	0.52	0.76

The cyclic shear strain was applied in a sinusoidal waveform. The excess pore water pressure ratio was more than 0.9. Representative results from an undrained strain-controlled test were demonstrated in Figure 4.1. When the shear stress-shear strain hysteresis loop was examined, it was obvious that the induced shear stress decreased as the number of cycles increased and liquefaction was approached at the 50th cycle.

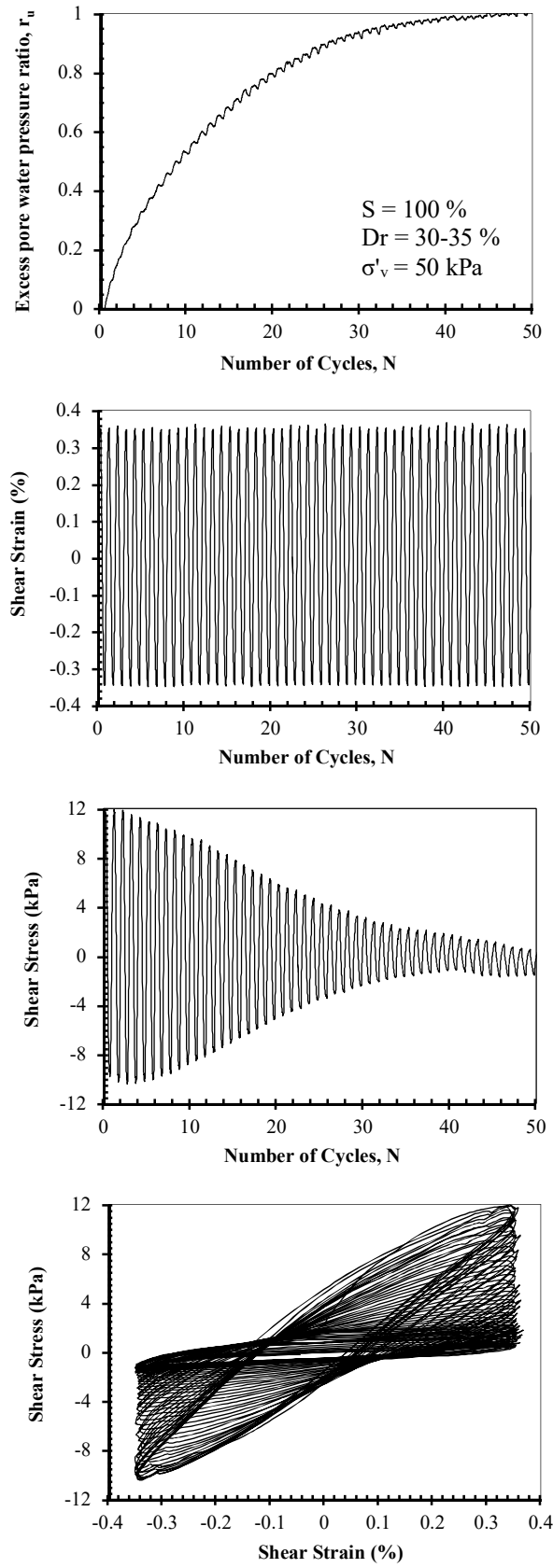


Figure 4.1 : The typical results of undrained strain-controlled tests in a fully saturated sample (a) r_u vs. N , (b) γ vs. N , (c) Shear stress vs. N and (d) Shear stress-shear strain hysteresis loop.

The effect of vertical effective stress and shear strain amplitudes on the excess pore water pressure ratios were plotted in Figure 4.2. The excess pore water generation was recorded in terms of the number of cycles. The number of cycles required for liquefaction decreased when testes under the shear strain amplitude of 0.35 %, whereas it increased under the shear strain amplitude of 0.3 %. Additionally, the rate of pore water pressure generation was higher with decreasing vertical effective stress. At shear strain of 0.35 %, liquefaction occurred at $r_u = 1.0$ under vertical effective stresses of 50 kPa and 100 kPa at the 44th and 57th cycles, respectively.

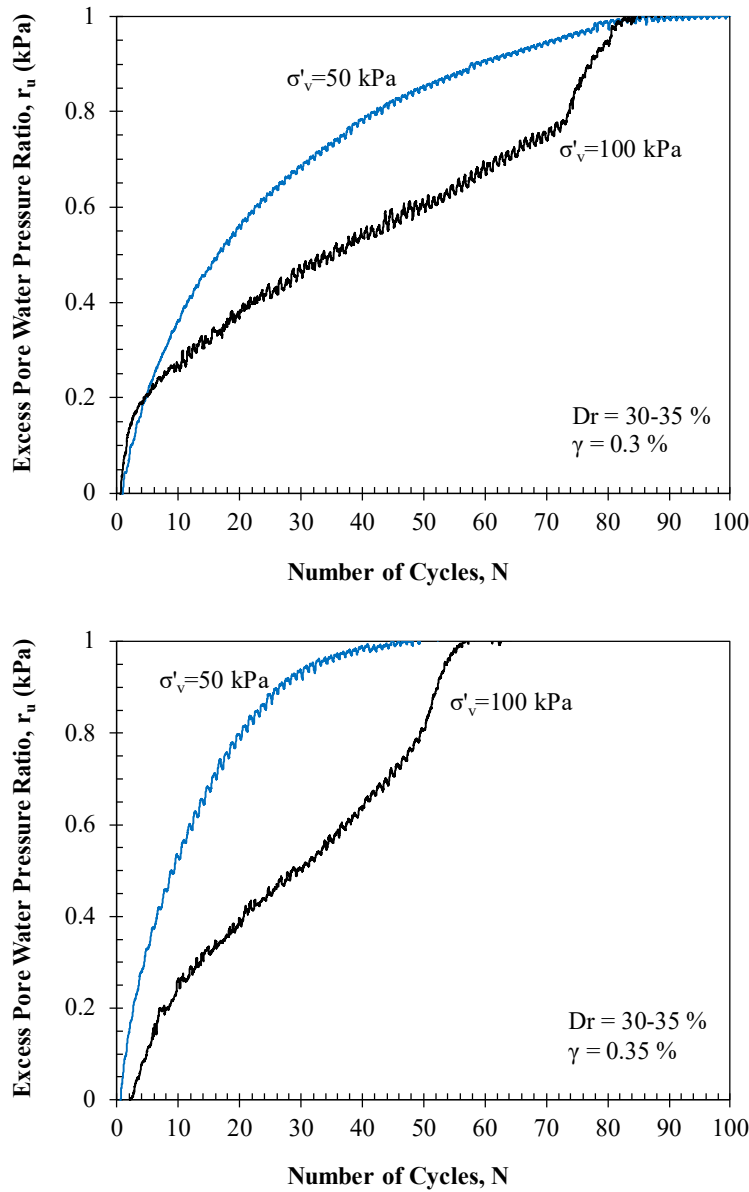


Figure 4.2 : The undrained strain-controlled tests in fully saturated sample under 50 kPa and 100 kPa vertical effective stresses at (a) shear strain amplitude of 0.3 %, (b) shear strain amplitude of 0.35 %.

Figure 4.3 showed the effect of relative density on excess pore water pressure ratio under the vertical effective stress of 50 kPa. As the density decreased, larger excess pore pressures were developed. The rate of excess pore water pressure generation was higher in the looser sand sample. In denser sample, the rate of excess pore water pressure generation was slowly within 30 loading cycles, and then it increased rapidly. It can be caused due to the deformation of the sample with the increasing number of cycles.

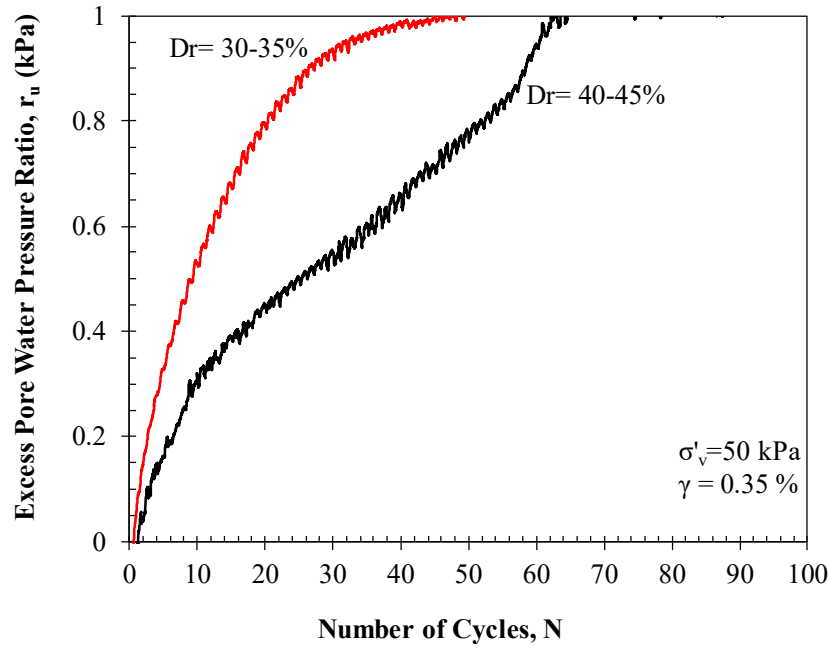


Figure 4.3 : The excess pore pressure ratio versus number of cycles at different relative densities.

4.1.2 Undrained strain-controlled tests on partially saturated sand samples

A series of undrained strain-controlled tests were performed for partially saturated sand samples. As explained in the previous sections, the samples were prepared at degrees of saturation varying in the range of 75 % to 90 % by using the IPS method. The undrained test results of partially saturated sand samples were tabulated in Table 4.2 and Figure 4.4.

The specimens were consolidated under vertical effective stress of 50 kPa before undrained strain-controlled stages. The tests were completed after 50 cycles of loading. The excess pore water pressures did not approach 100 % of the vertical effective stress. The largest increment in excess pore water pressure occurred in the loosest sample.

The degree of saturation of the sample was 90 % and it was tested under shear strain amplitude of 0.4 %. The lowest excess pore water pressure generation was obtained under shear strain amplitude of 0.2 %. The sample had a degree of saturation of 75 %. The test results revealed that the reduction in the degree of saturation led to a reduction in the excess pore water pressure. It was also observed that at the same degree of saturation, as the soil got denser more number of cycles was required to higher excess pore water ratio. The shear strain amplitude was also an effect on the excess pore water pressure generation.

Table 4.2 : The undrained strain-controlled tests of partially saturated sand samples.

Test No	S (%)	Dr (%)	σ'_v (kPa)	u_o (kPa)	γ (%)	r_u @N=15	r_u @N=25	r_u @N=50
1	90	30-35	50	50	0.4	0.25	0.30	0.44
2	90	30-35	50	50	0.4	0.18	0.24	0.35
3	80	30-35	50	50	0.4	0.20	0.23	0.30
4	80	30-35	50	50	0.4	0.10	0.13	0.17
5	75	30-35	50	50	0.4	0.12	0.15	0.18
6	90	40-45	50	50	0.4	0.23	0.28	0.34
7	80	40-45	50	50	0.4	0.11	0.15	0.23
8	80	40-45	50	50	0.4	0.16	0.19	0.28
9	75	40-45	50	50	0.4	0.09	0.11	0.16
10	75	40-45	50	50	0.4	0.12	0.14	0.18
11	90	30-35	50	50	0.3	0.11	0.14	0.25
12	90	30-35	50	50	0.3	0.10	0.15	0.28
13	80	30-35	50	50	0.3	0.10	0.12	0.16
14	80	30-35	50	50	0.3	0.06	0.10	0.13
15	80	40-45	50	50	0.3	0.07	0.09	0.11
16	75	40-45	50	50	0.3	0.10	0.11	0.12
17	75	30-35	50	50	0.3	0.05	0.07	0.09
18	90	30-35	50	50	0.2	0.06	0.11	0.15
19	80	30-35	50	50	0.2	0.08	0.09	0.12
20	80	30-35	50	50	0.2	0.07	0.09	0.14
21	75	40-45	50	50	0.2	0.04	0.05	0.06
22	75	30-35	50	50	0.2	0.06	0.08	0.10

As can be seen in the figures, the degradation of the shear stress-shear strain hysteresis loop in the partially saturated sand sample (S=90 %) was not as clear as the fully saturated sand sample. The decrement in induced shear stress was not significant as the number of cycles increased. Since the excess pore water pressure ratio was lower than 1.0, a progressive reduction on shear stresses was not observed. In other words, developed excess pore water pressure did not cause a significant loss of shear strength.

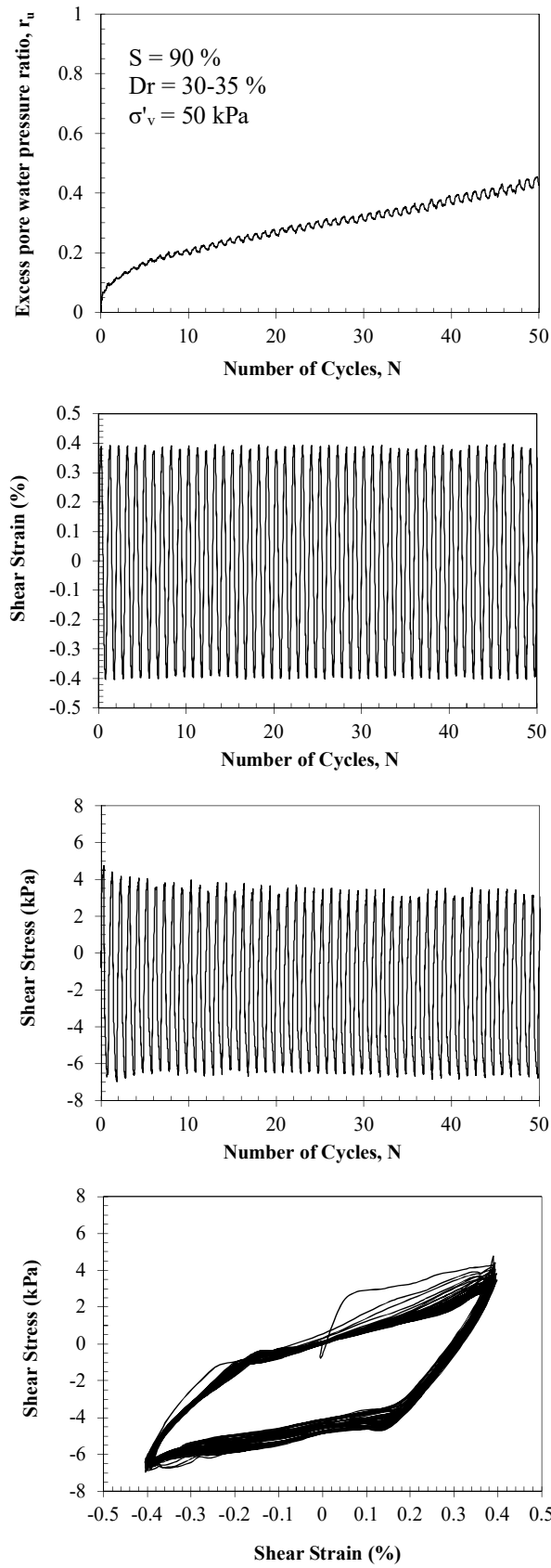


Figure 4.4 : The typical results of undrained strain-controlled tests in a partially saturated sample (a) r_u vs. N , (b) γ vs N , (c)Shear stress vs. N and (d)Shear stress-shear strain hysteresis loop.

4.2 Drained Cyclic Simple Shear Tests

In the second part of the study, drained cyclic simple shear tests were performed to evaluate the volumetric strains that would occur in the specimens after the dissipation of excess pore water pressures. The stages of tests were carried out in the same way as undrained experiments except for the cyclic simple shear stage. Unlike undrained cyclic simple shear tests, drainage valves were opened, and then cyclic simple shear was applied.

During the drained strain-controlled test stage, the vertical effective stress was expected to remain steady, yet it reduced. Thus, these initial tests were considered unreliable. These unexpected results were reported to VJ-Tech, and we have been informed that it was due to Clisp Studio software error. In order to overcome this technical problem on the software, the company recommended the use of anisotropic consolidation setting with $K_0=1.0$, without changing the stress conditions. First, the tests were conducted to understand whether anisotropic setting with $K_0=1$ would yield comparable results to undrained test results. Upon a thorough verification, the recommended software setting was applied to all drained cyclic simple shear tests on fully and partially saturated sand samples.

4.2.1 Drained strain-controlled tests on fully saturated sand samples

The drained strain-controlled cyclic simple shear tests were performed at different shear strains (γ), vertical effective stresses (σ'_v) and relative densities (D_r). The preliminary tests on fully saturated samples were performed without the software setting and the results were listed in Table 4.3. As it can be seen in the table, the relative density and the vertical effective stress were a significant role on the volumetric strains for fully saturated sand samples.

Unexpectedly, the vertical effective stresses decreased during the tests. The vertical effective stress was expected to remain steady and hence these results were considered unreliable. The volumetric strain and the reduction in vertical effective stresses vs. the number of cycles were plotted in Figure 4.5. It can be seen in the figure, the vertical effective stress decreased from 50 kPa to 36 kPa as the number of cycles increased. This technical problem on the software also affected the volumetric strains and thus the experiments were repeated with the recommended settings.

Table 4.3 : The unreliable results of fully saturated sand samples obtained prior to using the recommended software settings.

Test No	S (%)	Dr (%)	σ'_v (kPa)	γ (%)	ε_{vd} (%) @N=15	ε_{vd} (%) @N=25	ε_{vd} (%) @N=50
1	100	30-35	50	0.4	0.42	0.55	0.60
2	100	30-35	50	0.3	0.48	0.62	0.77
3	100	30-35	50	0.2	0.27	0.31	0.38
4	100	40-45	100	0.4	0.02	0.07	0.10
5	100	40-45	100	0.3	0.03	0.09	0.11
6	100	40-45	100	0.2	0.02	0.09	0.15

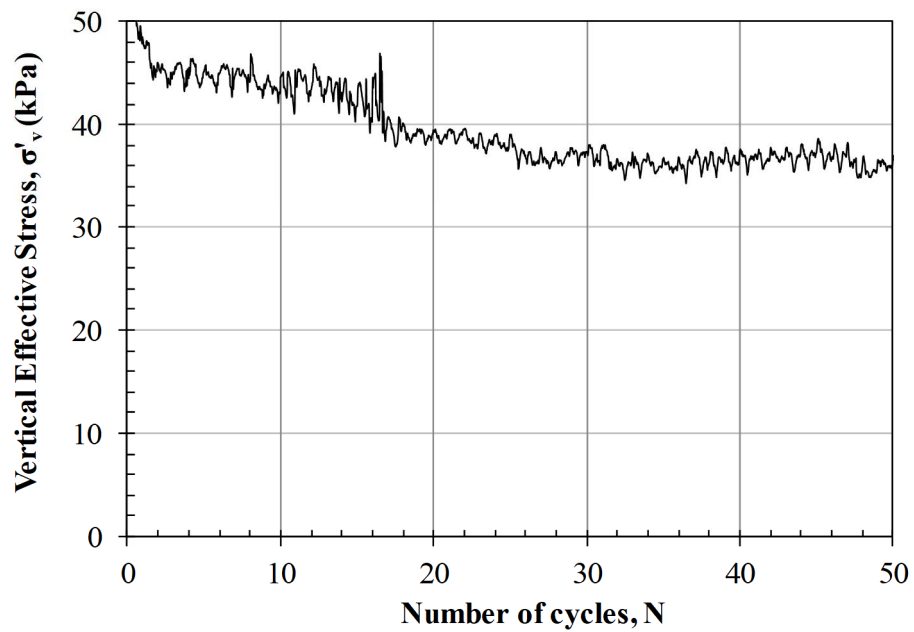
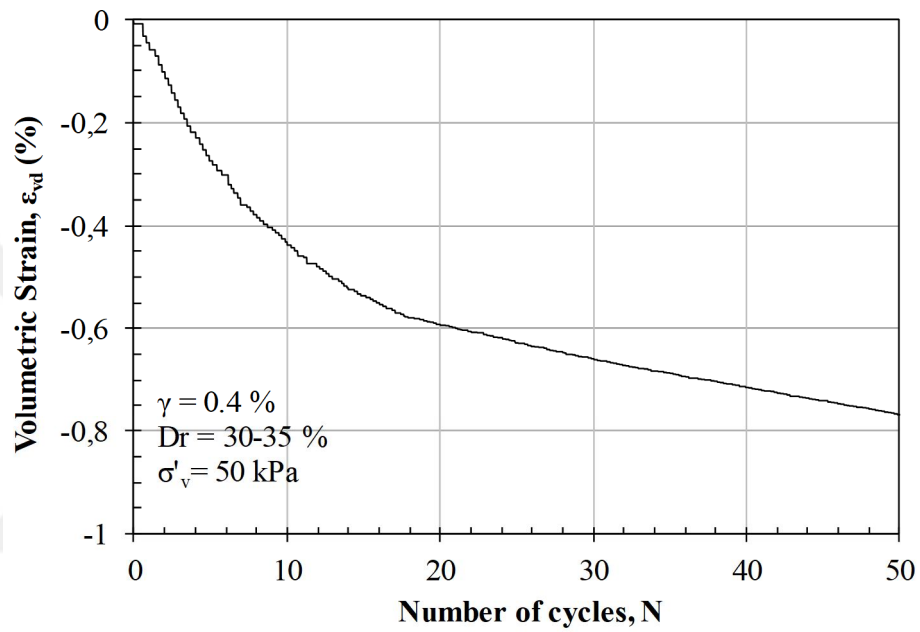


Figure 4.5 : The result of a drained strain-controlled test on fully saturated sand specimen without the recommended $K_0=1.0$ setting.

The second set of the drained strain-controlled cyclic simple shear tests was performed with the software setting. With this approach, the problem of varying vertical effective stress was overcome. However, another problem was faced during these tests. The vertical load ram could not be maintained in its position in fully saturated samples and samples collapsed due to sudden increases and fluctuations in vertical total stress (Figure 4.7). To prevent collapsing, various approaches were taken. First, the sensitivity calibration of the vertical load control of DSC was decreased from 4.0 to 1.0. Second, the frequency and K_0 values decreased from 1 to 0.1 and to 0.5, respectively. However none of these approaches overcame the collapsing problem. The collapsed specimens can be seen in Figure 4.6. Due to these problems, experiments could not be performed on fully saturated samples.



Figure 4.6 : Samples that collapsed during the experiment with the recommended $K_0=1.0$ setting.

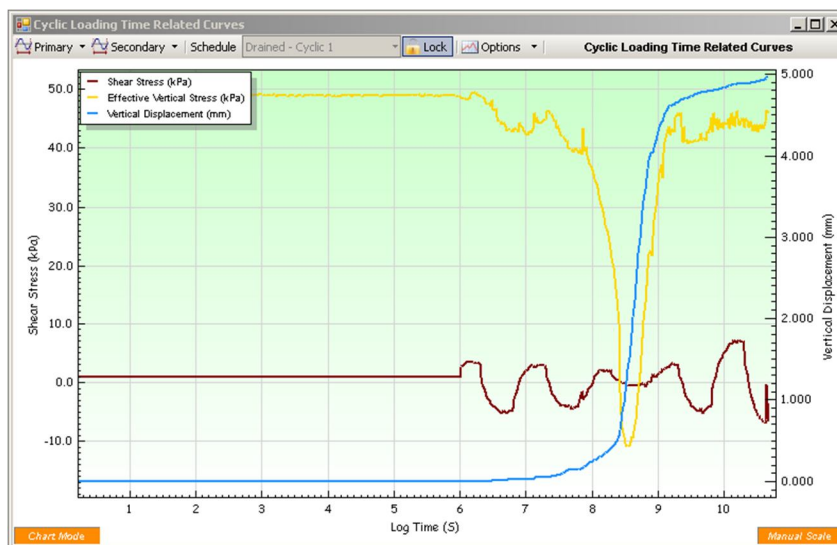


Figure 4.7 : Sudden changes of vertical effective stress in drained strain-controlled test on fully saturated sand specimen with the recommended $K_0=1.0$ setting.

In the fully saturated sand specimens, vertical total stress significantly dropped as shear stress started changing (Figure 4.7). The vertical total stress was expected to remain constant, thus to avoid the observed change, the vertical motor moved down which can be seen as the vertical displacement curve. Then, the vertical motor tried to increase the vertical stress to the initial value of 50 kPa. This could indicate that the sample collapsed at the moment the cyclic simple shear started.

4.2.2 Drained strain-controlled tests on partially saturated sand samples

The initial testing series on the partially saturated samples were performed without the software setting and the results were not considered reliable as fully saturated samples. The tests were conducted under vertical effective stress of 50 kPa and degrees of saturation ranging from 75 % to 90 %. The results of partially saturated samples without the recommended software settings were listed in Table 4.4. As it can be seen in the table, the volumetric strains did not affect significantly with the different degree of saturation and relative density. However, the results were considered unreliable due to the technical problem on the software. It was thought that, reduction of the vertical effective stress affected the samples and thus the volumetric strains obtained lower than the expected values. The volumetric strains and vertical effective stresses vs the number of cycles were illustrated in Figure 4.8.

Table 4.4 : The unreliable results of partially saturated sand samples obtained prior to using the recommended software settings.

Test No	S (%)	Dr (%)	σ'_v (kPa)	γ (%)	ϵ_{vd} (%) @N=15	ϵ_{vd} (%) @N=25	ϵ_{vd} (%) @N=50
1	90	30-35	50	0.4	0.100	0.163	0.272
2	80	30-35	50	0.4	0.100	0.146	0.296
3	80	30-35	50	0.4	0.080	0.149	0.264
4	75	30-35	50	0.4	0.090	0.110	0.229
5	90	40-45	50	0.4	0.080	0.154	0.275
6	80	40-45	50	0.4	0.080	0.132	0.202
7	80	40-45	50	0.4	0.110	0.177	0.277
8	75	40-45	50	0.4	0.080	0.102	0.127

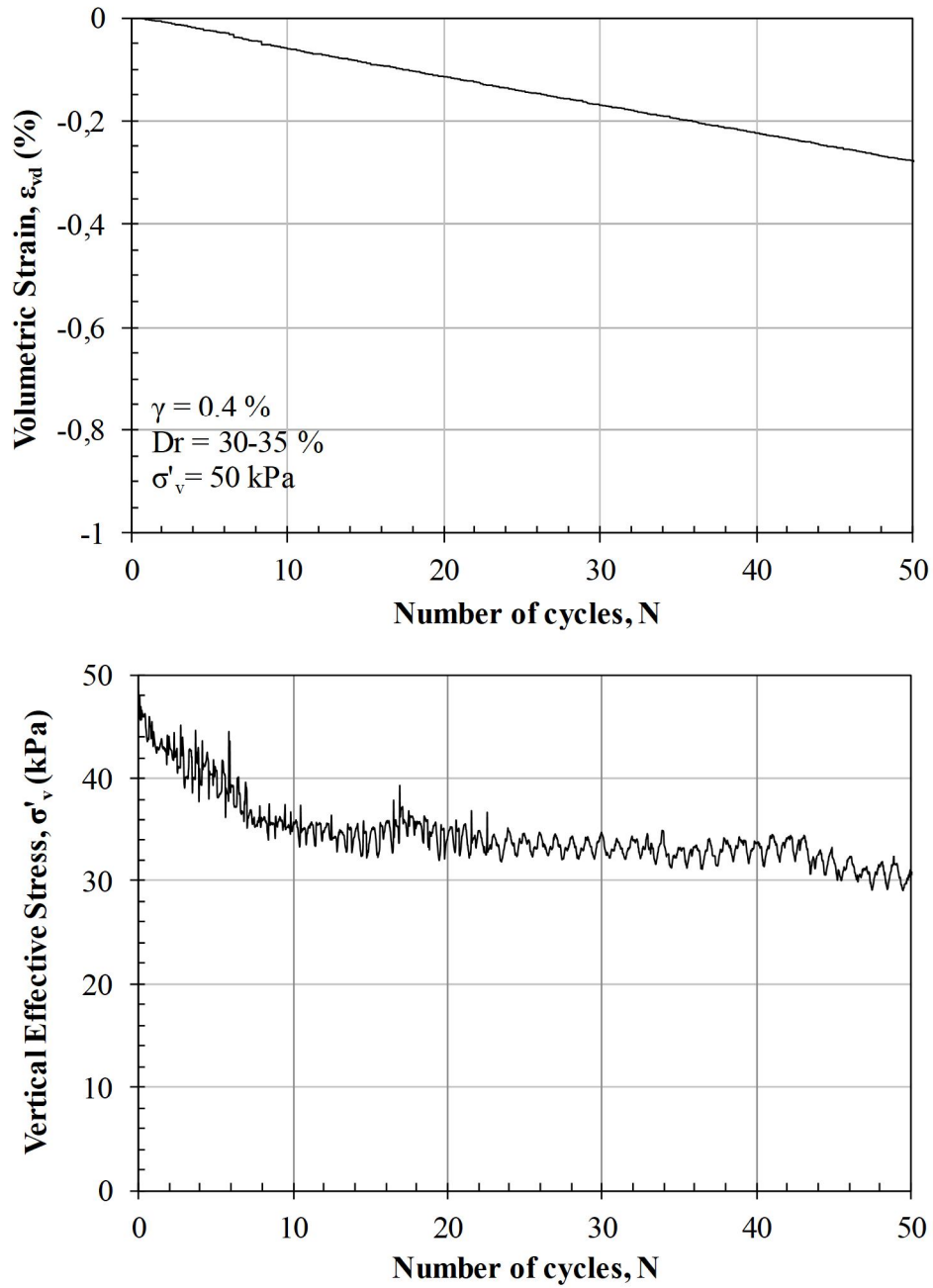


Figure 4.8 : The result of a drained strain-controlled test on partially saturated sand specimen without the recommended software setting.

The second testing series were performed with the software setting. Similar to the fully saturated samples, partially saturated samples collapsed after 30 cycles. Figure 4.9 showed that shear stress, vertical total stress and vertical displacement vs the number of cycles on a trial partially saturated sample. Vertical total stress preserved its initial value (100 kPa) for the first 30 seconds and then started fluctuating. Until $t=30\text{s}$, the sample height decreased 0.2 mm as expected. At $t=30\text{s}$, the vertical controller lost control due to the stiffened sample and the sample collapsed.

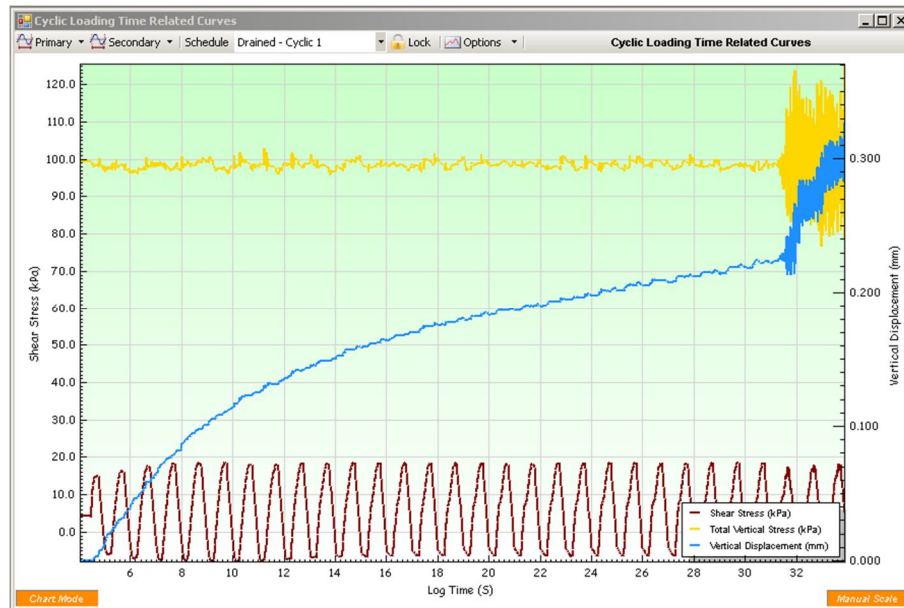


Figure 4.9 : Sudden changes of vertical effective stress in isotropic consolidated partially saturated sand specimen during drained strain-controlled test.

The tests were repeated by reducing the sensitivity calibration of the vertical load control of DSC, and results were shown in Figure 4.10. The recommended settings worked fine for partially saturated sand specimens and tests were performed under vertical effective stress of 50 kPa. The volumetric strain was obtained in a different range of shear strain amplitudes (0.2-0.4 %) and relative densities. The results of the tests with testing conditions were tabulated in Table 4.5.

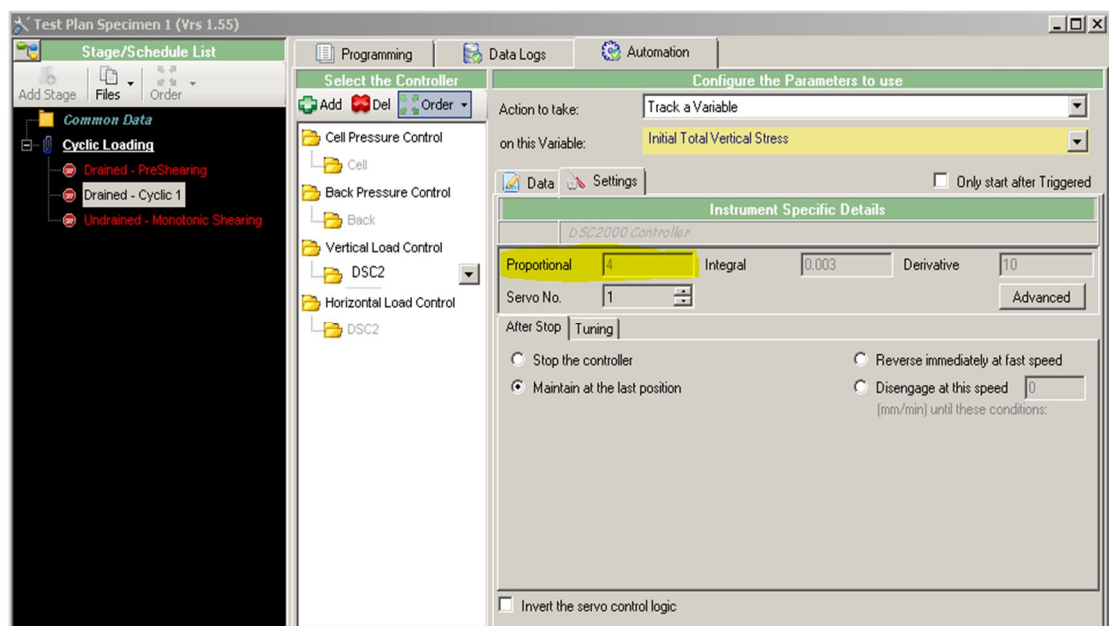


Figure 4.10 : The sensitivity calibration set up from 4 to 1 before the test.

Table 4.5 : The test results of partially saturated sand samples obtained after using the recommended $K_0=1.0$ and calibration settings.

Test No	S (%)	Dr (%)	σ'_v (kPa)	u_0 (kPa)	γ (%)	ϵ_{vd} (%) @N=15	ϵ_{vd} (%) @N=25	ϵ_{vd} (%) @N=50
1	90	30-35	50	50	0.4	0.77	1.79	3.18
2	80	30-35	50	50	0.4	0.92	1.16	1.75
3	80	30-35	50	50	0.4	0.25	0.61	1.34
4	75	30-35	50	50	0.4	0.56	0.69	0.97
5	90	40-45	50	50	0.4	0.43	0.98	2.58
6	80	40-45	50	50	0.4	0.78	0.88	1.31
7	80	40-45	50	50	0.4	0.38	0.63	1.15
8	75	40-45	50	50	0.4	0.18	0.30	0.47
9	75	40-45	50	50	0.4	0.28	0.38	0.55
10	90	30-35	50	50	0.3	0.51	0.82	2.47
11	80	30-35	50	50	0.3	0.12	0.40	1.23
12	80	30-35	50	50	0.3	0.25	0.44	1.08
13	75	30-35	50	50	0.3	0.46	0.65	0.93
14	90	40-45	50	50	0.3	0.50	1.08	2.20
15	80	40-45	50	50	0.3	0.40	0.56	1.12
16	75	40-45	50	50	0.3	0.37	0.52	0.74
17	90	30-35	50	50	0.2	0.68	1.17	1.98
18	80	30-35	50	50	0.2	0.18	0.55	1.01
19	80	30-35	50	50	0.2	0.09	0.14	0.48
20	75	30-35	50	50	0.2	0.21	0.33	0.54
21	75	40-45	50	50	0.2	0.09	0.15	0.29

All in all, the volumetric strain increased as both the cyclic shear strain amplitudes and the number of loading cycles increased. Additionally, as the degree of saturation increased, the greater volumetric deformation was observed.

The results of partially saturated samples were compared to the different correlations with fully saturated samples found by Finn et al. (1976) and Byrne (1991). The correlations of the researchers were discussed in Chapter 2. Briefly, Finn et al. (1976) based the volumetric strain correlation formulas for the relative density of 45%. To convert relative density, potential volumetric strain values ($\Delta\epsilon_{vd}$) were reduced by using the coefficient of R (Eqn. 4.1) in denser samples. Since the coefficient was not

subjected to a lower relative density of 45 %, yet it was calculated with the same formula (Eqn. 4.2). The calculated volumetric strains according to the equation by Finn et al. (1976) was in a good fit with the Byrne correlation for a lower relative density of 45 %.

$$(\Delta\epsilon_{vd})_{Dr} = R(\Delta\epsilon_{vd})_{45} \quad (4.1)$$

$$R = 0.00031(100 - Dr)^2 + 0.062(45 < Dr < 80) \quad (4.2)$$

The experimental results of volumetric strains on partially saturated samples and the results of Finn et al. (1976) and Byrne (1991) correlations on fully saturated samples were compared under different relative densities and shear strain amplitudes. The volumetric strains vs the number of cycles for relative density of 35 % and 45 % were plotted in Figure 4.11, and Figure 4.12. Experimental volumetric strains tended to reduce with decreasing degree of saturation and shear strain amplitudes.

The accumulation of the experimental volumetric strains was in a good agreement at $S=90$ % and shear strain amplitudes of 0.2-0.4 % with the Finn et al. (1976) and Byrne (1991) correlations. The results showed a lack of fit at a degree of saturation lower than 90 % as expected. Even though the correlations were in a good agreement at a relative density of 35 %, a lack of fit was observed at a relative density of 45 %. Generally, the experimental results for $S=90$ % followed a trend between Finn et al. (1976) and Byrne (1991) correlations as can be seen in Figure 4.11, and Figure 4.12. The lower volumetric strains were observed at a relative density of 45 % when compared to a relative density of 35 % at the same degree of saturation. It was clearly showed that, the degree of saturation and relative density was a significant role on the volumetric strains.

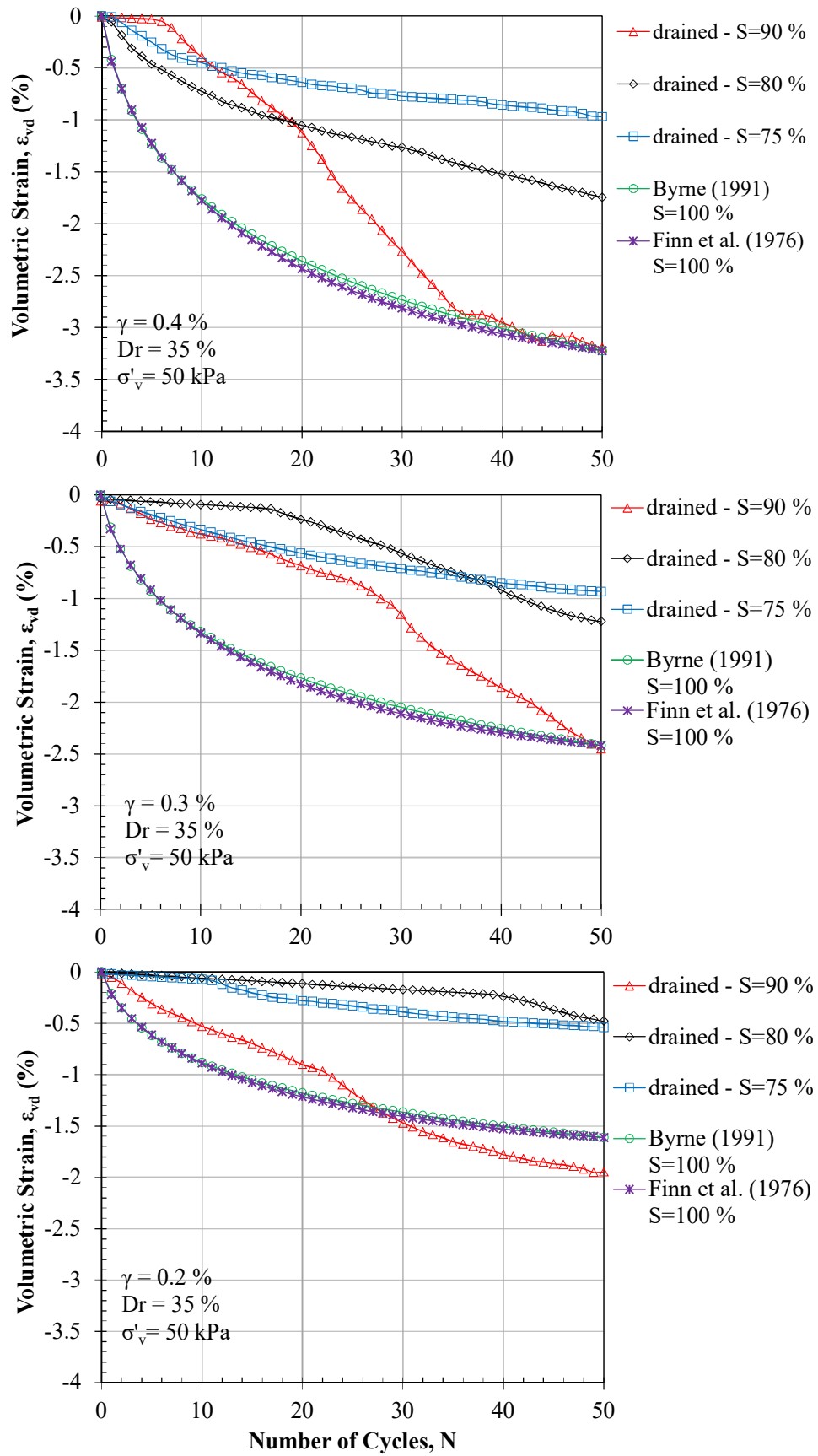


Figure 4.11 : Drained strain-controlled tests for $D_r = 35\%$, $S = 90-80-75\%$ under shear strain amplitude of (a) 0.4% , (b) 0.3% and (c) 0.2% .

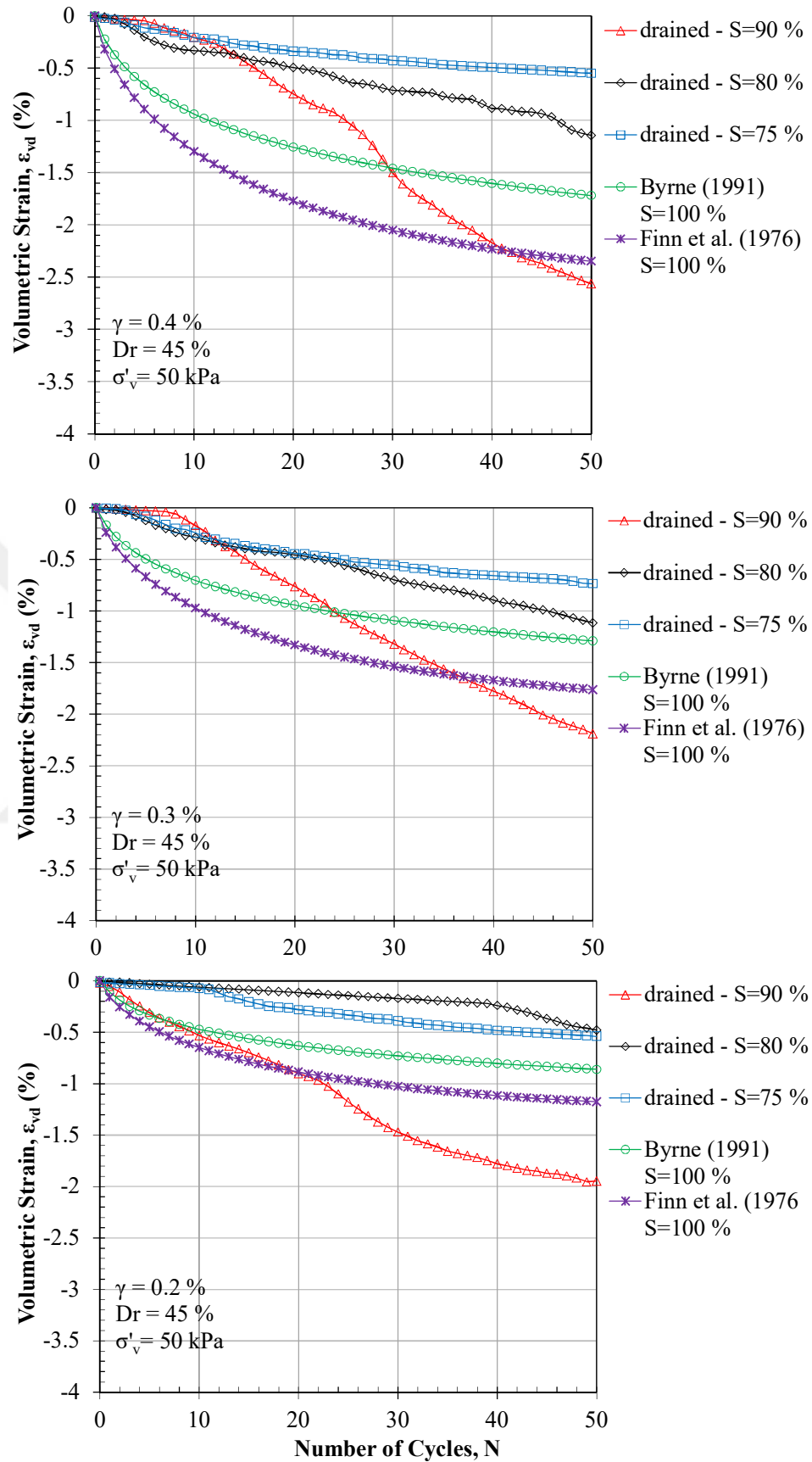


Figure 4.12 : Drained strain-controlled tests for $Dr = 45\%$, $S=90-80-75\%$ under shear strain amplitude of (a) 0.4% , (b) 0.3% and (c) 0.2% .

As a general trend, for both relative densities of 35 % and 45 %, the experimental results at $S=90$ % reflected that the volumetric strains were in a good agreement with correlated values. The volumetric strains of the partially saturated samples were smaller than the correlated values. As mentioned before, Finn et al. (1976) and Byrne (1991) correlation only established for fully saturated samples and hence needs further improvement for partially saturated samples.

Shear stresses and volumetric strains related to the shear strain amplitude of 0.4 %, 0.3 % and 0.2 % with $S = 90$ % were plotted in Figure 4.13-15. In the shear stress-shear strain hysteresis loop graphics of drained strain-controlled tests, it was observed that shear-induced volumetric strains can lead to hardening behavior. The shear stress increased as the number of cycles increased.

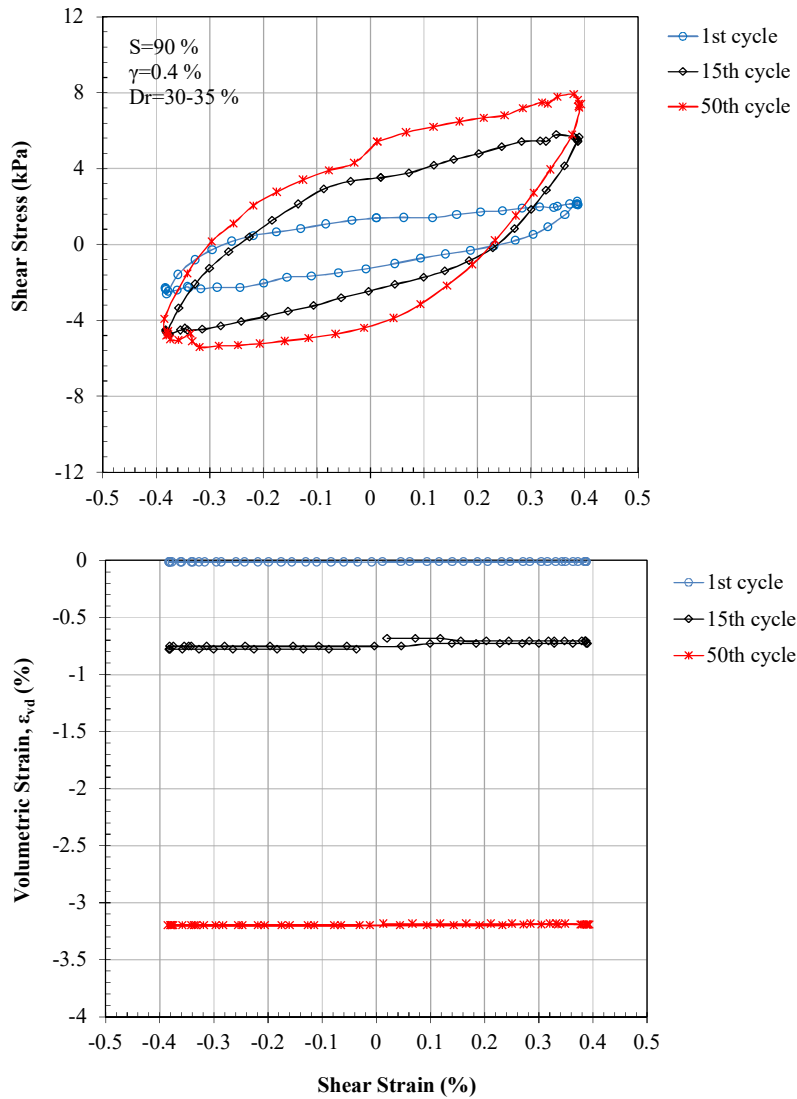


Figure 4.13 : Shear stress and volumetric strains related to cyclic shear strains with 0.4 % amplitude in a sample 90 % degree of saturation.

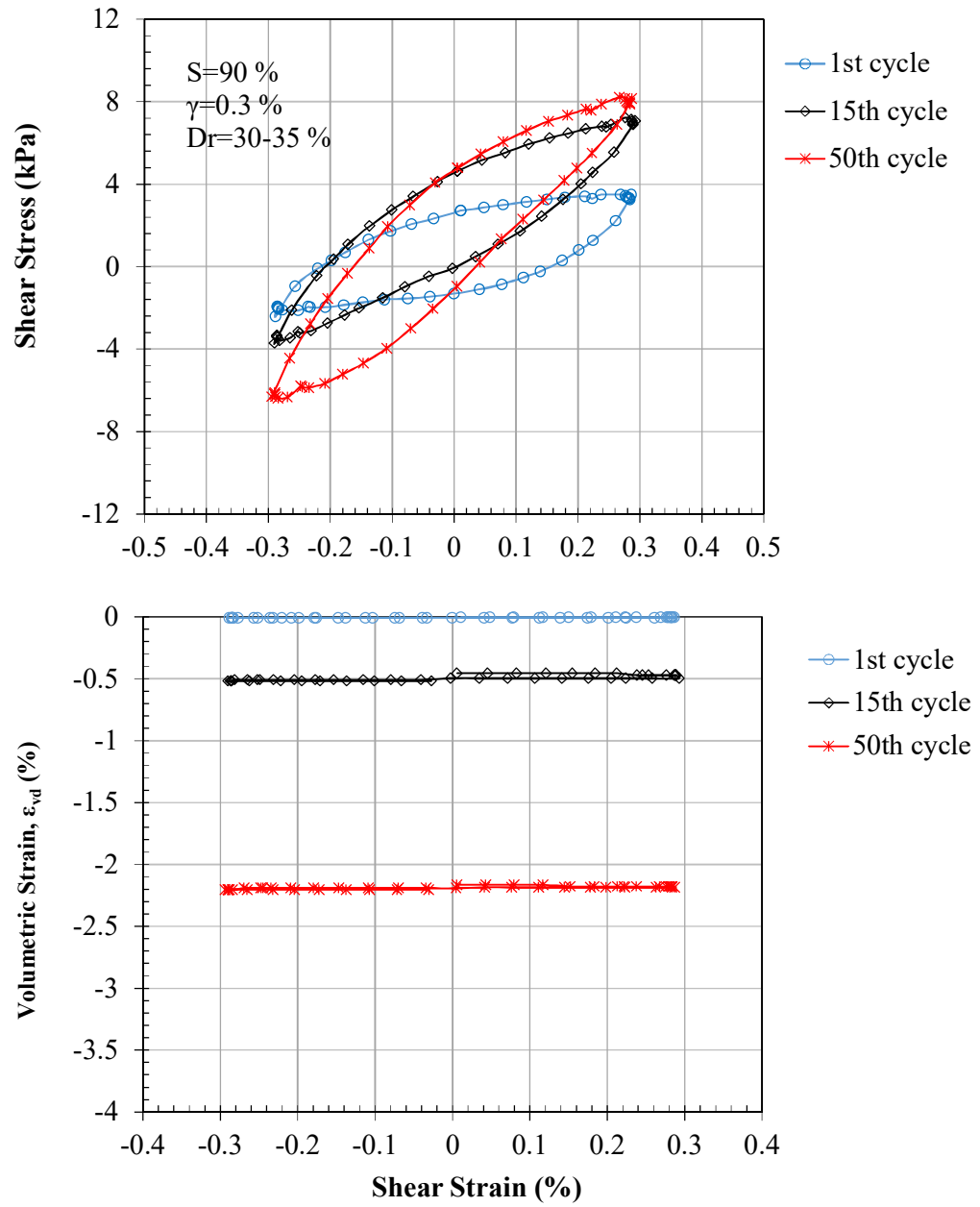


Figure 4.14 : Shear stress and volumetric strains related to cyclic shear strains with 0.3 % amplitude in a sample with 90 % degree of saturation.

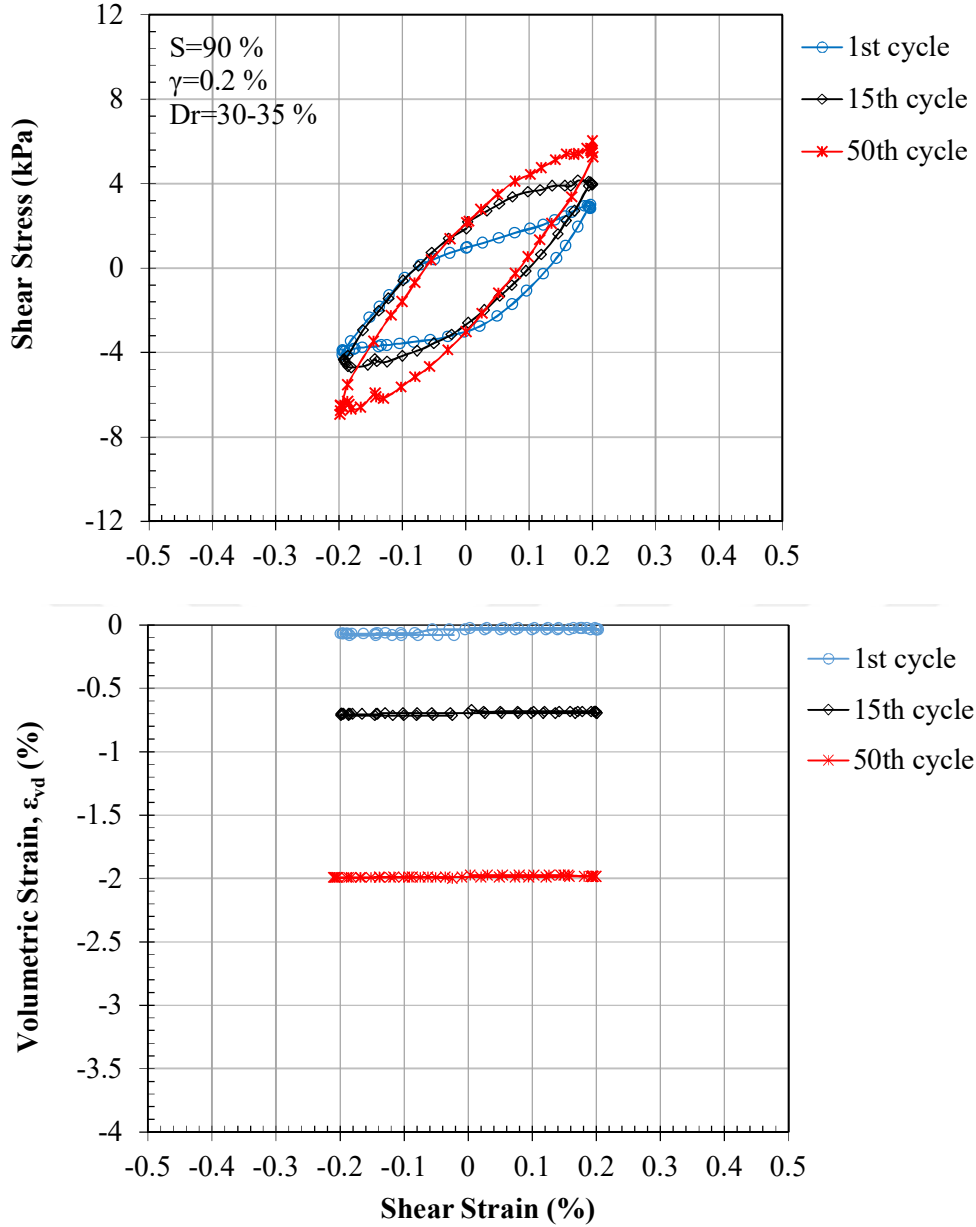


Figure 4.15 : Shear stress and volumetric strains related to cyclic shear strains with 0.2 % amplitude in a sample with for 90 % degree of saturation.

The shear stresses under different shear strain amplitudes were examined to estimate secant shear modulus (equivalent shear modulus). The relationship between the secant shear modulus and shear strain amplitudes, in terms of the number of cycles, was displayed in Figure 4.16. All in all, the secant shear modulus (G_{sec}) progressively increased within 15 cycles, and then continued its development at a slower rate with further application of loading cycles. Secant modulus was the slope of a line drawn from the origin of the shear stress-shear strain hysteresis loop and intersecting the curve at the point of interest.

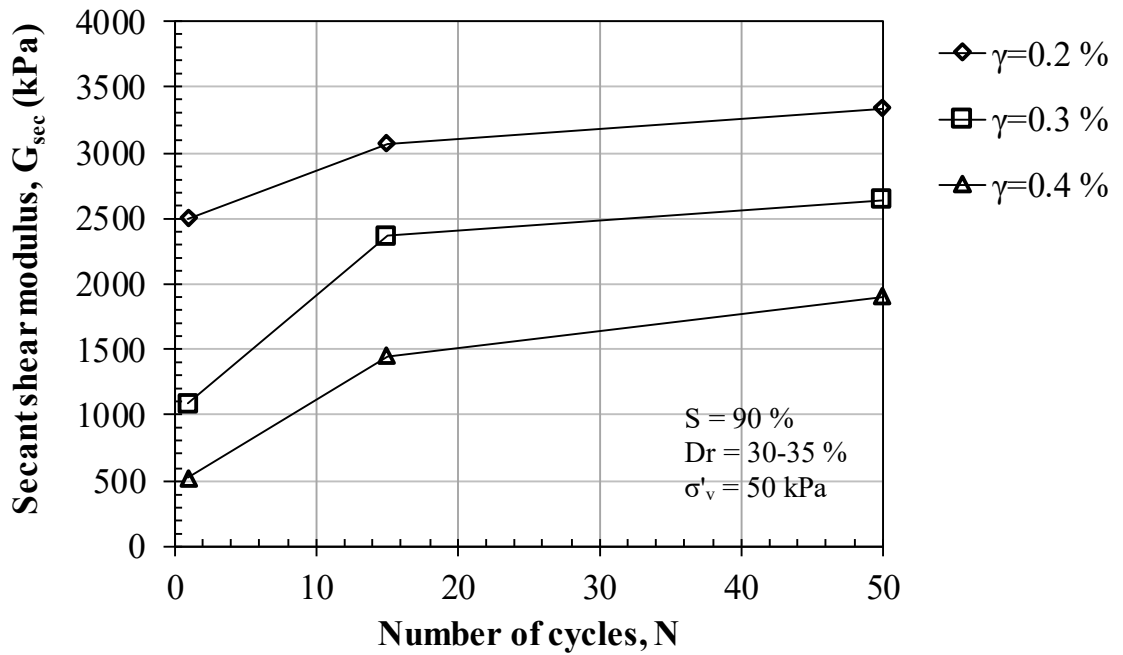


Figure 4.16 : Secant shear modulus behavior for the specimen with $S=90\%$ at different shear strain amplitudes.

After determination of secant shear modulus, maximum shear modulus was also calculated. It was observed from tangent of the first cycle of the shear stress-shear strain hysteresis loop. Additionally, the larger dynamic shear modulus ratio (G/G_{max}) were observed at the shear strain amplitude of 0.2% (Table 4.6). The dynamic shear modulus reduction curves were compared with Darendeli (2001) and Seed and Idriss (1991) upper limit boundary at $S=100\%$.

Table 4.6 : Shear modulus values.

Shear strain amplitudes (%)	Number of cycles, N	G_{max} (kPa)	G_{sec} (kPa)	G/G_{max}
0.2	1	4000	2500	0.63
	15		3070	0.77
	50		3340	0.84
0.3	1	3570	1087	0.30
	15		2367	0.66
	50		2643	0.74
0.4	1	3500	520	0.15
	15		1450	0.41
	50		1905	0.54

Figure 4.17 illustrated the shear modulus ratio of the sample with $S=90\%$ at the first, 15th and 50th cycles and Darendeli (2001) and Seed and Idriss (1991) upper limit boundary curves. Due to the experiments were performed strain-controlled, smooth curves was not observed and similar trend could not obtained with Darendeli (2001) and Seed and Idriss (1991) upper limit boundary curves.

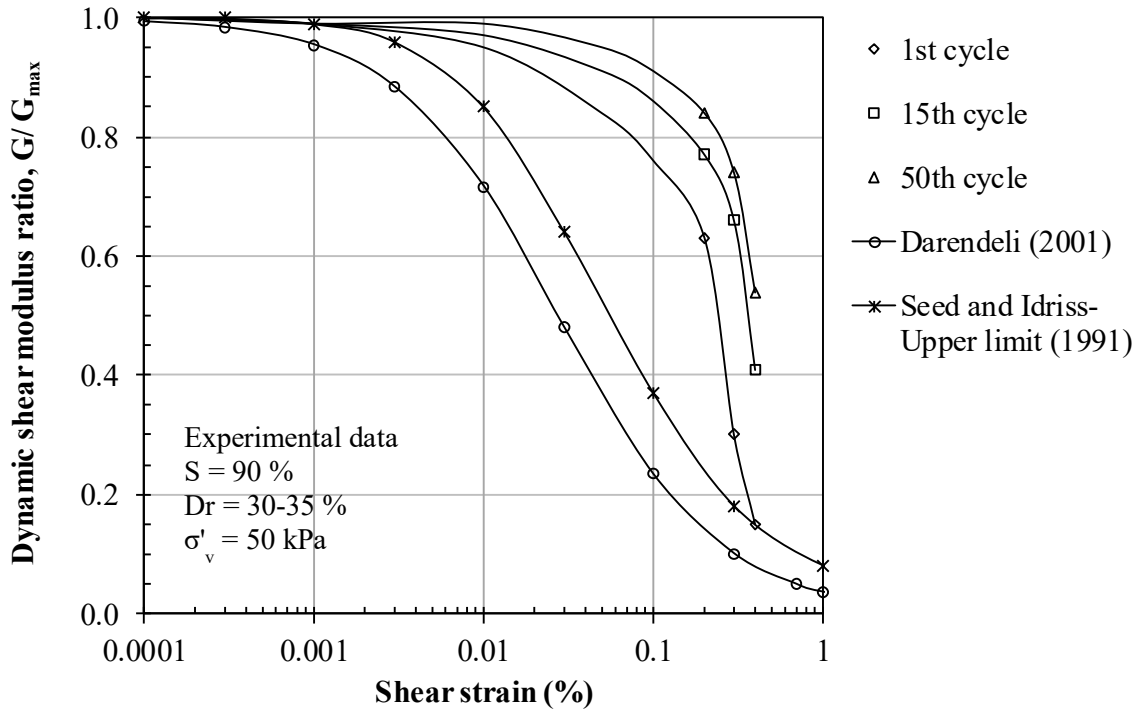


Figure 4.17 : The shear modulus degradation curves.

4.3 Comparison of Undrained and Drained Strain-Controlled Tests in Partially Saturated Sand Samples

The comparison of undrained and reliable drained results was investigated after the recommended $K_0=1.0$ and calibration setting.

The parameters affected r_u and ε_{vd} were determined to be the degree of saturation, relative density, and shear strain amplitude. The excess pore water pressure ratio and volumetric strain were plotted according to a different range of degrees of saturation and shear strain amplitudes in Figure 4.20 to Figure 4.22. The figures demonstrated that the excess pore water pressure ratio and volumetric strain increased as the degree of saturation and shear strain amplitudes increased. The loose sand samples displayed tendency to contract under shear-induced volumetric strains. As a result of strain-

hardening behavior with dynamic compaction, the loose samples became denser. The shear strain amplitude of 0.3 % and 0.4 % developed a significant change in the excess pore water pressure ratio. At the shear strain amplitude of 0.2 %, although the volumetric strain increment was noticeable, the excess pore water pressure generation was not clearly observed.

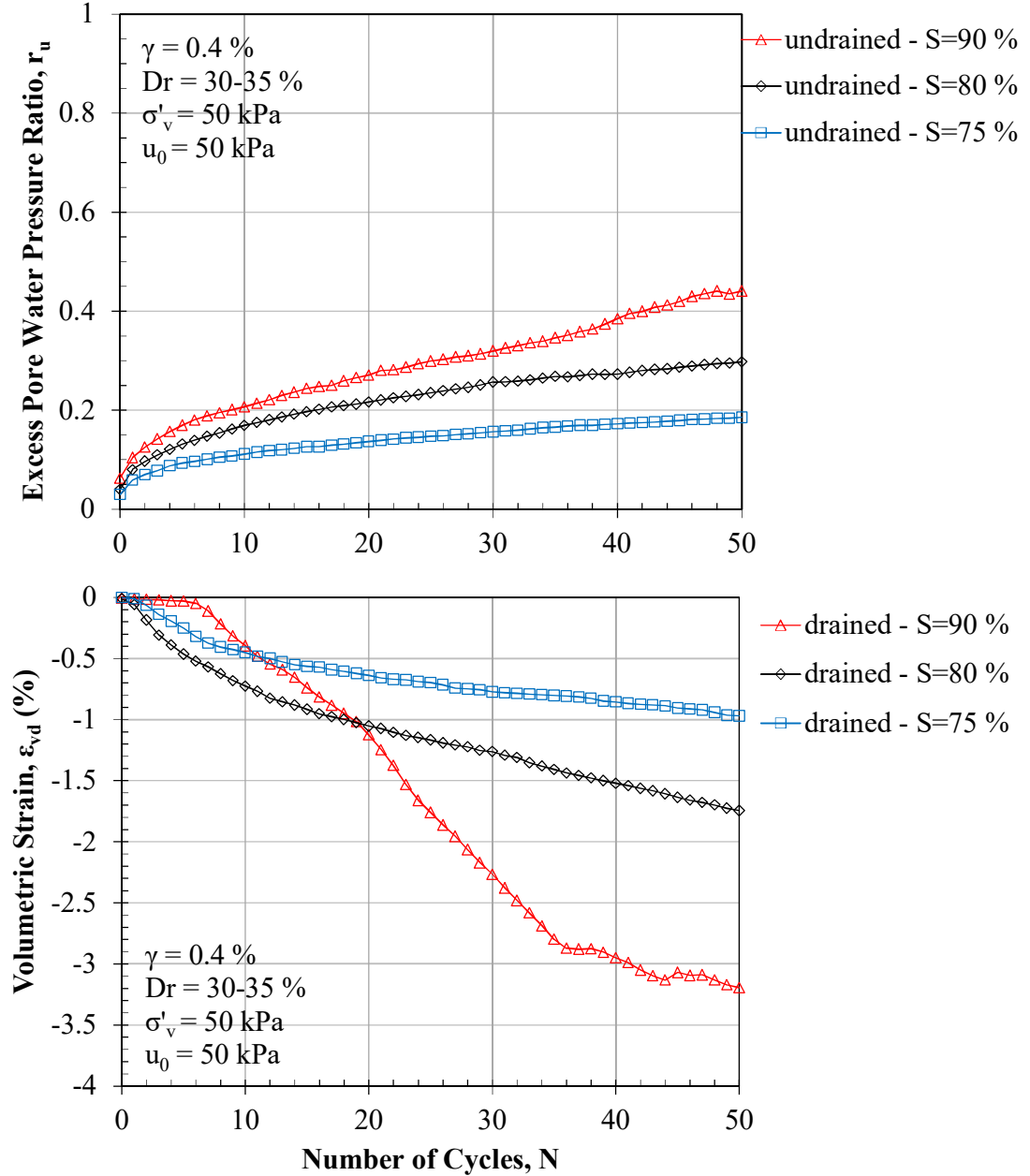


Figure 4.18 : Comparison of r_u and ϵ_{vd} under $\sigma'_v = 50 \text{ kPa}$, $\gamma = 0.4 \%$ and $Dr = 30-35 \%$ at different range of degrees of saturation.

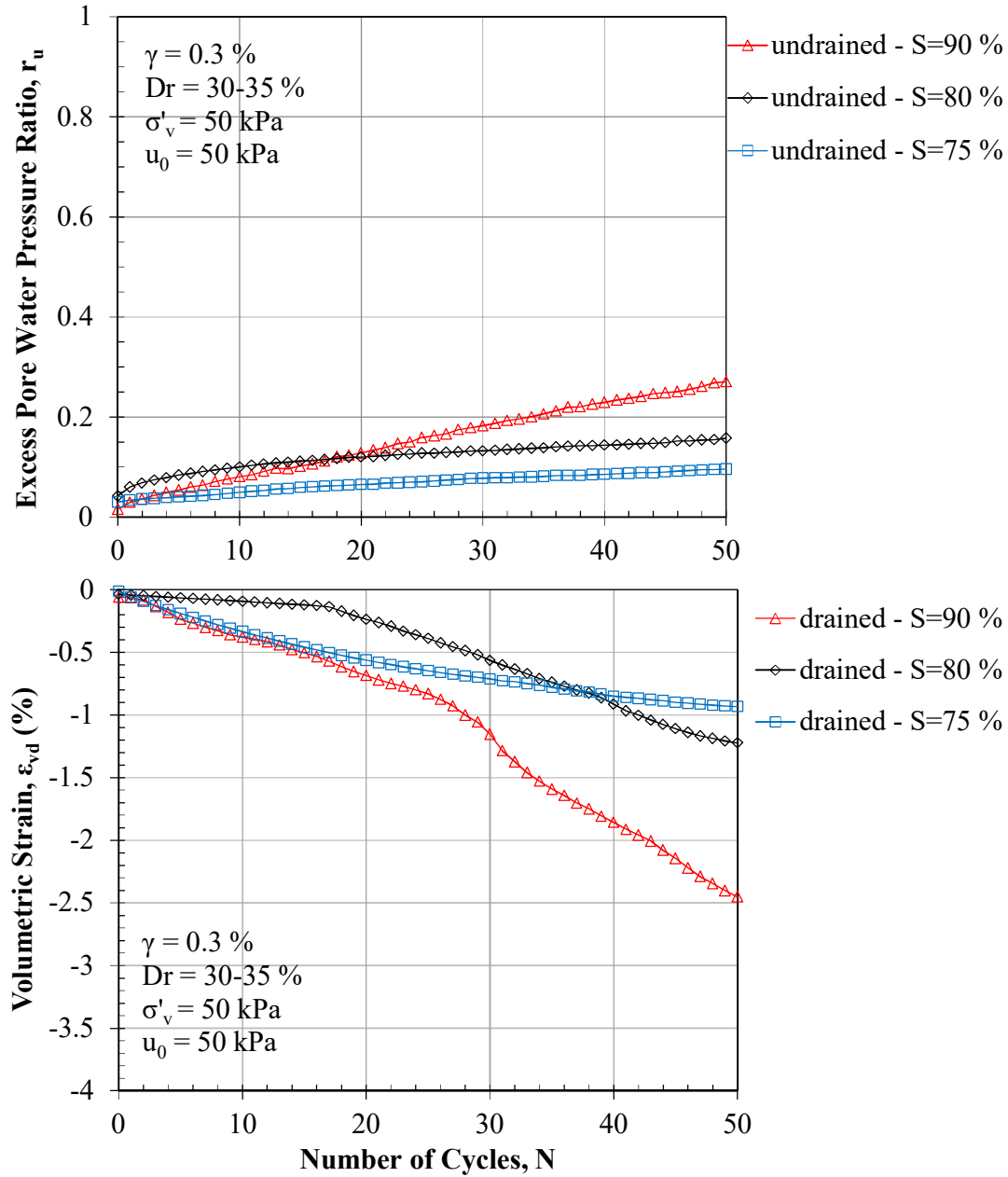


Figure 4.19 : Comparison of r_u and ϵ_{vd} under $\sigma'_v = 50 \text{ kPa}$, $\gamma = 0.3\%$ and $Dr = 30-35\%$ at different range of degrees of saturation.

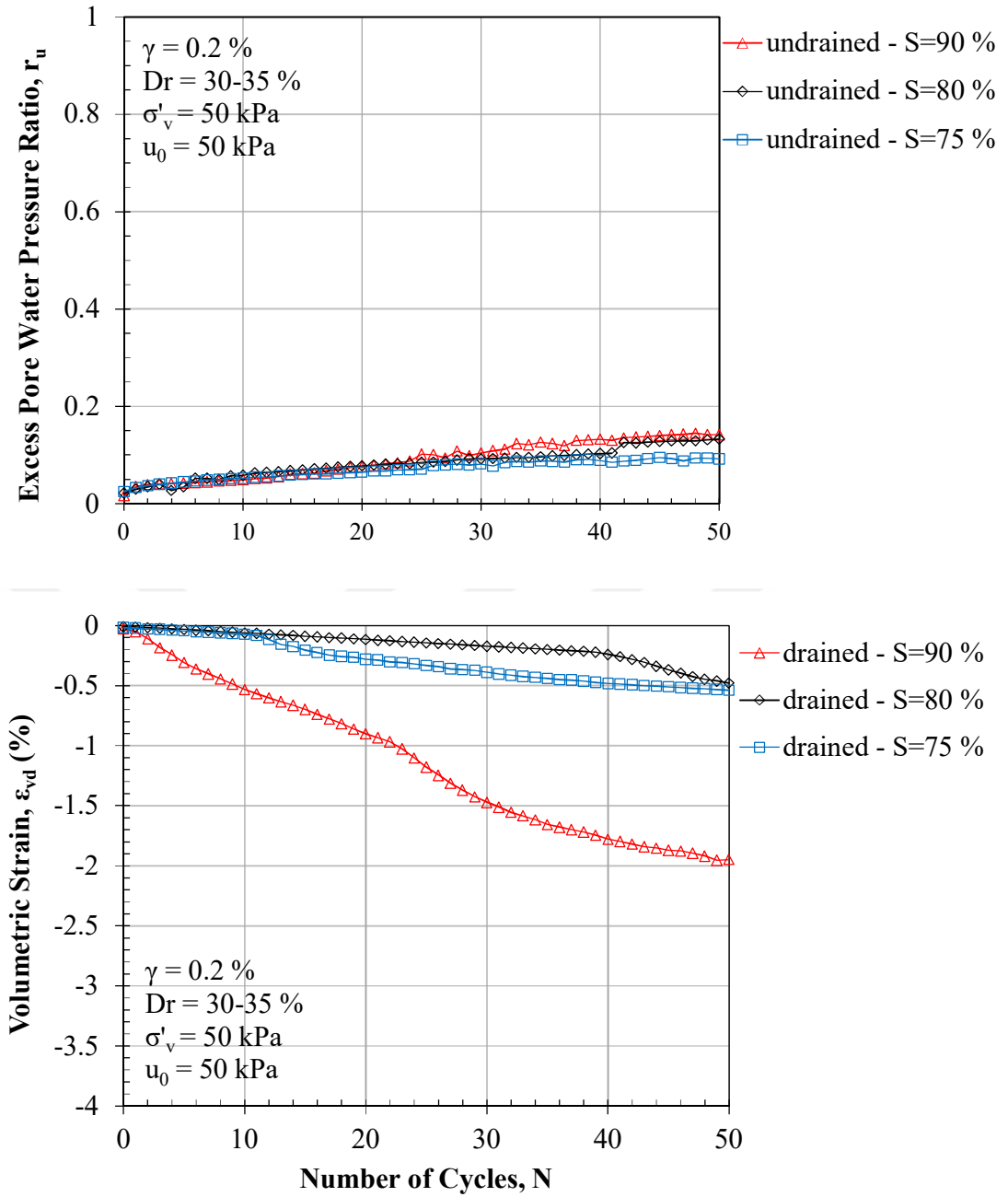


Figure 4.20 : Comparison of r_u and ϵ_{vd} under $\sigma'_v=50$ kPa, $\gamma=0.2$ % and $Dr=30-35$ % at different range of degrees of saturation.

A relationship between volumetric strain and excess pore water pressure ratio was illustrated in Figure 4.23. The volumetric strains and excess pore water pressure ratios were determined from 50th cycle of the undrained and drained strain-controlled tests. It was observed that the excess pore water pressure ratio can be used to estimate volumetric strains. Figure 4.23 was also shown that a reduction in degree of saturation yielded smaller excess pore water pressure ratio and volumetric strain.

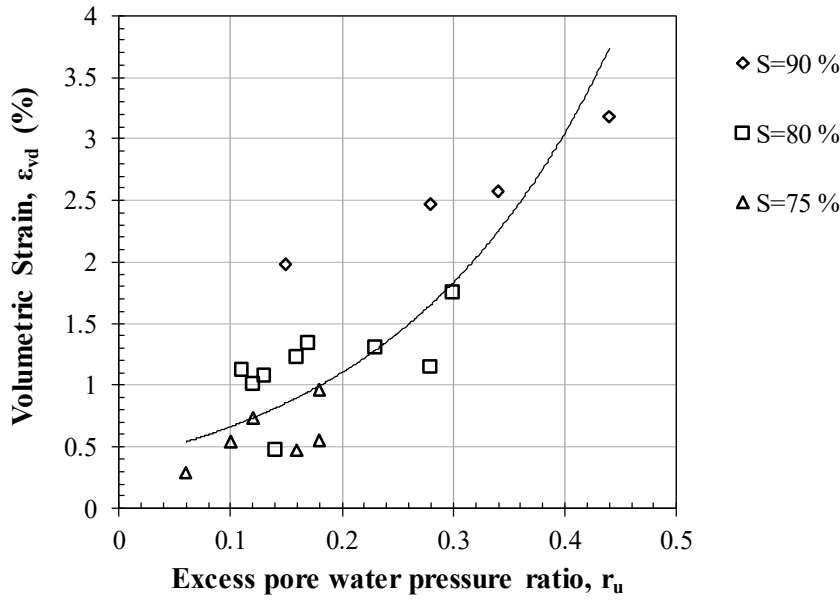


Figure 4.21 : Volumetric strains vs. excess pore water pressure ratios.

4.4 Estimation of Air-Water Bulk Modulus (K_{aw})

When the drained and undrained experiments were evaluated together, the relationship between r_u and ϵ_{vd} by considering the sample which had similar soil structure was investigated. Firstly, the rebound modulus was calculated using the formula shown in equation 4.3 proposed by Finn et al. (1976) using the vertical effective stress obtained in undrained strain-controlled experiments. The bulk modulus of air-water mixture for partially saturated sand samples was calculated from the rebound modulus, the change in pore pressure and volumetric strain by using equation 4.5. Since the increment in pore water pressure in the undrained experiments was more clearly seen at shear strain amplitude of 0.4 %, the results of samples with relative densities of 30-35 % and 40-45 % were used. The following calculation steps for $D_r=45\%$, $\gamma=0.4\%$ and $S=75\%$;

$$\bar{E}_r = \frac{(\sigma'_v)^{1-m}}{mK_2(\sigma'_{v0})^{n-m}} \quad (4.3)$$

From the undrained test at 15th cycle: $\sigma'_v = 910$ psf (45.5 kPa) and $\sigma'_{v0} = 1000.0$ psf (50.0 kPa)

$$\bar{E}_r = \frac{(910)^{1-0.43}}{0.43 \times 0.0025 \times (1000)^{0.62-0.43}} = 12168.7 \quad (4.4)$$

$$\Delta u = \frac{\Delta \varepsilon_{vd}}{\frac{1}{\bar{E}_r} + \frac{n}{K_w}} \quad (4.5)$$

From the undrained test at 15th cycle: $\Delta u = 2.8$ psf (0.14 kPa), $\bar{E}_r = 12168.7$ and $n=0.4375$

From the drained test at 15th cycle: $\Delta \varepsilon_{vd} = 0.014$ %

$$K_{aw} = \left| \frac{n}{\frac{\Delta \varepsilon_{vd}}{\Delta u} \frac{1}{\bar{E}_r}} \right| = \left| \frac{0.4375}{\frac{0.00014}{2.8} - \frac{1}{12168.7}} \right| = 13596 \text{ psf} = 680 \text{ kPa} \quad (4.6)$$

The steps were repeated for each cycle and then average K_{aw} was calculated for different range of degrees of saturation. In Figure 4.24, it was determined that there is an exponential relationship between the K_{aw} value and the degree of saturation. As expected, the bulk modulus of the air-water mixture in partially saturated samples was lower than that of the water and it reduced as the degree of saturation decreased. The experimental results were compared with Koning (1963) correlation and empirical estimation using both laboratory and numerical analysis results by Seyedi-Viand and Eseller-Bayat (2020) (Table 4.6). The experimental and numerical results were in a good agreement with each other whereas the Koning (1963) correlation had significantly lower values (Figure 4.24).

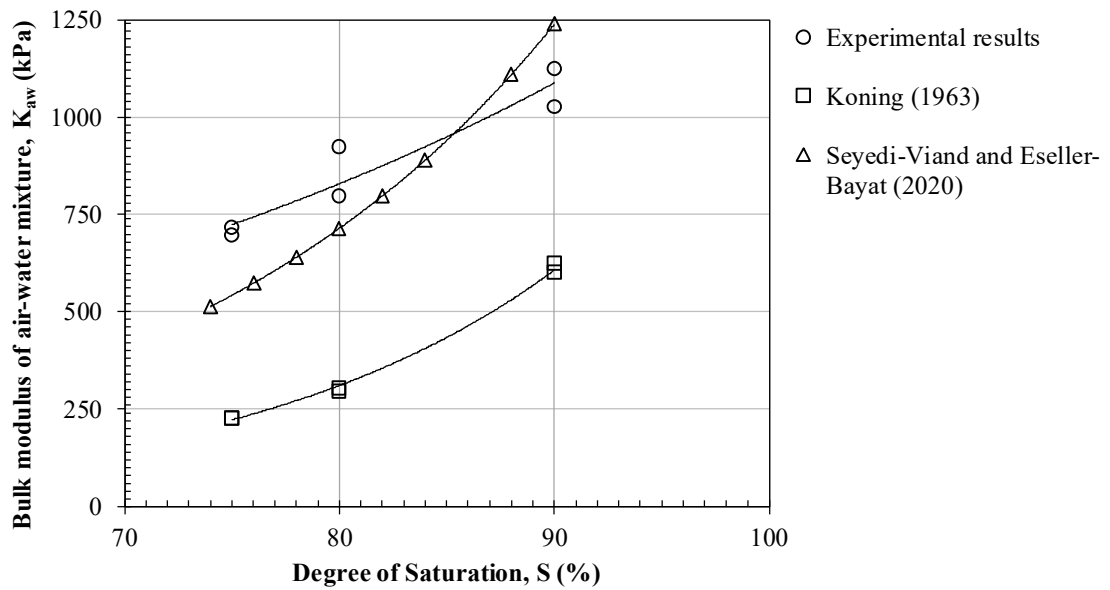


Figure 4.22 : Bulk modulus of air-water mixture related to degree of saturation.

Table 4.7 : The comparison of K_{aw} values.

Degree of Saturation, S (%)	Experimental Data	Seyedi-Viand and Eseller-Bayat (2020)	Koning (1963)
	K_{aw} (kPa)	K_{aw} (kPa)	K_{aw} (kPa)
75	697	514	227
75	718	574	228
80	799	715	296
80	923	798	304
90	1126	1110	603
90	1027	1239	626

The equation developed by Koning was a simplified model that formulated the compressibility of air and water mixture in the void of the sand. The experimental and analytical results presented in this study demonstrated that there is a great need to develop new correlations to predict K_{aw} .

5. CONCLUSIONS AND RECOMMENDATIONS

This study was conducted to investigate the volumetric strains to be occurred in sands mitigated by Induced Partial Saturation (IPS) technique. This technique allowed to control the degree of saturation by entrapping air/gas bubbles within the voids of the sand. The preliminary research about this mitigation technique was presented by Eseller (2004) and Yegian et al. (2007). Recent researches focused on the excess pore water pressure generation with respect to degree of saturation under strain-controlled cyclic simple shear (Eseller-Bayat et al. (2013 (a) and (b)), Eseller-Bayat and Gülen (2020)). Moreover, in this research, the effectiveness of IPS was evaluated by considering volumetric strains obtained from drained strain-controlled tests on fully and partially saturated sand specimens. The research reported in this thesis included the following tasks:

- IPS technique was used to prepare partially saturated sand specimens at desired degrees of saturation. Sodium percarbonate was used as air/gas bubble generating chemical. Depending on the chemical ratio, the degree of saturation can be controlled. The intended degree of saturation was determined from the amount of sodium percarbonate to sand ratio of R correlation.
- Partially saturated sand samples were prepared by wet pluviation. The sodium percarbonate-water solution was prepared for different concentrations varying in the range of 0.2-0.3 %.
- In fully saturated sand samples, the moist tamping method was chosen to obtain homogeneous and loose soil fabric. The moisture content was chosen 10%.
- VJ-Tech dynamic simple shear test device was used to investigate dynamic properties of the sand under the effect of vertical effective stress. The device was controlled by Clisp Studio program that enables saturation, consolidation and cyclic simple shear test stages.
- Fully saturated samples were saturated at higher cell pressures whereas lower pressures were applied for partially saturated samples. Then, isotropic

consolidation was preferred. Undrained and drained cyclic simple shear tests were performed on fully and partially saturated sands.

The following important remarks were observed related to the experimental procedure with DSS-C device:

- During the preparation of partially saturated sand samples, free water was collected between membrane and sand particles. When the cell pressure was applied, free water and water in the voids were compressed at the same time. Because of this reason, B-check 1 measured higher than it should be. To obtain reliable data, B-check 2 was performed and a relation was presented between B-check 2 and degree of saturation.
- The partially saturated samples were compressed laterally due to the cell pressure and their loose form. The sample diameter was affected by lateral deformation, and thus the relative density was recalculated with current diameter.
- The drained cyclic simple shear tests were performed on fully and partially saturated sands. In the initial test series, the vertical effective stress reduced which was expected to stay steady due to a software error. Therefore, the results were not considered reliable for both fully and partially saturated specimens. A software setting was recommended by VJ-Tech and the experiments were repeated. Unfortunately, the recommended software settings did not work for the fully saturated specimens.

The following conclusions were drawn according to the test results obtained:

- According to the undrained cyclic simple shear tests on fully saturated sands, the excess pore water pressure generation reached the initial vertical effective stress as expected. However, the excess pore water pressure ratio (r_u) did not reach 1.0 in the partially saturated sand samples. In both cases, r_u affected by relative density, shear strain amplitudes and degree of saturation.
- As a result of the suggested software settings, the drained cyclic simple shear tests on partially saturated sands were conducted. Volumetric strains (ϵ_{vd}) in the partially saturated sands were related to degree of saturation, relative

density and shear strain amplitude. Volumetric strains decreased in conjunction with;

1. Decreasing degree of saturation
 2. Increasing relative density
 3. Decreasing shear strain amplitude.
- The volumetric strains were compared with the different correlations established by Finn et al. (1976) and Byrne (1991). At $S = 90\%$, experimental volumetric strains followed similar trends with empirical correlations for $S = 100\%$, at two different relative densities ($35\% - 45\%$). The lower volumetric strains were observed as the degree of saturation decreased. The results show that the volumetric strain was not independent of degree of saturation. So the correlation developed by Finn and Byrne are only for fully saturated sands.
 - As the number of loading cycles increased, both the shear stress and the volumetric strain increased. As a result of dynamic compaction, the loose sample displayed a hardening behavior.
 - The excess pore water pressures and volumetric strains built up with increasing shear strain amplitudes, degree of saturation and number of cycles.
 - Shear modulus degradation was evaluated at different shear strain amplitudes. Secant shear modulus was obtained lower at the higher shear strain amplitudes. As the number of cycles increased, the dynamic shear modulus ratio ($G/G_{\max}$) increased.
 - Air-water mixture bulk modulus (K_{aw}) was defined as an important parameter to be used in numerical modeling of partially saturated sands. Although Koning (1963) proposed a widely used equation, various researches were performed to obtain more accurate estimations. This study evaluated several research; such as empirical K_{aw} of Seyedi-Viand and Eseller-Bayat (2020), Koning (1963) and Martin et al. (1975). The analytical equation proposed by Martin et al. (1975) was used to estimate experimental-based K_{aw} . While K_{aw} was confirmed with the experimental results and empirical estimation, lower values were predicted by Koning (1963) equation. The bulk modulus of air-water mixture decreased with relatively small reductions in the degree of saturation.

This thesis investigated Induced Partial Saturation (IPS) as a mitigation technique in terms of volumetric deformations. The results clearly implied that the volumetric strains under cyclic loading decreased as the degree of saturation reduced. The findings in this study will provide data and knowledge base for the future research to evaluate the post-liquefaction deformations in specimens mitigated by Induced Partial Saturation (IPS).



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APPENDICES

APPENDIX A: Calculation of Relative Density and Degree of Saturation for Fully and Partially Saturated Specimens



APPENDIX A

Table A.1 : Fully saturated specimen preparation calculations.

FULLY SATURATED SPECIMEN		
e_{max}		0.89
e_{min}		0.57
γ_s (g/cm ³)		2.65
Water content, w (%)		10
Specimen preparation stage		
Weight of sand, W_s (g)	Measured	160.0
Initial weight of water, W_w (g)	Measured	16.0
Diameter, D_0 (mm)	Measured	68.5
Average height of specimen, $H_{sp,0}$ (mm)	$H_{sp,0} = (H_{sp,i} + H_{sp,ii} + H_{sp,iii})/3$	29.3
Area of specimen, $A_{sp,0}$ (cm ²)	$A_{sp,0} = (\pi \times D_0^2)/4$	36.9
Volume of specimen, $V_{sp,0}$ (cm ³)	$V_{sp,0} = (A_{sp,0} \times H_{sp,0})/10$	108.0
Volume of sand, V_s (cm ³)	$V_s = W_s / \gamma_s$	60.4
Volume of voids, $V_{v,0}$ (cm ³)	$V_{v,0} = V_{sp,0} - V_s$	47.6
Void ratio, e_0	$e_0 = V_{v,0} / V_s$	0.8
Initial relative density, $D_{r,0}$ (%)	$D_{r,0} = ((e_{max} - e_0) / (e_{max} - e_{min})) \times 100$	31.7
Initial saturation degree, S_0 (%)	$S_0 = ((w/100) \times \gamma_s) / e_0$	33.6

Table A.2 : Fully saturated specimen saturation and consolidation calculations.

SATURATION		
Water escaping from specimen, V_{ew} (cm ³)	Clisp output data	0.0
APCH Volume change, $\Delta V_{APCH,0}$ (cm ³)	Clisp output data	-32.5
LVDT reading, H_{LVDT} (mm)	Clisp output data	0.1
Specimen height, $H_{sp,1}$ (mm)	$H_{sp,1} = H_{sp,0} - H_{LVDT}$	29.2
Horizontal deformation, δ (mm)	Measured *	0.0
Specimen diameter, D_1 (mm)	$D_1 = D_0 - \delta$	68.5
Area of specimen, $A_{sp,1}$ (cm ²)	$A_{sp,1} = (\pi \times D_1^2) \times 400$	36.9
Volume of specimen, $V_{sp,1}$ (cm ³)	$V_{sp,1} = A_{sp,1} \times H_{sp,1}$	107.7
Volume of voids, $V_{v,1}$ (cm ³)	$V_{v,1} = V_{sp,1} - V_s$	47.3
Void ratio, e_1	$e_1 = V_{v,1}/V_s$	0.8
Relative density, $D_{r,1}$ (%)	$D_{r,1} = ((e_{max} - e_1)/(e_{max} - e_{min})) \times 100$	33.1
Water content, w_1 (%)	$w_1 = ((W_w - \Delta V_{APCH,0})/W_s) \times 100$	30.3
Saturation degree, S_2 (%)	$(w_1 \times \gamma_s)/e_1$	102.4
CONSOLIDATION		
Water escaping from specimen, V_{ew} (cm ³)	Clisp output data	0.0
APCH Volume change, $\Delta V_{APCH,1}$ (cm ³)	Clisp output data	2.3
LVDT reading, H_{LVDT} (mm)	Clisp output data	0.0
Final specimen height, $H_{sp,2}$ (mm)	$H_{sp,2} = H_{sp,1} - H_{LVDT}$	29.2
Horizontal deformation, δ (mm)	Measured*	0.0
Final specimen diameter, D_2 (mm)	$D_2 = D_1 - \delta$	68.5
Final area of specimen, $A_{sp,2}$ (cm ²)	$A_{sp,2} = (\pi \times D_1^2) \times 400$	36.9
Final volume of specimen, $V_{sp,2}$ (cm ³)	$V_{sp,2} = A_{sp,2} \times H_{sp,2}$	107.7
Final volume of voids, $V_{v,2}$ (cm ³)	$V_{v,2} = V_{sp,2} - V_s$	47.3
Void ratio, e_2	$e_2 = V_{v,2}/V_s$	0.8
Relative density, $D_{r,2}$ (%)	$D_{r,2} = ((e_{max} - e_2)/(e_{max} - e_{min})) \times 100$	33.4
Water content, w_2 (%)	$w_2 = ((W_w - \Delta V_{APCH,0} - \Delta V_{APCH,1})/W_s) \times 100$	28.9
Saturation degree, S_2 (%)	$S_2 = (w_2 \times \gamma_s)/e_2$	97.8

*Horizontal deformation was calculated from the digital photos.

Table A.3 : Partially saturated specimen preparation calculations.

PARTIALLY SATURATED SPECIMEN		
e_{max}		0.89
e_{min}		0.57
γ_s (g/cm ³)		2.65
Specimen preparation stage		
R ratio (%)	Measured	0.2
Weight of sand, W_s (g)	Measured	160
Weight of sodiumpercarbonate, W_{Na} (g)	$W_{Na} = (R \times W_s)/100$	0.32
Initial weight of water, W_w (g)	Measured	50.0
Diameter, D_0 (mm)	Measured	68.3
Average height of specimen, $H_{sp,0}$ (mm)	$H_{sp,0} = (H_{sp,i} + H_{sp,ii} + H_{sp,iii})/3$	28.2
Area of specimen, $A_{sp,0}$ (cm ²)	$A_{sp,0} = (\pi \times D_0^2)/4$	36.6
Volume of specimen, $V_{sp,0}$ (cm ³)	$V_{sp,0} = (A_{sp,0} \times H_{sp,0})/10$	103.3
Volume of sand, V_s (cm ³)	$V_s = W_s / \gamma_s$	60.4
Volume of voids, $V_{v,0}$ (cm ³)	$V_{v,0} = V_{sp,0} - V_s$	42.9
Void ratio, e_0	$e_0 = V_{v,0} / V_s$	0.7
Relative density, $D_{r,0}$ (%)	$D_{r,0} = ((e_{max} - e_0) / (e_{max} - e_{min})) \times 100$	55.9
Initial height of free water, $H_{fw,0}$ (mm)	Measured	30.3
Weight of free water, $W_{fw,0}$ (g)	$W_{fw,0} = ((H_{fw,0} - H_{sp,0})/10) \times A_{sp,0}$	8.8
Weight of used water in specimen, W_{wu} (g)	$W_{wu} = W_w - W_{fw,0}$	41.2
Initial saturation degree, S_0 (%)	$S_0 = (W_{wu} / V_v) \times 100$	96.0
Before the test		
Average height of specimen, $H_{sp,1}$ (mm)	$H_{sp,1} = (H_{sp,1i} + H_{sp,1ii} + H_{sp,1iii})/3$	29.5
Diameter, D_1 (mm)	Measured	68.3
Area of specimen, $A_{sp,1}$ (cm ²)	$A_{sp,1} = (\pi \times D_1^2)/4$	36.6
Volume of specimen, $V_{sp,1}$ (cm ³)	$V_{sp,1} = (A_{sp,1} \times H_{sp,1})/10$	108.1
Volume of voids, $V_{v,1}$ (cm ³)	$V_{v,1} = V_{sp,1} - V_s$	47.7
Void ratio, e_1	$e_1 = V_{v,1} / V_s$	0.8
Relative density, $D_{r,1}$ (%)	$D_{r,1} = ((e_{max} - e_1) / (e_{max} - e_{min})) \times 100$	31.2
Water content, w_1 (%)	$w_1 = (S_1 \times e_1) / \gamma_s$	22.9
Weight of used water in specimen, $W_{wu,1}$ (g)	$W_{wu,1} = (w_1 \times W_s)/100$	36.6
Total weight of specimen, W_t (g)	$W_t = W_{wu,1} + W_s$	196.6
Saturation degree, S_1 (%)	$S_1 = (W_{wu,1} / V_{v,1}) \times 100$	76.7

Table A.4 : Partially saturated specimen saturation and consolidation calculations.

SATURATION STEP		
Water escaping from specimen, V_{ew} (cm ³)	Clisp output data	1.2
APCH Volume change, ΔV_{APCH} (cm ³)	Clisp output data	-6.9
Volume change in specimen, ΔV_{sp} (cm ³)	$\Delta V_{sp} = \Delta V_{APCH} + V_{ew}$	-5.7
LVDT reading, H_{LVDT} (mm)	Clisp output data	0.1
Specimen height, $H_{sp,2}$ (mm)	$H_{sp,2} = H_{sp,1} - H_{LVDT}$	29.4
Horizontal deformation, δ (mm)	Measured	0.15
Specimen diameter, D_2 (mm)	$D_2 = D_1 - \delta$	68.15
Area of specimen, $A_{sp,2}$ (cm ²)	$A_{sp,2} = (\pi \times D_2^2) \times 400$	36.5
Volume of specimen, $V_{sp,2}$ (cm ³)	$V_{sp,2} = A_{sp,2} \times H_{sp,2}$	107.1
Volume of voids, $V_{v,2}$ (cm ³)	$V_{v,2} = V_{sp,2} - V_s$	46.8
Void ratio, e_2	$e_2 = V_{v,2}/V_s$	0.8
Relative density, $D_{r,2}$ (%)	$D_{r,2} = ((e_{max} - e_2)/(e_{max} - e_{min})) \times 100$	36.1
Water content, w_2 (%)	$w_2 = ((W_{wu,1} - \Delta V_{sp})/W_s) \times 100$	26.4
Saturation degree, S_2 (%)	$(w_2 \times \gamma_s)/e_2$	90.4
CONSOLIDATION STEP		
Water escaping from specimen, V_{ew} (cm ³)	Clisp output data	0.0
APCH Volume change, ΔV_{APCH} (cm ³)	Clisp output data	1.2
LVDT reading, H_{LVDT} (mm)	Clisp output data	0.0
Final specimen height, $H_{sp,3}$ (mm)	$H_{sp,3} = H_{sp,2} - H_{LVDT}$	29.4
Horizontal deformation, δ (mm)	Measured *	0.0
Final specimen diameter, D_3 (mm)	$D_3 = D_2 - \delta$	68.2
Final area of specimen, $A_{sp,3}$ (cm ²)	$A_{sp,3} = (\pi \times D_3^2) \times 400$	36.5
Final volume of specimen, $V_{sp,3}$ (cm ³)	$V_{sp,3} = A_{sp,3} \times H_{sp,3}$	107.1
Final volume of voids, $V_{v,3}$ (cm ³)	$V_{v,3} = V_{sp,3} - V_s$	46.7
Void ratio, e_3	$e_3 = V_{v,3}/V_s$	0.8
Relative density, $D_{r,3}$ (%)	$D_{r,3} = ((e_{max} - e_3)/(e_{max} - e_{min})) \times 100$	36.3
Water content, w_3 (%)	$w_3 = ((W_{wu,1} - \Delta V_{sp} - \Delta V_{APCH})/W_s) \times 100$	25.7
Saturation degree, S_3 (%)	$S_3 = (w_3 \times \gamma_s)/e_3$	87.9

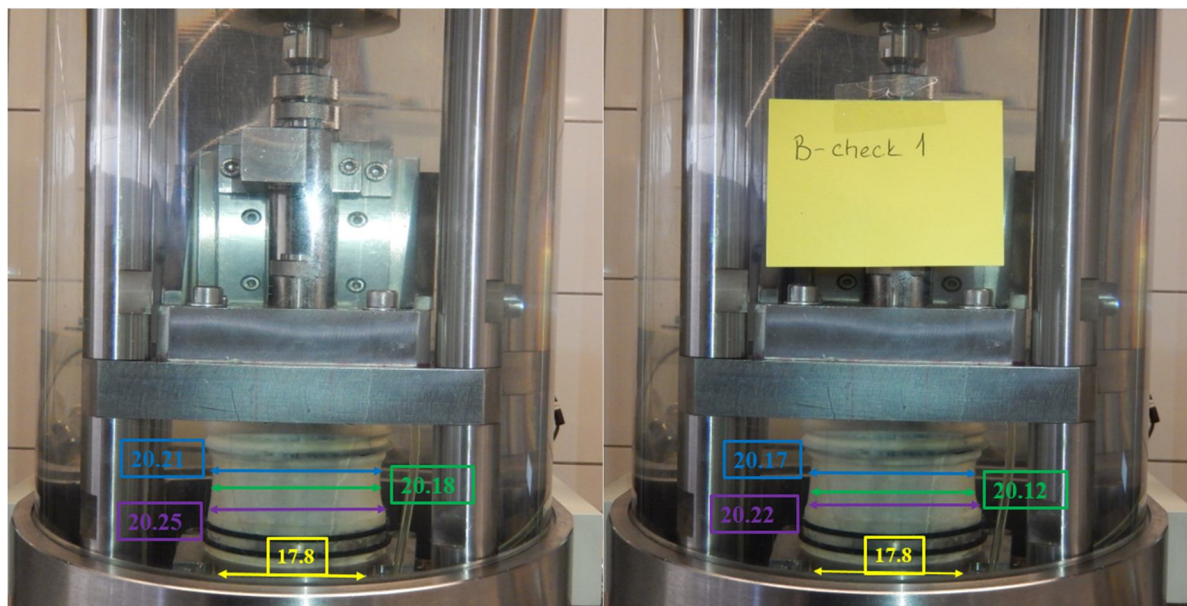


Figure A.1 : Partially saturated samples pictures before and after B-check 1

Table A.5 : Calculations of horizontal deformation.

Horizontal displacement (δ)				
	Reference length	Top diameter of the sample	Middle diameter of the sample	Bottom diameter of the sample
Length from the photos before B-check 1 (cm)	17.8	20.21	20.18	20.25
Length from the photos after B-check 1 (cm)	17.8	20.17	20.12	20.22
Differences between the diameters (cm)	0.0	0.04	0.06	0.03
Average difference from the photos (cm)	0.0	0.043		
Actual length (cm)	6.0	0.015		

Reference length from the photos: 17.8 cm

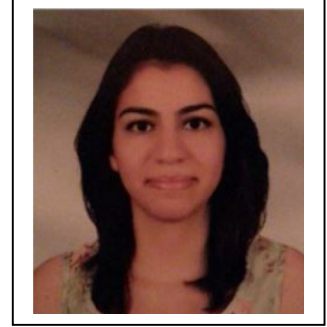
Actual reference length: 6 cm

Ratio: 2.97

Average difference between diameters before and after B-check 1 from the photos:
0.043 cm

Actual average horizontal displacement: $0.043/2.97 = 0.015$ cm

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