Mitigation of liquefaction of sand using microbial methods

He, Jia

2013

He, J. (2013). Mitigation of liquefaction of sand using microbial methods. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/53528>

<https://doi.org/10.32657/10356/53528>

**MITIGATION OF LIQUEFACTION OF SAND
USING MICROBIAL METHODS**

HE JIA

**SCHOOL OF CIVIL AND ENVIRONMENTAL
ENGINEERING**

2013

MITIGATION OF LIQUEFACTION OF SAND USING MICROBIAL METHODS

HE JIA

School of Civil and Environmental Engineering

A thesis submitted to the

Nanyang Technological University

in partial fulfilment of the requirement for the degree of

Doctor of Philosophy

2013

ACKNOWLEDGEMENTS

I would like to give my deep appreciation to my supervisor, Professor Chu Jian, for his persistent academic supervision, help, and encouragement throughout my Ph.D. programme. It is his insightful research supervision and warm-hearted supports in many aspects that helped me achieve this goal.

I would like to extend my sincere thanks to my co-supervisor, Dr. Volodymyr Ivanov, for his precious ideas and advices in my research work. This Ph.D. project on a brand new inter-disciplinary topic could not be completed without him.

I would like to express my gratitude to my fellow research team members, Dr. Stabnikov Viktor, Miss Maryam Naeimi, Miss Li Bing, Mr. Wu Shifan, Mr. Aung Naing Myo, and my research colleague Dr. Guo Cheng-Hong, for their technical support and help in my experimental work.

I want to thank the technical staff, Mr. Vincent Heng, Mr. Eugene Tan, Mr. Andy Koh, Ms. Lim-Ding Susie from Geotechnics Lab, and Mr. Ong Chee Yung from Environmental Lab, who have assisted me in the preparation of the experiments.

My thanks go to Professor Liu Hanlong of Hohai University, who was the supervisor in my M.Eng. programme, for his valuable suggestions in my research career and his recommendation that helped me obtain the opportunity to study in Singapore.

Last but not least, I want to thank my wife and parents for their continued love and care. This Ph.D. thesis is the outcome of their sacrifice.

TABLE OF CONTENTS

Acknowledgements	i
Table of contents	iii
Summary	vii
List of tables	ix
List of figures	xi
List of symbols	xv
Chapter 1 Introduction	1
1.1 Background	1
1.2 Objectives	2
1.3 Scope of study	3
1.4 Outline of thesis	3
Chapter 2 Literature review	5
2.1 Introduction	5
2.2 Critical state soil mechanics framework for sand	5
2.3 Undrained behaviour of sandy soils	9
2.3.1 Undrained response of sand under triaxial compression	9
2.3.2 Other influential factors on undrained behaviour of sand	13
2.3.3 Peak strength and residual strength	16
2.3.4 Instability of sands	17
2.3.5 Undrained behaviour under cyclic loading	21
2.4 Liquefaction assessment	23
2.4.1 An overview of liquefaction assessment procedures	23
2.4.2 Pore pressure prediction	24
2.5 Liquefaction countermeasures	25
2.5.1 Conventional liquefaction countermeasures	25
2.5.2 Liquefaction remediation using biocement	29
2.6 Desaturation as a liquefaction countermeasure	30
2.6.1 Effect of gas bubbles on the stress-strain behaviour of sand	30
2.6.2 Methods of artificial desaturation	34
2.6.3 Determination of gas content	38

2.7 Microbial methods in geotechnical engineering	42
2.7.1 Biocements	42
2.7.2 Biological denitrification for gas generation	47
2.8 Summary	50
Chapter 3 Preliminary study on the biogas generation	53
3.1 Introduction	53
3.2 Materials and methods	53
3.2.1 Medium composition	53
3.2.2 Isolation and cultivation of denitrifying bacteria	54
3.2.3 Identification of bacteria	55
3.2.4 Batch experiment	57
3.2.5 Gas generation test	57
3.3 Results	59
3.3.1 Batch experiments	59
3.3.2 Gas generation tests	66
3.4 Discussion	69
3.5 Conclusions	71
Chapter 4 Behaviour of desaturated sand in the triaxial tests	73
4.1 Introduction	73
4.2 Materials and testing set-up	74
4.2.1 Materials	74
4.2.2 Sample preparation methods	75
4.2.3 Tests with gas-tubes	77
4.2.4 Triaxial testing system	78
4.2.5 Test procedure and test arrangement	80
4.3 Test results	83
4.3.1 Relationship between pore pressure coefficient B- value and degree of saturation	83
4.3.2 Undrained behaviour of loose sand	86
4.3.2.1 <i>Compression tests</i>	86
4.3.2.2 <i>Extension tests</i>	88
4.3.2.3 <i>Gas tube tests</i>	93
4.3.2.4 <i>Comparison between compression and extension test results</i>	95
4.3.2.5 <i>Instability lines of desaturated sand</i>	99

4.3.3 Undrained behaviour of medium dense sand	101
4.3.3.1 <i>Compression tests</i>	101
4.3.3.2 <i>Extension tests</i>	106
4.4 Conclusions	111
Chapter 5 Cyclic behaviour of biogas desaturated sand	113
5.1 Introduction	113
5.2 Materials and methods	114
5.2.1 Denitrifying bacteria and soil desaturation solution	114
5.2.2 The test set-up	114
5.2.3 Laminar box	115
5.2.4 Shaking table	116
5.2.5 Sample preparation	119
5.2.6 Instrumentations	120
5.3 Test results	124
5.3.1 Test conditions	124
5.3.2 Pore water pressure generation	124
5.3.3 Volumetric strain	129
5.3.4 Sinking of structures in sand	131
5.3.5 Response of the laminar box	134
5.4 Discussions	137
5.4.1 Pore pressure and volumetric change	137
5.4.2 Pore pressure and sinking of structures	138
5.4.3 Relationship between model test results and field performances	140
5.5 Conclusions	141
Chapter 6 Morphology and stability of biogas in sand	143
6.1 Introduction	143
6.2 Methodologies	143
6.2.1 Gas observation using the photo imaging method	143
6.2.2 Sand observation using the computer tomography (CT) technique	146
6.2.3 Gas stability test	147
6.3 Results and discussions	150
6.3.1 Observation of gas using the photo imaging	150
6.3.2 Computer tomography (CT) observation of sand samples	157

6.3.3 Gas stability in a sand column	165
6.4 Conclusions	171
Chapter 7 Conclusions and recommendations	173
7.1 Conclusions	173
7.2 Recommendations	177
References	179

SUMMARY

Many existing liquefaction mitigation methods such as compaction and cement mixing are usually too expensive to be applied to a large area. An attempt to develop a more cost-effective method for mitigation of soil liquefaction, the so-called biogas desaturation method, was made in this study. The liquefaction potential of fully saturated sand can be greatly reduced when the sand is made slightly unsaturated by the inclusion of gas.

The microbial denitrification process, which is able to produce nitrogen (N_2) gas, was selected for the desaturation of liquefiable sand. N_2 gas is suitable for this purpose because it is inert and its solubility in water is low. A preliminary study on the biogas desaturation process showed that the degree of saturation of initially fully saturated sand could be effectively reduced to as low as around 80% in 4 days.

Triaxial undrained compression and extension tests were carried out on loose and medium dense sands. For loose sand, the undrained shear strength showed a significant improvement when Skempton's pore water pressure coefficient B-value reduced from around 0.95 (fully saturated) to around 0.3 and other conditions were the same. There was also a change in the stress-strain behavior from complete strain softening to strain hardening with the decrease in the degree of saturation. For medium dense sand, reduction in the degree of saturation led to an increase in the yielding stress and a decrease in the maximum pore water pressure.

A fully instrumented laminar box model with a shaking table was adopted to conduct model tests to verify the effectiveness of the biogas method under cyclic

load. Liquefaction occurred for saturated samples at loose states under $a_{max}=0.5$ m/s^2 and at medium dense states under $a_{max}=1.5$ m/s^2 . Liquefaction did not occur for biogas desaturated samples with the degree of saturation of 80% to 95% at the same conditions.

Both a photographic system and a computer tomography (CT) system were used for the observation of gas distributions in sand. It was observed that gas bubbles were either in the single bubble form or in the form of gas pocket with small gas bubbles aggregating together.

A sand column test was conducted to study the stability of gas bubbles in sand under both hydrostatic and flow conditions. Under the hydrostatic condition, there was almost no change in the degree of saturation. When water flows under a hydraulic gradient of 0.1 through sand, it took 3 to 5 days for desaturated sands with the degree of saturation of 89% to gradually become fully saturated. This implies that the stability of the gas bubbles is affected by the groundwater flow.

LIST OF TABLES

2.1	Factors affecting cyclic undrained behaviour of soils (after JPHI (1997))	22
2.2	Liquefaction countermeasures (after Chu et al. (2009))	27
2.3	Potential application of microbial methods in geotechnical engineering	46
3.1	Results of batch experiments	59
3.2	Results of gas generation tests	68
4.1	Typical engineering properties of ASTM graded sand	74
4.2	Triaxial testing programme	82
4.3	The triaxial test results for medium dense samples	105
5.1	Degree of saturation determined by various methods in the laminar box test	120
5.2	Test conditions and test results of the laminar box test	123
6.1	Degree of saturation and void ratio of samples for gas observation tests	146
6.2	Initial conditions of samples in gas stability test	149

LIST OF FIGURES

2.1	Illustration of critical state concept	7
2.2	Illustration of infinite numbers ICL for sands	7
2.3	Characterization of various undrained manners under triaxial compression	10
2.4	The definition of state parameter	12
2.5	Comparisons of samples with the same state parameter and relative density (after Been and Jefferies (1985))	12
2.6	Behaviour of undrained plane-strain tests on loose sand conducted under deformation-controlled and load-controlled loading conditions (after Wanatowski, 2005)	19
2.7	The influences of void ratio and effective confining stress on instability line (after Chu et al. (2012a))	20
2.8	Relationship between the void ratio and the slope of instability line (after Chu et al., 2012a)	21
2.9	Pore pressure generation models (after Chang et al. (2007))	25
2.10	Test data for cyclic strength of sands (after Yang et al. (2004))	32
2.11	Relationship of cyclic strength and pressure wave velocity for partially saturated sands (after Tsukamoto et al. (2002))	40
3.1	Microscopic image of cells of isolated strain	56
3.2	Colony morphology of isolated strain on spread plate	56
3.3	Set-up of gas generation test	58
3.4	Dynamics of nitrate concentrations in the tests with different electron donors	61
3.5	Dynamics of nitrite concentrations in the tests with different electron donors	62
3.6	Dynamics of OD ₆₀₀ of bacterial suspension in the tests with different electron donors	63
3.7	Dynamics of pH in the tests with different electron donors	64
3.8	Dynamics of total organic carbon (TOC) in the tests with different electron donors	65
3.9	Gas generation curve in the tests with sand (a) and without sand (b)	67

4.1	Image of sand particles of ASTM graded sand	74
4.2	Particle size distribution	75
4.3	Picture of the gas tube method	78
4.4	Triaxial testing system	79
4.5	Picture of triaxial test apparatus	79
4.6	Design of loading pistol and top cap of triaxial extension apparatus	80
4.7	Relationship between pore pressure coefficient B-value and degree of saturation	85
4.8	Undrained behaviour of desaturated loose sands in triaxial compression	88
4.9	Undrained behaviour of desaturated loose sands ($e \approx 0.77$) in triaxial extension	91
4.10	Undrained behaviour of desaturated loose sands ($e \approx 0.71$) in triaxial extension	93
4.11	Undrained behaviour of gas tube samples	95
4.12	Comparison of undrained behaviour of desaturated loose sands ($e \approx 0.77$) in triaxial compression and extension tests	97
4.13	Normalized undrained shear strength for loose samples in compression and extension tests	98
4.14	Pore water pressure ratio for loose samples in compression and extension tests	98
4.15	Instability lines (IL) in compression and extension tests	100
4.16	Undrained behaviour of desaturated medium dense sands in triaxial compression	104
4.17	Relationship between the yielding strength and the B-value	105
4.18	Relationship between pore water pressure ratio r_u and pore pressure coefficient B-value	106
4.19	Undrained behaviour of desaturated medium dense sands in triaxial extension	109
4.20	Comparison of undrained behaviour of desaturated medium dense sands in triaxial compression and extension tests	110
5.1	Model test set-up	114
5.2	Design of the laminar box and instrumentation of transducers	115
5.3	The plastic sheet liner that holds the sand sample	116
5.4	Design of the manual shaking table	117
5.5	Plot of frequency against payload of the shaking table	118

5.6	A typical input acceleration (Test 5c)	118
5.7	Installation of pore water pressure transducers and LVDT	121
5.8	Arrangement of laser displacement transducers for lateral displacement measure of laminar box	121
5.9	The arrangement of laser displacement transducers for the measure of structure sinking	122
5.10	Pore water pressure in the laminar box test	127
5.11	Plots of pore water pressure ratios against relative densities	128
5.12	Volumetric strain for tests with various degrees of saturation	130
5.13	Plot of volumetric strains against relative densities	130
5.14	Sinking of structures	133
5.15	Acceleration amplification coefficient versus relative densities	134
5.16	Lateral displacement	136
5.17	Lateral displacements at one of the peak points of shaking waves	137
5.18	Plot of volumetric strains against pore pressure ratios	138
5.19	Plot of sinking of structures against pore pressure ratios	139
5.20	Relationship between volumetric strain and induced pore water pressure ratio in the field condition (after Tokimatsu and Seed, 1987)	140
6.1	Sample for the observation of gas morphology	145
6.2	Sample images of sands	145
6.3	Set-up of gas stability test	148
6.4	Illustration of the set-up of gas stability test	149
6.5	Images of sand with $S_r = 94.4\%$ and $e = 0.69$	152
6.6	Images of sand with $S_r = 90.6\%$ and $e = 0.68$	153
6.7	Images of sand with $S_r = 87.9\%$ and $e = 0.69$	154
6.8	Images of sand with $S_r = 84.1\%$ and $e = 0.68$	155
6.9	Plot of degree of saturation determined by imaging method and volume change method	157
6.10	Illustration of CT images	158
6.11	CT images of the sample with $S_r=100\%$ and $e=0.63$	161
6.12	CT images of the sample with $S_r=97\%$ and $e=0.64$	162
6.13	CT images of the sample with $S_r=94\%$ and $e=0.66$	163
6.14	CT images of the sample with $S_r=88\%$ and $e=0.66$	164
6.15	Change in the visual observation of saturated zone	166
6.16	Change in overall degree of saturation	167
6.17	Change in the coefficient of permeability	168

6.18	Change in the void ratio	169
6.19	Plot of gas content against flow quantity	170

LIST OF SYMBOLS

A	Skempton's pore pressure coefficient A
a_{max}	Maximum acceleration
B	Skempton's pore pressure coefficient B
b	Intermediate principal stress coefficient
C	Concentration of certain substance in water
C_w	Compressibility of water
D	Coefficient of diffusion
D_r	Relative density
e	Void ratio
e_{cs}	Void ratio at critical state
f	Frequency
g	Gravitational acceleration
I_1	The first invariant
I_B	Brittleness index
I_p	State pressure index
I_s	State Index
J	Diffusive flux
k	Coefficient of permeability
m	Mass
m_v	Volumetric compressibility coefficient
n	Porosity
p'	Mean effective stress
q	Deviatoric stress
r_d	Stress reduction coefficient for the calculation of CSR
r_u	Pore water pressure ratio
S_r	Degree of saturation
s_r	Residual strength
s_u	Peak strength
u	Pore water pressure
$\bar{u}$	Absolute pore water pressure
v	Specific volume
V_p	Pressure wave velocity
V_s	Shear wave velocity
Γ	Critical state parameter, $=v + \lambda \ln(p')$

ν	Poisson's ratio
ν_b	Poisson's ratio of soil skeleton
$\sigma_1, \sigma_2, \sigma_3$	The first, the second and the third principal stresses
σ_c'	Effective confining stress
σ_{v0}	Total over-burden stress
σ_{v0}'	Effective over-burden stress
ε_V	Volumetric strain
η_{IL}	Slope of instability line
λ	Critical state parameter, the slope of CSL in $e-p'$ space
M	Critical state parameter, the slope of CSL in $p'-q$ space
τ_{av}	Average shear stress
ψ	State parameter
CPT	Cone penetration test
CRR	Cyclic resistance ratio
CS	Critical state
CSD	Constant shear drained test
CSR	Cyclic stress ratio
CSL	Critical state line
CT	Computer tomography technique
FS_L	Factor of safety of liquefaction
ICL	Isotropic consolidation line
IL	Instability line
LVDT	Linear variable differential transformer transducers
MICP	Microbial induced calcite precipitation method
MT	Moist tamping sample preparation method
OCR	Over-consolidation ratio
OD	Optical density
PCR	Polymerase chain reaction
PT	Phase transformation
QSS	Quasi-steady state
SPT	Standard penetration test
SS	Steady state
SSL	Steady state line
TOC	Total organic carbon
WS	Water sedimentation sample preparation method

Chapter 1

INTRODUCTION

1.1 Background

Soil liquefaction is a phenomenon that soil loses most of its strength and large deformation occurs in a relatively short duration. Liquefaction is one of the most catastrophic geo-hazards. It can cause severe damages, such as slope failure, loss of bearing capacity, excessive ground subsidence, dislocation of retaining wall, and floating of buried structures. Besides earthquake, other causes can also lead to soil liquefaction, such as static loading, fluctuation of tide, and machine vibration.

A wide range of techniques have been developed for the prevention or the mitigation of soil liquefaction. There include densification, solidification, pore pressure relief, lowering ground water table, and restraint of shear deformation. These techniques have been proved to be effective in practice. However, costs of most of these techniques are high. Thus, it is normally too expensive to apply the existing liquefaction mitigation technique for a large area.

It has been observed in previous studies that gas bubbles in soil lead to an overestimation of its shear strength in cyclic triaxial tests (Martin et al., 1975; Sherif et al., 1977; Chaney, 1978; Yoshimi et al., 1989b; Xia and Hu, 1991). In recent years, some researchers began to explore the potential of using desaturation as a means to reduce liquefaction potential (Pietruszczak et al., 2003; Okamura and Soga, 2006;

Okamura and Teraoka, 2006; Okamura et al., 2006; Yegian et al., 2006; Yegian et al., 2007). This so-called desaturation or gas induction technique involves the use of gas bubbles in soil to alleviate the pore pressure build-up. In this study, an attempt to develop a new desaturation technique, the so-called biogas method, is made. In this method, the microbial denitrification process is adopted to generate gas bubbles in sand to lower down the degree of saturation of the originally saturated sand.

Application of biotechnology in geotechnical engineering is a relatively new topic. Most of the studies have focused on either the enhancement of the shear strength and stiffness of soil (DeJong et al., 2006; Whiffin et al., 2007; Ivanov and Chu, 2008; Harkes et al., 2010) or the reduction of soil permeability (Stabnikov et al., 2011; Chu et al., 2012) so far. Materials that can be used for these purposes are termed biocement. Several types of biocement have been adopted. The most common approach is the urea hydrolysis method. The biogas method is another branch of the microbial application in geotechnical engineering which is intended to mitigate the liquefaction potential of soils (Rebata-Landa and Santamarina, 2012; He et al., 2013). This is also the theme of this thesis. The major concerns of the paper Rebata-Landa and Santamarina (2012) are the correlations between gas content, soil type, nutrient availability, and elastic wave velocity. This thesis, however, covers a wide range of technical issues regarding the biogas method, with emphasis on the undrained behaviour and the seismic performance of biogas desaturated sands.

1.2 Objectives

The objectives of this study are:

- 1) To explore the feasibility of using the microbial denitrification process for gas generation in sand using laboratory microbial experiments;
- 2) To study the stress-strain behaviour of biogas desaturated sands using triaxial undrained compression and extension tests;

- 3) To study the liquefaction response of biogas desaturated sand under dynamic load using a laminar box and shaking table testing system;
- 4) To investigate the morphology of gas bubbles in sand using a photographic system and a computer tomography system;
- 5) To study the stability of gas bubbles in biogas desaturated sand under hydrostatic and seepage flow conditions.

1.3 Scope of study

The scope of this study includes:

- 1) Select denitrifying bacteria and investigate the conditions that favor the denitrification reaction.
- 2) Carry out triaxial undrained compression and extension tests on loose sands and medium dense sands with various degrees of saturation to study the effects of degree of saturation on the undrained behaviour of sand.
- 3) Design a fully-instrumented laminar box and shaking table testing system and use it to conduct model tests on sands with different degrees of saturation and void ratios.
- 4) Set-up a photographic system using a camera with a macro lens and use it to observe the morphology of the biogas desaturated sand. Adopt a computer tomography (CT) system to observe the gas bubble distributions in biogas desaturated sand.
- 5) Set-up a sand column testing system to study the gas bubbles stability in sand under upflow, downflow, and hydrostatic conditions.

1.4 Outline of thesis

This thesis comprises 7 chapters. Chapter 2 presents a literature review of previous studies on soil liquefaction and application of microbial methods to geotechnical engineering. A preliminary study on the gas generation process using microbial

denitrification is given in Chapter 3. A study on the behaviour of biogas desaturated sands using triaxial undrained tests is presented in Chapter 4. Results of model tests using a fully instrumented laminar box and shaking table testing system are discussed in Chapter 5. Chapter 6 covers the study on the morphology of gas bubbles in sand using a photographic system and computer tomography (CT) technique and the study on the stability of gas in sand under flow and hydrostatic conditions using a sand column set-up. The conclusions and the recommendations for future studies are made in Chapter 7.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

This chapter is a literature review of two topics related to this study: one is soil liquefaction and its countermeasures; the other is biological methods in geotechnical engineering. In the first topic, a wide range of previous studies regarding soil liquefaction are reviewed, including: the critical state soil mechanics which provide a theoretical framework for the subsequent discussion, the undrained behaviour of sand under both quasi-static and cyclic loading conditions, liquefaction assessment methods, commonly used liquefaction countermeasures, the undrained behaviour of desaturated sand, methods for the desaturation of sand, and techniques for the detection of gas in sand. In the second topic, reviews of previous studies involve parts: the review of biocement, and the assessment of the potential of using microbial denitrification for sand desaturation.

2.2 Critical state soil mechanics framework for sand

Void ratio plays a key role in the mechanical behaviour of particulate materials such as soils. It is well known that the stress-strain behaviour of the same type of soil at different void ratios differs significantly. Dense materials tend to dilate while loose ones tend to contract. Thus, it is natural to think that there exists a critical void ratio at which soil will not either dilate or contract. The concept of critical state was thus introduced to describe a state at which all the stresses and volume will not change any

more. A framework based on the concept of critical state and critical state soil mechanics was established in 1960s (Schofield and Wroth, 1968). Using this framework, critical state-based soil models, such as Cam Clay model, have been developed. These soil models have expanded our ability to handle complicated problems by invoking the intrinsic properties of soils. To date, the critical state framework is widely used in many aspects of the studies of soil mechanics. Soil liquefaction can also be studied in the light of critical state soil mechanics.

The critical state line (CSL) on the p' - q plane and v - $\ln(p')$ plane as shown in Figure 2.1 can be expressed by Equations 2.1 and 2.2, as:

$$q = Mp' \quad (2.1)$$

$$\Gamma = v + \lambda \ln(p') \quad (2.2)$$

Where, q : deviatoric stress; p' : mean effective stress; v : specific volume ($=1+e$) (e is the void ratio); and M , λ , and Γ are critical state parameters. M in Equation 2.1 is influenced by intermediate principal stress and this influence can be expressed as a function of Lode angle (Cornforth, 1964; Bishop, 1966; Jefferies and Shuttle, 2002). The parameter Γ equals to the value of v when $p'=1$. The parameter λ in Equation 2.2 is an indication of soil compressibility and has a similar meaning to compression index C_c obtained in oedometer tests. Although Equation 2.2 is a straight line in the v - $\ln(p')$ plot, some researchers demonstrated that the CSL on e - $\ln(p')$ plane was curved and became steeper at an effective mean stress higher than around 1 MPa for quartz sands (Been et al., 1991; Lade and Yamamuro, 1996; Yamamuro and Lade, 1996). The curvature of CSL is thought to be due to the particle breakage.

Figure 2.1 Illustration of critical state concept

(a) CSL on q - p' plane; (b) CSL on v - p' plane

Noting that, in Figure 2.1(b), there is an isotropic consolidation line (ICL) that is parallel to CSL. However, the existence of a unique ICL for sands is questionable (Jefferies and Been, 2000). Some experimental results have shown that there can be infinite numbers of ICL within a certain range in the e - p' space for sands as shown in Figure 2.2 (Ishihara et al., 1975; Jefferies and Been, 2000). The implication of infinite number of ICL is that the void ratio and the over-consolidation ratio (OCR) are two independent parameters for sands, and the relationship of undrained strength and OCR for clay soil cannot be applied to sands (Jefferies and Been, 2000). The influence of void ratio on undrained behaviour of sands will be discussed in Section 2.3.

Figure 2.2. Illustration of infinite numbers ICL for sands

The concept of steady state was firstly defined by Poulos (1981) as a state of continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant velocity. There was an argument whether or not steady state line (SSL) and critical state line (SCL) were coincident (Casagrande, 1975; Sladen et al., 1985; Alarcon-Guzman et al., 1988). SSL is usually obtained in undrained test on loose samples, while SCL usually uses drained test. Although the definitions of steady state and critical state are not the same, some researcher pointed out that the two concepts are essentially the same (Been et al., 1991; Chu, 1995). Besides, the term “constant velocity” as used in the definition of steady state has never been experimentally showed in any tests (Jefferies and Been, 2006). Therefore, equivalence of these two concepts is usually assumed at least for practical purposes.

Another controversy is whether or not CSL is unique for a specific sand on the $e-p'$ space. The discussion over the uniqueness of CSL is important because the CSL controls whether the sand at a certain void ratio displays a contractive or dilative behaviour in undrained shear. This in turn influences the liquefaction susceptibility. Many experimental studies have shown that the CSL is a unique line on the $e-p'$ space regardless of fabric, anisotropy conditions, confining pressure, drainage conditions, loading modes, and OCR (Dobry et al., 1985; Poulos et al., 1988; Been et al., 1991; Ishihara, 1993; Chu, 1995; Verdugo and Ishihara, 1996; Ishihara et al., 1998; Tsukamoto et al., 1998; Chen and Liao, 1999; Yoshimine et al., 1999). However, some researchers argued that the SCL was not unique and influenced by initial density and stress level (Konrad, 1993), sample preparation methods (Vaid et al., 1990; Kuerbis and Vaid, 1988), stress path (Vaid et al., 1990; Negussey and Islam, 1994; Vaid and Thomas, 1995), drainage condition (Alarcon-Guzman et al., 1988), strain rate (Hird and Hassona, 1990), and consolidation history (Finno and Rechenmacher, 2003). Jefferies and Been (2006) analyzed these disputes, and have concluded that there may be flaws in these results: (1) shear band may arise and sand

may not reach the CSL at the end of the test; (2) steady state and pseudo steady state are confused. Thus, we can say that the uniqueness of the CSL can be assumed regardless of these factors within experimental precision.

2.3 Undrained behaviour of sandy soils

Liquefaction is often considered to take place under an undrained condition. The undrained behaviour of sand under both quasi-static and cyclic loading conditions is reviewed in this section.

2.3.1 Undrained response of sand under triaxial compression

Figure 2.3 shows five patterns of undrained responses that are usually seen in sand triaxial compression tests. From (a) to (e), sands behaves from a complete dilative manner to a complete contractive manner. Pattern (a) exhibits an increase of mean effective stress all the way to CSL. Pattern (b) firstly displays a decrease in mean effective stress, then the stress path turns to an increase trend in p' along CSL, giving an turning point on stress-strain curve. This turning point is defined as phase transformation by Ishihara et al. (1975). Pattern (c) experiences a temporary contractive manner after peak deviatoric stress then turning back to a dilative manner. This lowest turning point is defined as quasi-steady state by Alarcon-Guzman et al. (1988). Pattern (d) shows a contractive manner before reaching the final critical state. Pattern (e) displays a contractive manner all the way to zero deviatoric stress, indicating a complete liquefaction condition. It needs to be mentioned that: (1) although critical states are indicated on all five figures, in real triaxial undrained tests, critical state may not be reached due to the development of non-homogeneous deformation, such as shear band for relatively dense sand and necking in extension tests; (2) Normal triaxial tests should be proceeded to an axial strain of 20%, but this strain level may not be enough to bring the sand to a critical state.

Figure 2.3. Characterization of various undrained manners under triaxial compression

Using critical state soil mechanics framework, whether sand manifests a contractive or dilative manner in triaxial undrained compression can be explained by its relative location to the CSL on the $e-p'$ plane: when the point is located above the CSL, soil will behave a contractive manner, and when the point is below the CSL, it will show a dilative manner. Thus, it is natural to use the CSL as a reference to delineate the undrained behaviour of sand. Several parameters have been proposed in the literature. The state parameter proposed by Been and Jefferies (1985) is defined as the distance between the initial void ratio and the void ratio on CSL at the same mean effective stress as shown in Figure 2.4,

$$\psi = e - e_{cs} \quad (2.3)$$

A point located above the CSL has a positive value of state parameter and shows contractive manner, and soil located below the CSL has negative state parameter and shows dilative manner. Two series of undrained tests are shown in Figure 2.5 to validate the concept of state parameter. In Figure 2.5, I_1 is the first stress invariant (i.e. mean effective stress) and I_{ss} is the first stress invariant on the SSL at the same void ratio. Thus, I_1/I_{ss} is actually proportional to the state parameter for a specific sand (Been and Jefferies, 1985). As can be seen in Figure 2.5(a), Test 103 and Test 108 have similar value of I_1/I_{ss} (i.e. state parameter), but neither void ratio nor initial confining pressure is the same, these two tests show a similar pattern of undrained shear which is like Pattern (b) in Figure 2.3. In contrast, Test 37 and Test 103 have the same void ratio while different state parameters, and they display completely different undrained shear patterns. Same manner is also found in Figure 2.5(b). It is worth noting that Test 103 has a relative density of 33% and shows a non-liquefiable undrained behaviour, while Test 112 has a relative density of 75% and shows a liquefiable behaviour. This implies that it is by no means correct to use relative density as an undrained behaviour indication.

Figure 2.4. The definition of state parameter

Figure 2.5. Comparisons of samples with the same state parameter and relative density (after Been and Jefferies (1985))

Other parameters that are similar to the state parameter include state index and state pressure index. Ishihara (1993) and Ishihara et al. (1998) defined the state index as:

$$I_s = \frac{e_0 - e_c}{e_0 - e_s} \quad (2.4)$$

e_c is the initial void ratio, e_s is the void ratio on CS or QSS line at the same mean effective stress, e_0 is a reference void ratio and can be seen as the critical state void ratio at $p'=0$.

Wang et al. (2002) proposed another parameter, the state pressure index, which is defined as:

$$I_p = \frac{p'}{p_c'} \quad (2.5)$$

p' is the initial confining pressure and p_c' is the critical mean effective stress at the same void ratio. Geometric analysis can easily show that state pressure index is actually proportional to state parameter for a given sand.

Both state index and state pressure index can serve a very similar purpose to state parameter and can be used as indications of stress-strain behaviour during undrained shear.

2.3.2 Other influential factors on undrained behaviour of sand

Stress path and intermediate stress

In addition to the conventional triaxial compression tests, other stress paths and stress conditions can be imposed by the triaxial extension test, the simple shear test, the plain strain test, and the torsional shear test. In the torsional shear test, three principal stresses can be controlled, thus various stress paths can be attained. Torsional shear test is widely used in the study of constitutive behaviour of soils. A simple parameter

that helps differentiate these types of test is intermediate principal stress coefficient, which is defined as:

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad (2.6)$$

The range of b is between 0 and 1. Triaxial compression and extension tests are two extreme conditions, having values of 0 and 1, respectively. Simple shear test and plain strain test have a value of b in-between. By comparing the results in triaxial compression, triaxial extension, simple shear, and torsional shear tests, Yoshimine et al. (1998) observed that soil manifested a more contractive manner with the increase of b .

Fabric and anisotropy

Conventionally, sample preparation induced initial fabric is thought to have a less important effect on the soil behaviour. However, some results show that soil with the same void ratio but different fabrics can show a significant discrepancy in the undrained behaviour. Some results show that sand samples prepared by different methods shows differences in stiffness (Arthur and Menzies, 1972; Oda, 1972; Oda et al., 1980; Miura et al., 1984; Been and Jefferies, 1985). Samples prepared by the moist tamping method with more random particle arrangement usually give stiffer stress-strain behaviour than those prepared by the water sedimentation method with more regular alignment of particles. The undrained behaviour of sand is also strongly influenced by the angle of maximum principal stress to the bedding plane (the plane perpendicular to the direction of deposition). The larger the angle, the higher the strength for both undrained (Yoshimine et al., 1998) and drained cases (Oda, 1972; Tatsuoka, 1987; Wan and Guo, 2004). This reason partly accounts for the contractive behaviour in triaxial extension tests compared with compression tests (Yoshimine et al., 1998).

Fines content, gradation, particle shape, and mineralogy

The effect of fines content on the undrained response of sand has been widely studied. Whether sand shows a higher or lower liquefaction resistance with fines content is controversial. Instead of using overall void ratio, some researchers preferred to use intergranular void ratio e_s to characterize the undrained response. Thevanayagam (1998) showed that: if e_s is smaller than the maximum void ratio of the host sand (same sand but without fines), undrained response is similar to host sand; if e_s is larger than maximum void ratio of host sand, the undrained strength is low and depends on the initial confining stress; if the fines content is higher than a value of about 30%, soil behaves in a manner similar to that with pure fine particles. Pitman et al. (1994), Verdugo and Ishihara (1996), and Murthy et al. (2007) demonstrated that fines content gave a downward shift of CSL on $e-p'$ plane, thus soil became more susceptible to liquefaction. While Pitman et al. (1994) and Murthy et al. (2007) also stated that soils showed a higher critical state friction angle and alleviated strain softening behaviour with fines content. Yamamuro and Lade (1997b), Yamamuro and Lade (1998), Yamamuro and Covert (2001), , Shapiro and Yamamuro (2003), and Ng et al. (2004) show that loose sand with fines content can manifest a “reverse” manner, that is, soil at low confining pressure is more liable to liquefaction. They explained that the presence of fines between contact points of granular particles gave a more contractive manner. However, this so-called “reverse” behaviour may be just because of the use of different consolidation pressures.

Moreover, it is not only the fines content that influences the undrained response. Gradation, particle morphology, and mineralogy also have effects. Jefferies and Been (2006) pointed out that hard, round, quartz sand is less compressible and has a low value of λ , and crushable, angular sand is more compressible and has a high value of λ . The CSL of well graded sand in the $e-\ln p'$ plane is usually located below that of uniformly graded sand. Santamarina and Cho (2004) and Cho et al. (2006) showed

that increased angularity led to a decreased stiffness, higher critical state friction angle, and increased I , but weak effect on λ . Kokusho et al. (2004) showed that the critical state strength (residual strength) for well-graded sand were at least 8 times higher than that for poorly-graded sand under undrained monotonic loading, indicating the significance of gradation in liquefaction susceptibility.

2.3.3 Peak strength and residual strength

The selection of proper soil strength at different stages of liquefaction analysis is important. Generally, the peak strength is used in pre-liquefaction analysis and the residual strength is used in post-liquefaction analysis. For dense sand that shows dilative manner all the way to the end of the test, the residual strength is the same as the peak strength. For sand that shows strain softening manner and quasi-steady state, critical state strength and QSS strength is often taken as residual strength, respectively.

Ishihara (1993) gave the relationship of peak deviatoric stress as:

$$\frac{q_p}{p_c'} = M_L \frac{p_p'}{p_c'} + (M - M_L) \frac{p_s'}{p_c'} \quad (2.7)$$

Where the subscript p denotes peak, the subscript s denotes SS or QSS, the subscript c denotes initial state, and M_L is the inclination of the collapse line as defined by Sladen et al. (1985). By substituting the tested soil parameters into the above equation, a simple expression of peak strength for Toyoura sand can be written as:

$$s_u / \sigma_0' = 0.17 + 0.22I_s \quad (2.8)$$

Where I_s is the state index as defined in Equation 2.4.

The residual strength is defined by some studies (Castro, 1987; Dealba et al., 1988; Marcuson et al., 1990; Ishihara, 1993) as:

$$s_r = \frac{q_s}{2} \cos \phi_s = \frac{M}{2} \cos \phi_s (p_s') \quad (2.9)$$

Wroth (1984) proposed a relationship between the residual strength and OCR, which became widely used for the determination of residual strength of clay. By using a similar form, Jefferies and Been (2006) gave the equation of residual strength for sand as:

$$\frac{s_r}{p_0'} = \frac{M}{2} \exp\left(-\frac{\psi_0}{\lambda}\right) \quad (2.10)$$

Jefferies and Been (2006) pointed out that Equation 2.10 is only applicable to loose sand; while for dense sand strain localization often plays a dominant role in residual strength and Equation 2.10 tends to overestimate the strength, which agrees with the experimental results (Konrad, 1990a, 1990b; Yoshimine et al., 1999).

The brittleness index I_B proposed by Bishop can help describe the drop of strength after peak:

$$I_B = \frac{s_u - s_r}{s_u} \quad (2.11)$$

Hird and Hassona (1990) and Jefferies and Been (2006) showed an overall trend of increasing brittleness index with an increase of contractive manner of soils. Thus, the smaller the I_B , the stronger the liquefaction resistance.

2.3.4 Instability of sands

Other than the static liquefaction that is usually observed in strain-controlled tests, another type of the collapse of sand, named instability, is found to occur in load-controlled condition. Instability is defined as a condition where large strains are generated due to the instability of soil carrying a given load (Lade et al., 1987; Lade et al., 1988; Lade and Pradel, 1990; Chu et al., 1993; Yamamuro and Lade, 1997a; Chu and Leong, 2001; Chu and Wanatowski, 2008, 2009). It was reported that

instability should account for some slope failure cases, e.g. Aberfan slide (Bishop, 1971; Bishop, 1973) and Nerlerk Berm failure (Lade, 1993). In real soil grounds, load-controlled condition is much more commonly seen than deformation-controlled condition. The stress-strain responses of soils in both conditions are different. A comparison between the undrained responses of loose sand under load-controlled condition and deformation-controlled condition is shown in Figure 2.6 (Wanatowski, 2005). Before reaching the peak, stress-strain behaviour at both loading conditions are almost the same. After the peak, the stress-strain responses are rather different: the deformation-controlled test shows a strain-softening manner with a moderate stress reduction rate; the load-controlled test shows a rapid runaway collapse. The residual strengths at both conditions are of much difference, the residual strength in load-controlled test is much smaller than that in deformation-controlled test. Chu (2007) suggested that, instead of using post-liquefaction strength, peak strength should be used to characterize the liquefaction susceptibility of soils.

Instability of soils occurs when strain goes beyond the peak deviatoric stress. Instability line is thus defined as the line that connects the peak deviatoric stress and the original point in $p'-q$ stress path (Lade and Pradel, 1990; Ishihara, 1993; Chu and Leong, 2002). Instability line is also named collapse surface by Sladen et al., (1985). Sand at a stress state above the instability line is susceptible to instability irrespective of drainage conditions (Chu et al., 2003; Chu et al., 2012a). It is reported that instability can happen under drained conditions along a constant shear drained (CSD) path with reducing mean effective stress and constant shear stress (Chu et al., 2003). Chu et al. (2003) also reported that under a CSD path instability can even lead to the collapse of dilative sand.

Figure 2.6. Behaviour of undrained plane-strain tests on loose sand conducted under deformation-controlled and load-controlled loading conditions (after Wanatowski, 2005)

Although the manifestations of instability and strain softening are apparently different, Chu and Leong (2001) pointed out that, except loading modes, the mechanisms for strain softening and instability are the same. Thus, instability and static liquefaction should be studied together. Since there is no difference in the stress-strain behaviour between deformation-controlled and load-controlled tests before reaching the peak stress, it is easy to determine the instability line using conventional deformation-controlled tests. The slope of instability line increases with the decrease in void ratio (Figure 2.7(a)), but is less affected by the stress within a certain stress range (Figure 2.7(b)). The relationship between the void ratio and the slope of instability line is further illustrated in Figure 2.8, which clearly shows that

the susceptibility of sand to instability can be alleviated by densification.

(a)

(b)

Figure 2.7. The influences of void ratio and effective confining stress on instability line (after Chu et al. (2012a)) (a) samples with different void ratios; (b) samples with different effective confining stress

Figure 2.8. Relationship between the void ratio and the slope of instability line (after Chu et al., 2012a)

2.3.5 Undrained behaviour under cyclic loading

In addition to the liquefaction under quasi-static loading conditions, the cyclic induced liquefaction is more frequently encountered and often led to catastrophic consequences. Fundamentally, the mechanisms that cause static liquefaction and cyclic liquefaction are essentially the same: the plastic volumetric strain develops so quickly that the pore water cannot escape, leading to increases of excess pressure. Reasons that lead to the plastic strain are different: for static liquefaction, plastic strain through the stress-dilatancy response being greater than work hardening to support the increased stress is a necessary condition; while for cyclic liquefaction, it is the densification effect that leads to the increase of pore pressure (Jefferies and Been, 2006). It should be noted that the term liquefaction in cyclic loading condition usually refers to a runaway strain failure; while cyclic softening with limited strain is usually termed cyclic mobility.

In laboratory, different types of cyclic tests are used to explore the liquefaction behaviour of soils. These involve cyclic triaxial tests, cyclic simple shear tests, cyclic

torsional shear tests, and shaking table tests. Among these types of tests, the cyclic triaxial test is the most commonly used and serves as a reference in the liquefaction evaluation. Test results from other types of tests can be correlated with the results from cyclic triaxial test. The commonly adopted failure criterion in cyclic triaxial test is to attain either of these two states: 5% double amplitude strain, or pore pressure reaching effective confining pressure. Factors affecting the cyclic shear strength are similar to that affecting static liquefaction, as summarized in Table 2.1(JPHI, 1997).

Table 2.1. Factors affecting cyclic undrained behaviour of soils (after JPHI (1997))

1. Loading conditions	Cyclic shear stress amplitude Waveform Vibrational frequency Irregularity in the waveform Multidirectional shear Reversal of shear stress
2. Soil conditions	Density (relative density) Grain size distribution Degree of saturation Structure of soil skeleton Cyclic shear history Cementation effect
3. Stress conditions	Mean effective stress Over-consolidation ratio Initial shear stress
4. Test conditions	Specimen dimension Specimen preparation method Consolidation time Membrane penetration B-value Type/structure of test equipment

2.4 Liquefaction assessment

2.4.1 An overview of liquefaction assessment procedures

Professor H. B. Seed and his co-workers from University of California at Berkeley proposed a methodology named “simplified procedure” for assessing earthquake-induced liquefaction resistance of soils (Seed and Idriss, 1971). Since then, this methodology has become a standard and a large volume of literatures has been added to this system.

The starting point of liquefaction assessment is the comparison of cyclic strength and mobilized shear stress, which is expressed as:

$$FS_L = \frac{CRR}{CSR} \quad (2.12)$$

CRR (cyclic resistance ratio) is defined as the cyclic strength over the vertical effective stress. CSR (cyclic stress ratio) is defined as the mobilized shear stress over the vertical effective stress. Generally speaking, a factor of safety larger than unit means that soil is free from liquefaction and smaller than unit means that soil is liable to liquefaction. However, a FS_L close to unit may require extra prudent analysis to determine the liquefaction susceptibility.

The determination of CSR in the simplified procedure that initially proposed by Seed and Idriss (1971) is:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \quad (2.13)$$

where a_{max} is the peak horizontal acceleration at ground surface; g is gravitational acceleration; σ_{vo} and σ'_{vo} are total and effective over-burden stress, respectively; r_d is stress reduction coefficient, which indicates that the shear stress underneath is smaller than that on the ground surface due to the deformability of soil column (Ishihara, 1993). The value of a_{max} can be found in related design codes. However, it is also affected by specific geological conditions (Idriss, 1991), so detailed analysis

needs proper selection of a_{max} .

There are several experimental methods for the determination of CRR, including laboratory tests, CPT, SPT, elastic wave velocity. A brief summary of different methods was given by Youd et al. (2001).

2.4.2 Pore pressure prediction

Another important factor with respect to liquefaction is the pore pressure generation. In general, the pore pressure rise during liquefaction equals to the decrease of effective stress according to the effective stress principle. In the literature, two types of models for the prediction of pore pressure generation under cyclic loadings are found. One type aims at obtaining the relationship between pore pressure and number of cycles as shown in Figure 2.9(a) (Seed et al., 1975). The equation is given as:

$$r_u = \frac{u_g}{\sigma'_0} = \frac{2}{\pi} \arcsin\left(\frac{N}{N_l}\right)^{1/2\theta} \quad (2.14)$$

where, r_u is the pore pressure ratio defined as the pore pressure over the initial effective over-burden pressure; N is the number of cycles; N_l is the number of cycles that causes liquefaction; and θ is an empirical parameter ranging from 0.5 to 0.9. The other pore pressure prediction method that correlates the pore pressure generation to shear strain level is shown in Figure 2.9(b) (Dobry et al., 1982). Note that, in Figure 2.9(b), there is a shear strain threshold below which pore pressure cannot be generated. This could imply shear modulus at small strain is larger than that at large strain. By measuring the in-situ pore pressure, Chang et al. (2007) pointed out that the induced shear strain rather than shear stress is more suitable for the simulation of pore pressure generation.

(a)

(b)

Figure 2.9. Pore pressure generation models (a) cyclic stress-controlled tests (Seed et al., 1975) (b) strain-controlled cyclic triaxial tests (Dobry et al., 1982) (after Chang et al. (2007))

2.5 Liquefaction countermeasures

2.5.1 Conventional liquefaction countermeasures

Several approaches have been adopted in practice for the remediation of liquefaction potential. These approaches include compaction, solidification, pore pressure dissipation, lowering of ground water table, and shear strain restraint etc. The

mechanism of each approach is briefly explained here. (1) Compaction is the most widely used method for liquefaction prevention (Japanese Geotechnical Society, 1998). Under shear force, loose soils tend to contract, so that pore water pressure will climb up due to the contracting tendency of soils. Compaction of soils is intended to densify soils in order to convert the contracting tendency to dilating tendency. (2) Solidification approach is to induce cementation effects between soil particles. Thus, soil skeletons will be more resistant to external loadings and deformations. (3) Pore pressure dissipation is intended to provide a pore water draining path (usually vertical drains) so that pore pressure can be relieved easily and effective stress can be preserved in soils. (4) Liquefaction of soils occurs in saturated soils below the ground water table. Therefore lowering of ground water table is another way to reduce the liquefaction susceptibility. (5) In shear strain restraint approach, a continuous underground wall is installed to restrain the shear deformation of soils during earthquakes.

Some commonly used liquefaction remedial measures are summarized in Table 2.2 (Chu et al., 2009). Details of these techniques can be found in liquefaction handbooks and design codes, such as Japanese Port and Harbour Institute (1997) and Japanese Geotechnical Society (1998). However, liquefaction hazards were still seen repeatedly in large and mid size earthquakes in last several decades. More recently, Christchurch Earthquakes in 2010 and 2011 in New Zealand have caused extensive damages in the region. One major reason could be that these liquefaction remediation measures are expensive and cannot be applied to all the liquefiable construction sites. Another reason is that these methods can hardly be applied to liquefiable grounds under existing structures. Innovative techniques against soil liquefaction, such as the biocement method and the gas induction method, have been raised in recent years which can potentially overcome the limitations of existing methods. This part will be reviewed in the rest of this chapter.

Table 2.2. Liquefaction countermeasures (after Chu et al. (2009))

Method Attainable	Most Suitable Soil Types	Effective Depth	Maximum Improvement	Advantages	Limitations
A1. Soil replacement	All soils	A few meters	High density fills to strong, cemented materials, including roller compacted concrete	1) Can design to the desired improvement level; 2) Easy to QA/QC	1) Expensive; 2) May require dewatering; 3) Excavations may impair stability of adjacent ground; 4) Temporary support of existing structures
A2. Lowering of ground water table	Sandy soils	Top few meters	Effective for soil above water table	1) Low cost 2) Simple	1). Limited usage; 2). May cause adjacent effect
B1. Dynamic compaction	Saturated sands and silty sands; partly saturated soils.	Up to 10 m	$D_r = 80\%$; $(N1)_{60} = 25$; $q_{e1} = 10-15$ MPa	1) Low cost; 2) Simple; 3) Good for large areas.	1) Limited effective depth; 2) Clearance required; 3) High mobilization cost; 4) Vibrations can impact adjacent structures.
B2. Vibro-compaction	Sands, silty sands, gravelly sands with < 10% fines	30 m	$D_r = 80+ \%$; $(N1)_{60} = 25$; $q_{e1} = 10-15$ MPa	1) Uniformity with depth in a given soil type; 2) Moderate cost	1) Special equipment needed; 2) Best in clean sand; 3) Unsuitable in soils with cobbles and boulders; 4) Backfill required in most cases
B3. Sand and gravel compaction piles	Can use in most soil types	20 m or the limit of the machine	Up to $(N1)_{60} = 25-30$, $q_{e1} = 10-15$ MPa, depending on soil type	1) Proven effectiveness; 2) Provides drainage and reinforcement; 3) Uniformity with depth.	1) Special equipment needed; 2) Slow; 3) High cost
B4. Vibro-replacement or stone columns	Silty sands, silts, clayey silts; or use vertical drains to enhance effectiveness	30 m	$(N1)_{60} = 20$; $q_{e1} = 10-12$ MPa	1) Provides drainage and reinforcement; 2) Uniformity with depth in a given soil; 3) Bottom feed dry process puts fill where needed.	1) Special equipment needed; Unsuitable in soil with cobbles and boulders; 2) Fines may intermix with and clog columns; 3) Backfill may be costly; 4) Difficult QA/QC
B5. Explosive compaction	Saturated clean sands and gravels	>30 m	$D_r = 75 \%$; $(N1)_{60} = 20-25$ $q_{e1} = 10-12$ MPa	1) Inexpensive; 2) Simple technology; 3) Can use at greater depths; 4) Can use in soils with cobbles and boulders.	1) Vibrations; 2) Safety issues; 3) Psychological barriers.

Table 2.2. Liquefaction countermeasures (after Chu et al. (2009)), continued

Method Attainable	Most Suitable Soil Types	Effective Depth	Maximum Improvement	Advantages	Limitations
C1. Penetration grouting	Sands and coarser materials, rocks	Unlimited	Void and crack filling and solidification	1) No excess pore pressure or liquefaction in treated zone 2) Can localize treatment area.	1) High cost; 2) Excessive fines content prevents use in many soils
C2. Deep cement soil mixing	Most soil types	Can be used to depths >30 m;	Design compressive strengths ranging from 1.0 to 1.4 MPa	1) Positive ground reinforcement; 2) Can contain liquefiable soil within high strength grid walls	1) Requires special equipment 2) Brittle elements; 3) Expensive; 4) Difficult QA/QC
C3. Jet grouting	Unlimited	Almost any soil	Design compressive strengths ranging from 1.0 to 1.4 MPa	1) Controllable treatment depth range; 2) Useful in soils with fines; 3) High strength columns; 4) Large treatment depth possible.	1) High cost; 2) Difficult to QA/QC; 3) Uncertain keying into underlying stratum; 4) Can induce hydraulic fracture in grouted formation
C4. Compaction grouting	Any rapidly consolidating, compressible soil	Unlimited	Up to $D_r=80\%$ ($N1$) ₆₀ = 25 q_{c1} = 10-15 MPa (Soil type dependent)	1) Controllable treatment zone 2) Useful in soils with fines	1) High cost; 2) Post-treatment loss of pre-stress; 3) Slow
D1. Prefabricated vertical drain or EQ drain	All types	Dependin g on the equipment	Reduce excess pore pressure buildup and minimize settlement	1). Quick in installation 2). Higher discharge capacity than gravel drains 3). Can combine with densification during installation	1). Uncertain long-term performance; No case histories yet.
D2. Gravel drains or granular columns	Sands, silty sands	20 m or more	Reduce pore pressure buildup during shaking, Intercept pore pressure plumes	1) Inexpensive; 2) Full area treatment not required	1) Require close spacing; 2) EQ-induced settlement not prevented; 3) Treated ground must have high hydraulic conductivity; 4) Limited performance record

2.5.2 Liquefaction remediation using biocement

Application of biocement is one of new research topics in geotechnical engineering in recent years. One potential application of biocement is liquefaction remediation. On one hand, biocement is usually more suitable for the treatment of sandy soils than clay soils due to the larger pore space in sandy soils which can accommodate microbial activities. On the other hand, liquefaction is a major problem for sandy soils. The reason is that sands have no or little cohesion at the particle contacts, and these contacts are easy to be disturbed during earthquake shaking. The application of biocement in liquefaction remediation is intended to induce cementation into sands so that sand is more resistant to cyclic loadings.

DeJong et al. (2006) reported that biocemented sand showed much higher strength and shear stiffness at the initial stage, followed by gradual reducing trends in deviatoric stress and shear stiffness. This mechanical behaviour was similar to chemically-treated soils, but apparently different from untreated sand. The high strength and stiffness at the initial stage was thought to be due to the cementation effect, and the gradual reducing trend afterwards was due to the degradation of the cementation agent. Regarding the spatial distribution of calcite, DeJong et al. (2010) reported that calcite tended to grow on the contact points between sand particles rather than on the particle surface where there was no particle contact. This could be regarded as a preferential factor for biocement. DeJong et al. (2010) also proved that, during shear, the breakage of cementation was mostly in the calcite rather than between the interface of calcite and sand particles. van Paassen et al. (2010b) conducted a large scale biocement test involving 100 m³ sands. The treatment fluid was injected from three injection wells and pumped from three extraction wells to enhance the extent of treated area. The result showed that most of the sands had a noticeable improvement in strength and stiffness. A more detailed review on biocement will be given in Section 2.7.

2.6 Desaturation as a liquefaction countermeasure

Some experimental data show that that gas bubble inclusion in a triaxial sample can cause a reduction in Skempton's pore pressure coefficient B -value and an overestimation of the cyclic strength of sands (Martin et al., 1975; Sherif et al., 1977; Chaney, 1978; Yoshimi et al., 1989; Xia and Hu, 1991; Tamura et al., 2002; Tsukamoto et al., 2002; Yang, 2002; Pietruszczak et al., 2003; Yang et al., 2004; Altun and Goktepe, 2006; Okamura and Soga, 2006; Bouferra et al., 2007). Here B -value is defined as the ratio of the increase of pore water pressure to the effective confining pressure. There is a relationship between the degree of saturation and the B -value as to be discussed in Chapter 4. It normally requires to achieve a B -value larger than 0.95 in triaxial tests so as to assure full saturation of test samples. Most of the early works intended to correct the erroneous test results of soil with gas bubble inclusion (Martin et al., 1975; Sherif et al., 1977; Chaney, 1978; Yoshimi et al., 1989; Xia and Hu, 1991). It is only recently researchers realized the potential of artificial desaturation as a remedial measures for liquefaction (Pietruszczak et al., 2003; Okamura and Soga, 2006; Okamura and Teraoka, 2006; Okamura et al., 2006; Yegian et al., 2006, 2007; Okamura et al., 2011). These studies will be reviewed in this section.

2.6.1 Effect of gas bubbles on the stress-strain behaviour of sand

Behaviour under cyclic loadings

The strong liquefaction resistance of gas-containing sand compared to the fully saturated sand has been demonstrated by several kinds of test, e.g. cyclic triaxial tests (Xia and Hu, 1991; Ishihara et al., 2001; Tamura et al., 2002; Tsukamoto et al., 2002; Pietruszczak et al., 2003; Okamura and Soga, 2006; Bouferra et al., 2007), cyclic torsional shear tests (Sherif et al., 1977; Yoshimi et al., 1989; Altun and Goktepe, 2006), and shaking table tests (Okamura and Teraoka, 2006; Yegian et al., 2006; Lee and Santamarina, 2007; Yegian et al., 2007; Eseller-Bayat et al., 2009).

Figure 2.10 shows some typical test results of torsional shear tests and triaxial tests. It can be seen that cyclic stress ratio doubles with the decrease of pore pressure coefficient B-value from about 0.95 (i.e. fully saturated) to about 0.2. Yang et al. (2004) suggested that data shown in Figure 2.10 could fit into a curve of normalized liquefaction strength against B-value, which was expressed as:

$$(CRR)_{PS} = (CRR)_{FS} e^{[0.710(1.0-B)]} \quad (2.15)$$

in which subscripts PS and FS denote partial saturation and full saturation, respectively. A more general form is given as:

$$(CRR)_{PS} = (CRR)_{FS} e^{[\alpha(1.0-B)]} \quad (2.16)$$

in which α is a parameter that needs to be calibrated from specific test results.

Another relationship proposed by Okamura and Soga (2006) correlates liquefaction resistance ratio to the potential maximum volumetric strain, which is expressed as:

$$(CRR)_{PS} = (CRR)_{FS} \log(6500\varepsilon_v^* + 10) \quad (2.17)$$

in which ε_v^* is the potential volumetric strain.

Test results from other sources (Huang et al., 1999; Yasuda et al., 1999) with various relative densities seem to fit into Equation 2.17 well. According to critical state soil mechanics, void ratio plays a dominant role in soil strength. Under undrained condition, void ratio is influenced by the volume change of gas bubbles inside due to the change of pore pressure. Thus, it is rational to build the relationship between CRR and volumetric strain, because they are both directly related to void ratio such as Equation 2.17.

Material	D_{50} (mm)	C_u	e_{max}	e_{min}	D_r	Liquefaction criteria	Test type	Reference
Ottawa sand	0.40	2.1	0.76	0.50	—	$EPP = \sigma'_c$	Torsional shear	Sherif et al. (1977)
Toyoura sand	0.175	1.52	0.976	0.605	60%	$DA = 5\%$	Torsional shear	Yoshimi et al. (1989)
Tongjiazhi sand	0.1	3.7	—	—	60%	$EPP = \sigma'_c$	Triaxial	Xia and Hu (1991)
Niigata sand	0.325	1.47	—	—	62%	$DA = 5\%$	Triaxial	Ishihara et al. (2001)

Note: DA=double amplitude of strain; EPP=excess pore pressure; and σ'_c =initial confining pressure.

Figure 2.10. Test data for cyclic strength of sands (after Yang et al. (2004))

The use of shaking table tests can show some effects such as ground subsidence, influences on the structure, and pore pressure distribution, etc, that cannot be seen in the triaxial tests. By conducting the shaking table tests on a sealed model box with reduced atmospheric acting on the soil surface, Okamura and Teraoka (2006) showed

that: (1) the sinking of structure resting on the ground could be substantially alleviated by introducing a small portion of gas bubbles into the sand, the volumetric strain was influenced by degree of saturation and pore pressure; (2) A Strong correlation between volumetric strain and number of cycles to liquefy was identified; and (3) The improvement of liquefaction resistance was less effective on the shallow soil than on the deep soil. By using a strain-controlled shaking table model box with flexible walls, Yegian et al. (2007) showed that a reduction of degree of saturation from 99.1% to 84.2% led to (1) an alleviation of pore pressure generation, and (2) smaller soil surface settlement.

Behaviour under quasi-static loading and instability

Unlike the improvement in liquefaction strength in cyclic loading, the presence of gas bubbles in sand can either strengthen or weaken the undrained strength under monotonic loading. The presence of gas in pore space is either in gas phase or dissolved liquid phase. Soils which contains large amount of dissolved gas are termed gassy soils. Sobkowcz (1984) showed that gassy soils could suffer from a drastic drop in mean effective stress in the unloading path of total stress due to the gas exsolution. This mechanism could possibly account for the seabed slope failure in the Fraser River Delta and some other slope failure cases (Christian and Cranston, 1997; Grozic et al., 1999). Grozic et al. (1999) also showed that, under the conventional undrained loading condition, loose sand with degree of saturation higher than around 90% exhibited a strain softening manner similar to that in saturated sand; while when the degree of saturation was lower than around 90%, loose sand showed a strain hardening manner. Rad et al. (1994) demonstrated that, under undrained monotonic loading, the undrained strength of gassy soils was dependant on the solubility of the specific gas and the amount of gas: the higher the solubility or the larger the amount of dissolved gas, the weaker the undrained strength.

As far as instability of sand concern, Unsaturation of sand can result in more stable behaviour. Experimental results (Lade and Pradel, 1990) and analytical study (Pradel and Lade, 1990) showed that, when the degree of saturation was low enough, the pore pressure generation was sufficiently buffered by the presence of compressible gas to prevent an unstable, runaway condition.

To summarize, the liquefaction strength of sand can be improve by the inclusion of gas bubbles in sand, except for the condition when the type of gas has high solubility in water and the sand tends to be loosened due to the exsolution of gas. Thus, introduction of gas into liquefiable sand is a possible and promising way for liquefaction mitigation.

2.6.2 Methods of desaturation

In order to use desaturation as an engineering method for liquefaction remediation, gas-generation methods should satisfy some requirements: (1) The degree of saturation can be controlled relatively precisely; (2) Uniformity of gas distribution is needed; and (3) Gas bubbles can retain in soil for a relatively long time (this requires the gas to have low solubility, inert chemical behaviour, and stability for staying underground). The following five methods can be found in the literature.

Air injection method

The operation of air injection for the mitigation of soil liquefaction has been experimentally investigated by Okamura et al. (2011). A liquefiable soil ground near quay wall was selected for the study. The air injecting point was around the bottom of the liquefiable layer by inserting a plastic pipe into the ground. Some small holes were drilled on the plastic pipe at the injecting point. Pressurized air was supplied by a flexible tube connecting to the injecting point of the plastic pipe. Test results showed that the desaturation zone was within 4 m from the injecting point. The

degree of saturation was in the range of 68% to 98%, which was almost low enough to double the cyclic strength. The air injecting pressure should be higher than the sum of hydrostatic pressure and capillary pressure to assure that gas bubbles could go into soil. And air injecting pressure should be smaller than the effective stress (vertical or horizontal, whichever was smaller), to avoid cracking or fissuring of soil around injecting point. The air flow rate increased linearly with the increase of pressure applied. 3D electric resistivity tomography was used to map the degree of saturation, and freezing sampling was also carried out. The results of these tests confirmed that the degrees of saturation measured by these two methods were coincident.

It is worth pointing out that the air sparging method, which is widely used for the control of migration of soil contaminants, is similar to air injection for liquefaction remediation. Tests carried out by Catney and Lynch (2001) revealed that: (1) The radius of desaturated zone increases rapidly with pressure at lower sparging pressure, and then the rate of increase slow down as sparging pressure rises further; (2) Small sparger pores are preferred to assure that the bubbles are small enough to form a suitable desaturated area; and (3) To avoid soil liquefaction caused by sparging pressure, over burden pressures can be applied to the ground surface. These conclusions are also useful for the optimization of air injection for liquefaction mitigation.

Water electrolysis method

In order to improve the liquefaction resistance of sand, Yegian et al. (2007) investigated the feasibility of the water electrolysis method for desaturation of sand. Strain-controlled shaking table tests were performed for both desaturated sands and saturated sands. Two rectangular meshes (20 cm × 30 cm) made of titanium coated mixed metal oxides were used as electrodes. The distance between two meshes was around 29 cm. When electric potential was applied between two electrodes, hydrogen

gas was generated at the cathode and oxygen gas was generated at anode:

The sand sample was prepared by wet pluviation. The relative density was around 20%. The cathode was placed at the bottom of the model box and the anode was placed at the top. The degree of saturation was measured by carefully inspecting the water level rise in the model box. Under an electric current of 525 mA for a duration of 3 to 5.5 hours, the degree of saturation was reduced from full saturation to 96.3%. The amount of gas produced due to electrolysis can be roughly evaluated by the Faraday's law:

$$n(\text{gas}) = n(\text{O}_2) + n(\text{H}_2) = \left(\frac{1}{4} \frac{I}{F} + \frac{1}{2} \frac{I}{F} \right) \Delta t \quad (2.20)$$

in which, n is the moles of gas, I is the electric current, F is Faraday's constant, and Δt is time duration. And mole of gas can be converted to volume as:

$$V = \frac{nRT}{P} \quad (2.21)$$

in which, R is ideal gas law constant, T is temperature, and P is absolute gas pressure. Yegian et al. (2007) also pointed out that this prediction overestimated the gas content produced because the electrolysis efficiency was lower than 100%.

Sodium perborate method

To assure a uniform distribution of gas bubbles in sand, chemical or microbial methods are preferred. Because the dissolved reaction substances can spread in pore water, and then gas bubbles can be generated throughout the sand. One chemical method is to use sodium perborate (NaBO_3) to generate O_2 gas (Eseller-Bayat, 2009). The reaction consists of two steps. The first step is the generation of hydrogen peroxide through the hydrolysis of sodium perborate, as:

The second step is the decomposition of hydrogen peroxide, as:

Eseller-Bayat (2009) reported that there was a close relationship between the degree of saturation and the amount of perborate added, indicating that the degree of saturation could be relatively precisely controlled by adding certain amount of sodium perborate. Pictures taken by a camera with micro lens showed that gas bubbles could be uniformly distributed in sand.

Microbial denitrification method

Several microbial processes are able to produce different types of gases. The commonly seen biogenic gases include N_2 , CH_4 , CO_2 , and H_2 , etc. These processes provide alternatives to achieve soil desaturation. Processes that can produce non-soluble and chemically-inert gases are preferred, because such gases can retain in soil for a relatively long time. N_2 gas produced from denitrification process is inert and its solubility in water is around 0.02 volume of gas per volume of water, which is very low. Therefore, biogas generated through the microbial denitrification are suitable for this purpose (Rebata-Landa and Santamarina, 2012; this study).

Rebata-Landa and Santamarina (2012) designed a device which could observe the gas generation process and measure P-wave velocity during the progress of the reaction. Test variables include soil types and nutrient availability. Results showed that P-wave velocity decreased as gases were produced. Thus P-wave is a good indication of gas content in soils and can be used to detect the degree of saturation of soil. The volume of gas generated is strongly correlated to the specific surface area of soil: The larger the specific area, the larger the gas volume generated. Moreover, the gas producing rate for fine-grain soils is faster than that for coarse-grain soils. The process can be controlled by adding the nutrient step by step, the gas producing rate

can thus be regulated.

Sand compaction pile method

The sand compaction pile technique has been used to enhance of the liquefaction resistance. The mechanism is thought to be the densification of soil and the increase of lateral effective stress. Moreover, it has been found that, accompanying the installation of sand compaction pile, a large amount of air bubbles can be brought into the ground through the casing pipe, which is considered to be a positive effect for the liquefaction resistance (Okamura et al., 2003; Okamura et al., 2006). Okamura et al. (2006) found that the degree of saturation measured from high quality freezing samples was lower than 77% in the sand piles and 91% in the improved sand layers. It was also found that degree of saturation after several years was noticeably, but not significantly larger than that shortly after ground improvement.

To summarize, there are several methods available that can achieve desaturation of sands. Chemical and microbial methods have the advantage that gas bubbles can be relatively uniformly distributed through a large extent. However, the environmental problems should be noticed. The air injection method and the water electrolysis method may not produce a uniform distribution of gas bubbles. But the liquefaction strength of saturated zone can also be improved due to the redistribution of excess pore water pressure during liquefaction.

2.6.3 Determination of gas content

In laboratory tests such as triaxial tests and torsional shear tests, pore pressure coefficient B-value is a convenient way to evaluate degree of saturation. The measurement of gas content in-situ is usually much more complicated. The following six methods have been examined in the previous studies.

Wave velocity method

The wave velocity method aims at obtaining the relationship between the wave velocity (usually P-wave or the ratio of P-wave to S-wave) and the degree or saturation (or pore pressure coefficient B-value determined in the lab). This method has been explored by several researchers (Yang and Sato, 2001; Tamura et al., 2002; Tsukamoto et al., 2002; Yang, 2002; Yang et al., 2004) and it is probably the most promising method for in-situ gas content detection because of the following advantages, as: the test is easy to conduct; comparison can be easily made between lab results and in-situ results; several soil parameters can be obtained simultaneously and one test can serve multiple purposes. The relationship between wave velocity and B-value can be expressed as (Tsukamoto et al., 2002):

$$\left(\frac{V_p}{V_s}\right)^2 = \frac{4}{3} + \frac{2(1-\nu_b)}{3(1-2\nu_b)(1-B)} \quad (2.24)$$

in which, V_p and V_s are pressure wave and shear wave velocities, respectively; ν_b is Poisson's ratio of soil skeleton. Very similar equations were also given by Tamura et al. (2002), Yang (2002), and Yang et al. (2004). Noting that Equations 2.16 and 2.17 give the relationship between B-value (or degree of saturation) and soils strength. And Equation 2.24 gives the relationship between B-value and wave velocity. Therefore, by combining these two relationships together, we can build the correlation between liquefaction resistance and wave velocities of desaturated sands. One of the examples is shown in Figure 2.11 (Tsukamoto et al., 2002). This correlation has experimentally been proved (Tsukamoto et al., 2002; Yang, 2002; Yang et al., 2004).

Figure 2.11. Relationship of cyclic strength and pressure wave velocity for partially saturated sands (after Tsukamoto et al. (2002))

In-situ freezing method

The in-situ freezing sampling method allows high quality undisturbed samples to be retrieved. The execution of this method was described in Okamura et al. (2006). Okamura et al. (2006) also pointed out that the degree of saturation measured in this way was about 2.5% higher than the real value in-situ due the higher solubility of gas in water and the gas volume contraction at lower temperature.

Electric resistivity method

The electric resistivity method for the measurement of degree of saturation was employed by Okamura et al. (2011) in a study of the air injection method. The relationship between electric resistivity and degree of saturation in a geo-material such as sand can be expressed by Archie's law:

$$\rho = a\rho_w n^{-m} S_r^{-p} \quad (2.25)$$

wherein, ρ and ρ_w is the overall resistivity and resistivity of pore fluid, respectively, n is void ratio, and a , m and p are soil constants. This method was proven to be effective for the evaluation of degree of saturation (Okamura et al., 2011). However, it should

be pointed out that the overall electric resistivity is also strongly influenced by the electric resistivity of pore fluid, which implies that chemical or biological desaturation methods may have problems using this technique. Because the ionic concentration is very high.

Back pressure method

A sampler for the measurement of degree of saturation was developed in Norwegian University of Science and Technology (Sandven et al., 2007). The principle of this new technique is Boyle's law, that is, the volume of certain amount of gas is inversely proportional to the pressure applied. The measurement is carried out in such a way that extra pressure is applied to the hermetically sealed soil sample step by step and the volume of deaired water that flows into the soil sample is recorded at each step. The volume of gas can thus be calculated.

BAT method (pore water collecting method)

BAT technique was initially developed for collecting ground water samples in geoenvironmental engineering. This technique was later modified for the measurement of gas content in offshore sediments (Rad and Lunne, 1994; Christian and Cranston, 1997). The basic principle of the technique is that pore water samples are taken by the BAT system, and then the gas types and contents are analyzed by chromatography. It should be noted that BAT method aims at measuring the gas content in dissolved form, implying that this technique is more suitable for offshore deposits (where dissolved gas content could be very high due to the high back pressure under water) than terrestrial soil deposits.

Time domain reflectometry (TDR) method

TDR is a remote sensing electrical measurement technique. It emits an electromagnetic step pulse at one end, and probes at the other end receive the pulse

and send to TDR receiver. From the wave form, the apparent dielectric constant, K_a , can be determined. Topp et al. (1980) found that there were strong correlation between K_a and volumetric water content for several soils. By using in triaxial tests, Grozic et al. (1999) proved that TDR method was effective for the determination of degree of saturation of sand.

To summarize, there are different methods for the detection of gas content in different situations. The wave velocity (seismic wave) method and the electromagnetic wave (TDR) method are the effective methods for detecting gas content in liquefiable sand layers. The in-situ freezing method and the back pressure method can also be used but they are expensive and time-consuming. The electrical resistivity method is also an effective way, but it is not applicable for chemically or microbially desaturated sands because of the presence of ions.

2.7 Microbial methods in geotechnical engineering

2.7.1 Biocements

Conventionally, the studies of geotechnical engineering focus mainly on physical and mechanical process. In recent years, the importance of microbiology in geotechnical engineering has also been recognized. One of these bio-topics is microbially induced carbonate precipitation (MICP) process. This process has been investigated by a number of researchers around the world. In addition, other processes such as denitrification process and iron reducing process are also adopted. A brief review of these processes is given below.

Urea hydrolyzing process

Using urea hydrolyzing process, CaCO_3 is produced and crystallized on the surface on soil particles which can bind soil particle together to increase the shear strength and fill the soil pores to reduce the permeability. Urea hydrolyzing process and other

microbial processes that can strengthen soil or control water flow through soil are termed biocement. Biocement offers some advantages over the conventional cement, such as eco-friendliness and low carbon emission. Two potential applications of biocement are to improve the strength and stiffness of soil (termed biocementation) and to control the seepage through soil (termed bioclogging).

The studies on biocement in recent years concentrated on urea- hydrolyzing method (DeJong et al., 2006; Whiffin et al., 2007; DeJong et al., 2009; van Paassen et al., 2009; van der Ruyt and van der Zon, 2009; DeJong et al., 2010; De Muynck et al., 2010a; De Muynck et al., 2010b; Harkes et al., 2010; Lu et al., 2010; Okwadha and Li, 2010; van Paassen et al., 2010b; DeJong et al., 2011; Stabnikov et al., 2011; Chu et al., 2012b). The process consists of two steps. The first is the hydrolysis of urea by enzyme urease of urease- producing bacteria as follows:

The second is the precipitation and crystallization of calcite:

The studies of urea-hydrolyzing process involve many aspects, such as mechanisms of the process, the optimization of the implementation, and the exploration of potential applications.

Two major problems are the fixation of the bacterial cells on the surface of the soil particle, and the hydrolyzing rate of urea. Fixation of bacterial cells on the soil is important. Because calcite crystallization can contribute to the soil strength only if it grows on surface of sand particles. A two-phase injection method was suggested by Whiffin et al. (2007) and Harkes et al. (2010). This method helps to fix bacterial cells onto the soil particles. Bacterial suspension was injected into the soil first, then fixation fluid containing certain type and concentration of chemicals was flushed through soil. This method facilitates a uniform fixation of cells in soil. Harkes et al.

(2010) recommended that the salinity of fixation fluid should be adjusted in order to adapt different soil conditions and optimize the treatment effect. Okwadha and Li (2010) concluded that the rate of urea hydrolysis was dependent more on biomass concentration than on urea concentration as long as the amount of urea is enough to sustain the bioreaction. Large scale tests have been done by van Paassen et al. (2010b). It was shown that adequate soil strength and stiffness could be obtained throughout a relatively large area provided proper treatment procedures. Geophysical techniques, especially the seismic wave propagation methods, were proved to be effective for monitoring the time-lapse cementation effect in both bench scale and large scale tests (DeJong et al., 2006; DeJong et al., 2010; van Paassen et al., 2010b; DeJong et al., 2011). van Paassen et al. (2010b) demonstrated that soil stiffness observed in shear wave measurement basically agreed with results from direct geotechnical unconfined compression tests.

When biocement is used for seepage control (Stabnikov et al., 2011; Chu et al., 2012b), it is possible to form a layer of calcite on the soil surface to lower the hydraulic permeability of soil (Stabnikov et al., 2011; Chu et al., 2012b). Chu et al. (2012b) and Stabnikov et al. (2011) showed that a 1mm thickness impermeable calcite crust formed on the sand surface can reduce the permeability of sand from an order of 10^{-4} to 10^{-7} m/s.

Other types of biocements

Some other types of microbial processes have been proposed for biocementation or bioclogging.

Iron-reducing bacteria can reduce iron from ferric to ferrous, and then ferrous hydroxide precipitation can fill the soil void and bind soil particles. The mechanism of iron-reducing bacteria as a biocement was explored by Ivanov and Chu (2008),

Chu et al. (2011), and Weaver et al. (2011). Clean sand treated with iron reducing process can reduce the permeability from 10^{-3} to 10^{-7} m/s (Chu et al., 2011). Chu et al. (2011) concluded that microbial iron-reducing process was more suitable for bioclogging than for biocementation.

Most prokaryotic organisms excrete slimy or gummy materials. These exopolymers may clog porous media under some conditions (Baveye et al., 1998). Two types of slime-forming bacteria that have the potential for bioclogging are nitrifying bacteria and oligotrophic microaerophilic bacteria. Test results showed that application of these two types of bacteria decreased permeability of sand from 10^{-4} to 10^{-6} m/s (Ivanov and Chu, 2008).

In this study, biogas for liquefaction mitigation is produced using dissimilative reduction of nitrate, called also denitrification process. Meanwhile, denitrification process also holds the potential for biocementation effect (van Paassen et al., 2010a). The basic mechanism is that carbonate ions are produced from the oxidization of organic matters, and with the presence of calcium ions, the calcite can be formed to serve as the biocement. The advantage of using denitrification process is that there is no useless by-product left if the composition of treatment substrate is properly designed. In contrast, there is useless anions from calcium salt when using urea hydrolyzing process (for example, chloride is useless when calcium chloride is used). However, the accumulation of nitrite may cause environmental issues, which will be seen later in this chapter.

All abovementioned methods are summarized in Table 2.3.

Table 2.3. Potential application of microbial methods in geotechnical engineering

Types of bacteria	Mechanism	Potential geotechnical application	References
Urea hydrolyzing bacteria	Formation of calcite crystallization, carbonate is obtained through urea hydrolyzation	Biocementation and bioclogging of sandy soils, suitable for treatment either in bulk or on surface of soils	All references mentioned in Section 2.7.1
Iron reducing bacteria	Formation of ferrous compound for soil void filling and soil binding, ferrous is obtained from iron reducing reaction	Bioclogging of sandy soils	Chu et al., 2011; Ivanov and Chu, 2008; Weaver et al., 2011
Slime-forming bacteria	Excretion of slime to form water impermeable layer	Bioclogging of sandy and clay soils by forming slime on soil surface	Baveye et al., 1998; Ivanov and Chu, 2008
Denitrifying bacteria	Formation of calcite crystallization, carbonate is obtained through oxidation of organic carbon	Biocementation and bioclogging of sandy soils	van Paassen et al., 2010a
Denitrifying bacteria	Generation of gas bubbles for the alleviation of pore water pressure increase during liquefaction	Mitigation of soil liquefaction for sandy soils	This study

2.7.2 Biological denitrification for gas generation

In this study, microbial denitrification is used for the generation of biogas. Microbial denitrification is a process that nitrate is reduced stepwise to nitrogen gas. The reaction equation using ethanol as the electron donor is as:

Although the application of denitrification for the mitigation of soil liquefaction has not been explored before, denitrifying bacteria and microbial denitrification process were widely studied for other purposes. Some results in these previous studies can provide useful information that serves the research in this study. A review of some factors affecting microbial denitrification is provided below.

pH

pH influences not only the rate of denitrification but also the types of products. Saleh-Lakha et al. (2009) reported that denitrification gene expression in *Pseudomonas mandelii* delayed at pH 5, but was relatively insensitive over the range of pH 6 to 8, and thus low pH negatively affected denitrification activity. Parkin et al. (1985) investigated the influence of pH on denitrification in an agricultural soil with low pH due to fertilization. It was concluded that the denitrification activity was higher in neutral pH, but noticeable activity was also detected in some soil even with pH 4. This suggested that acid-tolerant denitrifying population had been selected in this agricultural field in the past. Simek et al. (2002) pointed out that the relationship between pH and denitrification activity was not simple, as very high denitrification activity was found at both high and low pH conditions because the populations of denitrifiers were adapted to prevailing at natural pH.

The types of final products are also affected by pH. Simek et al. (2002) found that at pH above 7, nitrogen gas (N_2) is the major product rather than nitrous oxide (N_2O). Cuhel et al. (2010) also found that the ratio of $\text{N}_2\text{O}/(\text{N}_2\text{O}+\text{N}_2)$ in biogas decreased

with decreasing pH, but no change in N_2O production was observed.

These studies imply that, although denitrifying communities can adapt a wide range of pH, biogas method for soil liquefaction mitigation favours neutral and alkaline pH environment. In this pH range bacteria produce more N_2 and less N_2O . N_2 gas is wanted in biogas method because it has much lower solubility in water, and the chemical and biological properties are more inert than those of N_2O gas. Moreover, hydroxyl ion is also one of the products in biological denitrification as shown in Equations (2.28), thus pH tends to increase in the process of denitrification, which can be seen as a preferable condition for biogas method.

Nitrate concentration and nitrite accumulation

It was reported by Blaszczyk et al. (1985), Glass and Silverstein (1998), and van Paassen et al. (2010a) that, at relatively high concentration of nitrate, nitrite could accumulate and inhibit further progress of the reaction. Nitrite is more toxic than nitrate and can cause groundwater pollution. Thus, nitrite accumulation during the biogas formation should be avoided. It was found that nitrite accumulation during denitrification was closely related to the initial nitrate concentration. van Paassen et al. (2010a) used calcium nitrate and calcium acetate as electron acceptor and donor, respectively. It was reported that nitrite accumulation was found in the tests with initial nitrate concentration higher than 120 mM. Blaszczyk et al. (1985) found that around 1 g/L of nitrite was accumulated when the initial nitrate concentration was above 3 g/L. Carlson and Ingraham (1983) found that different species of denitrifiers could also influence the nitrite and nitrous oxide accumulation.

In the biogas method, the objective of the treatment is to obtain a degree of saturation as low as around 80%. This is equivalent to an initial nitrate concentration of about 20 mmol/L. Previous studies as reviewed above have shown that there is no nitrite

accumulation in this range of nitrate concentration. Thus, the problem of nitrite accumulation should not be a concern for the intended application.

Medium composition

Blaszczyk (1993) studied the effect of medium compositions on the denitrification process. Four types of media were used, in which three were synthetic mineral media, and one was nutrient broth. The data in Blaszczyk (1993) showed that the reaction rate using complex medium was much faster than that using synthetic mineral media. With initial nitrate concentration of 18 mmol/L, nitrate could be reduced in 12 hours in complex medium. However, it took several days in synthetic mineral media to reduce all the nitrate. For biogas generation for liquefaction mitigation purpose, a slow rate may be preferred because it is easy to control. Therefore, synthetic mineral media are more suitable for biogas generation. Fast denitrification can lead also a non-uniform distribution of gas bubbles.

Other factors

Other factors, e.g. temperature and presence of oxygen gas, can also influence the microbial denitrification process. It was reported that denitrification reaction rate doubles with 10°C temperature increase in the temperature range from 15°C to 35°C (Stanford et al., 1975). Similar results regarding temperature effect on denitrification can also be found in the papers of Fischer and Whalen (2005) and Saleh-Lakha et al. (2009). Saleh-Lakha et al. (2009) pointed out that the presence of oxygen gas causes a higher N_2O/N_2 ratio. However, underground porous water is usually anaerobic environment.

To sum up, the review of previous studies on denitrifying bacteria and denitrification process demonstrates that denitrification process is suitable for liquefaction mitigation through formation of biogas in situ. Factors that can potentially inhibit

denitrification can be avoided under the conditions where biogas method is intended to be used.

2.8 Summary

Critical state soil mechanics provides a theoretical basis to study the fundamental mechanism of undrained behaviour of sand and liquefaction problem. The definition of CSL implies that it is not only void ratio but also mean effective stress that control the undrained behaviour of sands. Some parameters, such as state parameter which considers the influence of both void ratio and mean effective stress, can be used to characterize the undrained response of sands in triaxial compression. For loose sand under triaxial undrained conditions, the post-peak stress-strain behaviour of deformation-controlled tests and load-controlled tests are of much difference, which poses a question that whether residual strength can be used to characterize the post-liquefaction behaviour of sands. Factors that influence the sand behaviour under cyclic loading are similar to those that influence the sand behaviour under quasi-static loading.

Liquefaction assessment methods include laboratory tests, SPT, CPT, and elastic wave, etc. These methods are basically the mapping between specific soil properties obtained in these tests and real liquefaction case histories. Liquefaction countermeasures can be categorized into two types: the direct prevention of liquefaction and the reduction of damage to structures. The direct prevention of liquefaction can be further categorized into two types: methods that work on the soil properties, and methods that work on the boundary conditions. Biogas method, which is the topic of this study, is classified as a method that works on soil properties.

Previous test results show that the cyclic strength of sands doubles when B-value decreases from about 0.95 to about 0.2. At quasi-static condition, the presence of gas

bubbles in sand can also improve the liquefaction strength except for the condition when the gas has high solubility in water and exsolution tends to loosen the sand. Methods that can potentially generate or introduce gas bubbles into sand include: air injection method, water electrolysis method, sodium perborate method, biogas production method, and sand compaction pile method. Sodium perborate method and biogas production method may be more suitable for sand desaturation purpose because these two methods can produce a relatively uniform distribution of gas bubbles. Methods for the measurement of degree of saturation include: wave velocity method, in-situ freezing method, electric resistivity method, back pressure method, BAT method, and TDR method. Wave velocity and TDR methods are effective ways for detecting gas in liquefiable sand layers. Other methods are either expensive or inapplicable to the condition in this study.

Recent studies on the microbial methods in geotechnical engineering concentrates on the enhancement of soil shear strength (biocementation) and control of water seepage through soil (bioclogging). Microbially mediated reagents that can serve these purposes are termed biocement. Urea hydrolyzing process can form calcite crystals to bind soil particles together. This is the most widely studied method. Other types of process that can serve as biocements include iron-reducing process, slime-forming process, and denitrification process.

Microbial denitrification process can produce N_2 gas from nitrate. This process can be used for the desaturation of sand so as to improve the liquefaction resistance. This is the means of soil desaturation that is adopted and studied in this thesis. Previous studies on denitrification are summarized and reviewed. These studies suggests that N_2 gas can be effectively produced given that factors such as availability and type of nutrients, pH, temperature, nitrate concentration, etc., are suitable. Environmental issues, such as nitrite accumulation should be noticed to avoid ground contamination.

Chapter 3

PRELIMINARY STUDY ON THE BIOGAS GENERATION

3.1 Introduction

Methods that can potentially generate or introduce gas bubbles into sand include: air injection method, water electrolysis method, sodium perborate method, and biogas production method, as introduced in Chapter 2. The biological aspects of the biogas method for liquefaction mitigation are discussed in this chapter. The microbial denitrification is studied on the factors affecting gas production and details of the gas generation process. Two experiments were conducted. One was a batch experiment to explore the denitrification process with various electron donors. Another experiment was made to measure the yields and rates of gas generation at various nitrate concentrations. A strain of denitrifying bacteria isolated from anaerobic digester in the municipal wastewater treatment plant was adopted in the experiments. The testing methods and results are presented and discussed in this chapter.

3.2 Materials and methods

3.2.1 Medium composition

For the cultivation of bacteria, the medium was composed of the following components: $\text{C}_2\text{H}_5\text{OH}$, 0.5 g; KNO_3 , 1.01 g; NH_4Cl 0.12 g; KH_2PO_4 , 0.75 g; K_2HPO_4 , 2.5 g; $\text{MgSO}_4 \cdot 7\text{H}_2\text{O}$, 0.1g; $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$, 0.01 g; $\text{CaCl}_2 \cdot 2\text{H}_2\text{O}$, 0.015 g; trace element solution 1ml; and addition of de-ionized water to 1 L. 1ml of trace element solution

contains: EDTA-2Na, 10 mg; $\text{MnCl}_2 \cdot 4\text{H}_2\text{O}$, 0.12 mg; $\text{ZnSO}_4 \cdot 7\text{H}_2\text{O}$, 0.12 mg; $\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$, 0.03 mg; $(\text{NH}_4)_6\text{Mo}_7\text{O}_{24} \cdot 4\text{H}_2\text{O}$, 0.05 mg; $\text{NiCl}_2 \cdot 6\text{H}_2\text{O}$, 0.1 mg; $\text{CoCl}_3 \cdot 6\text{H}_2\text{O}$, 0.1 mg; $\text{AlCl}_3 \cdot 6\text{H}_2\text{O}$, 0.05 mg; H_3BO_3 , 0.05 mg.

For batch experiments, the medium composition was almost the same as stated above, except the types and concentrations of electron donors. There were three batches in batch experiments. Ethanol (0.75 g/L), sodium acetate (1.59 g/L), or methanol (1.04 g/L) was used as electron donors, respectively. The amounts of electron donors used in the tests were calculated by considering 100% bioreduction of nitrate and inclusion of about 57% of added KNO_3 in the bacterial biomass.

There were six samples in the gas generation tests. The KNO_3 -N concentrations were varied from 125 to 374 mg/L. Ethanol was used as the source of carbon and electron donor at concentration 0.75 g/L for all tests. The concentrations of other electron donors were one-tenth of those used for the batch experiment. For large volume soil treatment, in order to simplify the procedure, tap water can be used instead of using de-ionized water, and trace elements need not add.

3.2.2 Isolation and cultivation of denitrifying bacteria

The source for isolation of denitrifying bacteria was anaerobic digester in the municipal wastewater treatment plant. Enrichment culture was obtained by anaerobic cultivation on the liquid medium for three times to ensure that denitrifying bacteria were dominant in the enrichment culture. Pure culture was isolated from the colony on solid medium using inoculation with the suspension of enrichment culture. First, the enrichment culture was serially diluted ten-fold with sterile 0.85% NaCl solution. Then the diluted suspensions were spread and cultivated on agar plates. Pure culture was obtained by taking cells from a single colony on the agar plate as inoculum for cultivation in liquid medium. The liquid medium used for agar plate and pure culture

was the same as that used for the enrichment culture. Solidification of the medium was made by addition of 12 g agar to 1 L of the liquid medium. For both enrichment and pure cultures, growth of bacteria was monitored by measuring optical density (OD₆₀₀) daily. The cultures were harvested at early stationary phase and stored in refrigerator at 4 °C.

3.2.3 Identification of bacteria

The nearly full-length 16S rRNA gene was amplified by Polymerase Chain Reaction (PCR) with forward primer 27F and reverse primer Universal 1492R (Lane 1991). Purified PCR products were sequenced using the ABI PRISM 3730xl DNA sequencer and the ABI PRISM BigDye Terminator Cycle Sequencing ready-reaction kit. Primers 27F, 530F, 926F, 519R, 907R and 1492R were adopted to sequence both strands of the 16S rRNA gene. The sequences were finally assembled to produce the full-length sequence, which was compared with all other sequences available in the NCBI Genbank database (<http://www.ncbi.nlm.nih.gov/>). The probable identity of the bacterial strain was thus determined. The results showed that the dominant species in the enrichment culture and the pure culture isolated from the enrichment culture most likely belonged to *Acidovorax* sp. or *Diaphorobacter* sp. The results of 16S rRNA gene sequence identity search against known species in NCBI Genebank database are as follows:

- 1) *Acidovorax ebreus* TPSY, complete genome: Identity = 99%
- 2) *Acidovorax* sp. JS42, complete genome: Identity = 99%
- 3) *Diaphorobacter* sp. GS-1 16S ribosomal RNA gene, partial sequence: Identity = 99%
- 4) *Diaphorobacter nitroreducens* strain NA10B 16S ribosomal RNA, partial sequence: Identity = 99%
- 5) *Diaphorobacter* sp. NA5 16S ribosomal RNA gene, partial sequence: Identity = 99%

The microscopic image of isolated strain is shown in Figure 3.1. This image was taken by Olympus BX-60 Epifluorescence Microscope. The cells have single rod shape. The length of cells is around 2 μm . The morphology of the colonies of isolated strain is shown in Figure 3.2. The colonies have circular shape with raised elevation and entire margin and the colour of the colonies is beige.

Figure 3.1. Microscopic image of cells of isolated strain

Figure 3.2. Colony morphology of isolated strain on spread plate

3.2.4 Batch experiment

Batch experiment was performed to investigate physiological characteristics of the isolated strain of denitrifying bacteria. The theoretical equations for reaction of nitrate bioreduction by ethanol, sodium acetate, or methanol are as follows:

Experiment was carried out in the sealed bottles. 10 mL of bacterial suspension of pure culture with OD₆₀₀ of 0.51 was added to 990 mL of autoclaved liquid medium. The test cultures were purged with N₂ gas initially and every time after taking the sample in order to minimize the negative influence of O₂ on denitrifying bacteria. The test was carried out at room temperature without shaking. The bottles were enveloped with aluminum foil to avoid the effect of light. Parameters that were measured included nitrate and nitrite concentrations, OD of bacterial suspension, pH, and dissolved total organic carbon (TOC). Nitrate, nitrite and OD₆₀₀ were measured using Shimadzu UVmini-1240 UV-Vis spectrophotometer. Nitrate and nitrite measurements were done using APHA 4500-NO₃I method and APHA 4500-NO₂B method, respectively (APHA, 2005). The pH was measured using Eutech CyberScan PCD 6500 pH/Ion/Conductivity/DO Meter. The measurement of TOC was using Shimadzu TOC-VCSH TOC Analyzer. Liquid samples were taken from the bottles twice a day, filtered using 0.2 µm syringe filters, and stored at 4°C in refrigerator before the measurement. The batch experiment was stopped when nitrate or organic source were consumed.

3.2.5 Gas generation test

Gas generation test was done to investigate the gas yield and gas generation rate of denitrifying bacteria. The set-up of the test was schematically illustrated in Figure 3.3.

Samples with sand and without sand were prepared in 60 mL syringes. The syringe was connected to a burette. During the reaction, the generated gas would shift the level of liquid in the burette, which was detected by a camera controlled by a computer. Software installed in the computer could take pictures at a certain time interval. The gas generation rate can thus be measured.

The test conditions for each sample are listed in Table 3.2. Soil treatment liquid contained 1% volume of bacterial suspension of pure culture and 99% volume of liquid medium. The bacterial suspension was the same as that used in batch experiment. For sand samples, treatment liquid was firstly poured into the syringe, and then sand was slowly deposited into the liquid. In this way, a pre-saturated sample could be formed. The amount of sand, the total volume of the sample, and the amount of gas generated were known, so that the volume of liquid, the void ratio of sand, and the degree of saturation after treatment could be calculated using phase relationships of soils. When the generation of gas stopped, the set-up was disassembled, and the liquid sample was taken for the measurement of pH, residual nitrate and nitrite concentrations.

Figure 3.3 Set-up of gas generation test

3.3 Results

3.3.1 Batch experiments

A summary of the test results is given in Table 3.1.

Table 3.1. Results of batch experiments

Substrate		1.01 g KNO ₃ + 0.75 g ethanol	1.01 g KNO ₃ + 1.59 g sodium acetate	1.01 g KNO ₃ + 1.04 g methanol
nitrate reduction rate	mmol/day	9.5	8.5	1.3
TOC consumption rate	mg/day	210	260	64
Growth rate	OD ₆₀₀ /day	0.338	0.275	0.128
pH at the beginning of the reaction		7.24	7.20	7.24
pH at the end of the reaction		7.02	7.53	6.78
Max. nitrite accumulation	mmol	0.058	0.332	2.083

Nitrate reduction

The data of nitrate reduction tests are shown in Figure 3.4. For the tests using ethanol and sodium acetate as electron donors, the nitrate reduction rates were relatively fast as shown in Figure 3.4 (a) and (b): the maximum nitrate reduction rates were 9.5 and 8.5 mmol/L/day, respectively. The reduction of nitrate completed within 3 days. For the test using methanol as electron donor, the rate was considerably slower (1.3 mmol/L/day) as shown in Figure 3.4 (c). Figure 3.4(c) also showed that the reaction with methanol ceased at the eighth day and only part of nitrate was reduced because of the shortage of the electron donor.

Nitrite accumulation

Figure 3.5 shows the nitrite concentrations for three tests using different electron donors: ethanol, sodium acetate, and methanol. For the tests using ethanol and sodium acetate, the maximum nitrite accumulation was 0.06 and 0.33 mmol/L, respectively. These concentrations are very small portions of the initial nitrate concentration, which was 10 mmol/L. The nitrite accumulations with these two electron donors were temporary. The test using methanol as electron donor showed

continuous accumulation of nitrite until electron donor was oxidized. The concentration of nitrite was high at the end of this test, 2.0 mmol/L.

Growth of biomass

Growth of biomass can be shown by the increase of optical density (OD) of bacterial suspension. Figure 3.6 shows the OD for the three tests with ethanol, sodium acetate, and methanol, respectively. Tests with ethanol and sodium acetate showed similar maximum specific growth rates of 0.34 and 0.27 day⁻¹ for the media with ethanol or sodium acetate, respectively. These rates were faster than the specific growth rate in the test with methanol, 0.13 day⁻¹. The yields of biomass in the three tests were similar.

pH

The pH variations with time are shown in Figure 3.7 for the tests with ethanol, sodium acetate, and methanol, respectively. Tests with ethanol and methanol showed a small decrease in pH, from 7.24 to 7.02 for ethanol, and from 7.24 to 6.78 for methanol. The test with sodium acetate showed small increase in pH, from 7.20 to 7.53. However, in all three tests, pH changes were small.

Consumption of total organic carbon

The consumption and oxidation of total organic carbon (TOC) for the three tests using ethanol, sodium acetate, and methanol, respectively, is shown in Figure 3.8. The rate the TOC change basically agreed with the nitrate reduction rate. The rates of TOC change in the tests with ethanol or acetate were 210 and 260 mg/L/day, respectively, which were similar. These rates were much faster than the rate for the test with methanol, 64 mg/L/day. Figure 3.8(c) shows that there was limitation of electron donor for the test with methanol. The Organic source limitation implies that the yield of N₂ gas on methanol is smaller than that on ethanol and sodium acetate.

(a)

(b)

(c)

Figure 3.4. Dynamics of nitrate concentrations in the tests with different electron donors: (a) ethanol; (b) sodium acetate; (c) methanol

(a)

(b)

(c)

Figure 3.5. Dynamics of nitrite concentrations in the tests with different electron donors: (a) ethanol; (b) sodium acetate; (c) methanol

(a)

(b)

(c)

Figure 3.6. Dynamics of OD_{600} of bacterial suspension in the tests with different electron donors: (a) ethanol; (b) sodium acetate; (c) methanol

(a)

(b)

(c)

Figure 3.7. Dynamics of pH in the tests with different electron donors: (a) ethanol; (b) sodium acetate; (c) methanol

(a)

(b)

(c)

Figure 3.8. Dynamics of total organic carbon (TOC) in the tests with different electron donors: (a) ethanol; (b) sodium acetate; (c) methanol

To summarize, the biological denitrification reaction rate and specific growth rate of bacteria in the tests with ethanol and sodium acetate as electron donors were faster than those rates in the test with methanol. Furthermore, tests with ethanol and sodium acetate showed low concentrations of temporary nitrite accumulation, whereas high nitrite accumulation was observed in the test with methanol. As reviewed in Chapter 2, nitrite is toxic and is not desirable in the process. These findings imply that ethanol and sodium acetate are more suitable for biogas production than methanol.

3.3.2 Gas generation tests

Gas generation tests were used to directly measure the volume of gas generated through denitrification process. The set-up of the test is shown in Figure 3.3. There were six samples of the test. Three samples were the tests in sand column, and the other three samples had no sand added. Tests were done with different initial nitrate concentrations. The curves of gas generation versus time are shown in Figure 3.9. The test results are summarized in table 3.2.

As shown in Figure 3.9, the rates of gas generation for all six samples were very similar, irrespective of nitrate concentration and the presence of sand. However, the beginning of the gas production in the tests without sand was delayed compared to the sand column test. A reasonable explanation is that the syringe used was transparent and light inhibits the growth of bacteria in the absence of sand. These gas generation curves also agree with the results in batch experiments. It seemed that there was no gas produced in the initial one or two days, and then gas was gradually produced until the end the reaction. The entire reaction took about 3 days for sand column tests and about half day longer for the tests without sand.

Table 3.2 shows that not all nitrate was converted to gas phase. The percentage conversion ranged from 70.6% to 86.3%. The average value was around 80%. Thus,

the value of 80% conversion rate is taken in the subsequent analysis in this thesis.

As seen in Table 3.2, the amount of residual nitrate and nitrite at the end of the test was related to the initial nitrate concentrations. Tests 1 and 4 had an initial NO_3^- -N concentration of 374.3 mg/L. The percentage of nitrate or nitrite left was 5~10%. When the initial nitrate concentration decreases to the range of 249.5 ~ 124.8 mg/L as in Tests 2, 3, 5 and 6, the residual nitrate and nitrate concentrations became much less.

The gas generation test showed a considerable pH increase at the end of the test, compared with the batch test in which pH change was small. This is because of the concentrations of K_2HPO_4 and KH_2PO_4 in the medium. K_2HPO_4 and KH_2PO_4 are commonly used as pH buffer. The concentrations of these two chemicals used in the gas generation test were one tenth of those used in the batch test as mentioned in Section 3.2.1. Although pH in the test increased to around 9, the progress of the reaction did not seem to be affected by the increase in pH.

Figure 3.9. Gas generation curve in the tests with sand (a) and without sand (b)

Table 3.2. Results of gas generation tests

	Test type	Void ratio	Initial nitrate (-N) concentration	Residual nitrate (-N)		residual nitrite(-N)		Volume of gas		% of nitrate converted to gas phase	pH at the end of the test
			mg/L	Cocncentration	% of Initial nitrate	Cocncentration	% of Initial nitrate	Volume	% of pore space (1-Sr)		
1	Sand column	0.678	374.3	35.5	9.5%	25.4	6.8%	5.40	23.5%	78.3%	8.47
2	Sand column	0.678	249.5	1.0	0.4%	0.2	0.1%	3.97	17.3%	86.3%	8.20
3	Sand column	0.678	124.8	0.5	0.4%	Trace		1.87	8.1%	81.3%	7.70
4	No sand	-	374.3	21.6	5.8%	33.6	9.0%	4.87	21.2%	70.6%	9.25
5	No sand	-	249.5	0.4	0.2%	0.1	0.0%	3.61	15.7%	78.5%	9.47
6	No sand	-	124.8	Trace		Trace		1.89	8.2%	82.2%	7.98

3.4 Discussion

The efficiency of conversion from nitrate to gas phase is important. Table 3.2 has shown that the volume of N_2 gas generated was equivalent to around 80% of the initial nitrate for all six samples. 20% of nitrate did not turn into gas phase possibly because: (a) a small amount of gas was dissolved in water (solubility of N_2 gas in water is about 0.02 volume of gas to 1 volume of water at standard condition); (b) there was a small amount of accumulations of intermediates (NO_2^- , NO, and N_2O). CO_2 would be dissolved in water because the amount produced was much less than its solubility.

Table 3.2 already shows that with a relatively high concentration of nitrate, there was a noticeable percentage of nitrate and nitrite left at the end of the test. Results shown in this chapter demonstrate that, in order to minimize the production of nitrite, proper electron donors should be selected, and the initial nitrate concentration should not be high. Moreover, the treatment liquid with KNO_3 -N concentration of 250 mg/L, which does not yield any nitrite accumulation according to the result in table 3.2, can produce a degree of saturation of 82.7%. This degree of saturation is low enough to significantly mitigate the liquefaction potential of soils according to the test results and analysis that will be shown in Chapters 4 and 5. Other undesired products could be NO and N_2O . These two gases contribute to greenhouse effect, ozone depletion, and photochemical air pollution (Kester et al., 1997). Although the amount of production of NO and N_2O is small enough to be ignored, measures should be taken to avoid these intermediates. It is reviewed in Chapter 2 that factors and conditions that are in favour of the production of N_2O include: low pH, presence of oxygen gas, low temperature, and high nitrite concentration (Carlson and Ingraham, 1983; Simek et al., 2002; Saleh-Lakha et al., 2009; Cuhel et al., 2010). A major reason for the accumulation of NO gas is the high nitrite concentration (Betlach and Tiedje, 1981; Kester et al., 1997). Soil grounds that are susceptible to liquefaction are usually

saturated and below the ground water table. Thus, oxygen gas is naturally absent. Other factors, such as pH, temperature, nitrite concentration, and species of denitrifying bacteria, are not difficult to satisfy. Furthermore, during the application of in situ denitrification method, the progress of the reaction should be monitored to make sure that the generations of these hazardous intermediates meet the safety requirement.

Compared with conventional liquefaction mitigation methods, the biogas method is more environmentally friendly. Denitrification is a naturally occurred process and it causes less disturbance to the environment. However, some environmental issues should be addressed. First, some strains of denitrifiers are pathogens and these strains should be avoided. A genetic identification of the bacteria used can tell whether the bacteria are pathogenic or not. Second, after the gas-generation reaction, the chemical substances left should meet the groundwater quality standards to avoid the contamination of groundwater, especially some toxic and hazardous intermediates, such as NO_3^- , NO, and N_2O . This requires a proper design of the composition of soil desaturation medium and monitoring of groundwater quality during the implementation.

Low cost is another advantage of the biogas method. To treat 1 m^3 of liquefiable soil requires around 1 kg of potassium nitrate and 0.5 kg of ethanol. The use of other chemicals listed in Section 3.2.1 also requires small quantities. The cost of materials is far below the cost of other methods, such as cement mixing, and microbially-induced carbonate precipitation (MICP) method. The cost of the operation of the biogas method should also be low. Because biocement liquid has a low viscosity and can be easily spread throughout the treated area just by pouring on or injecting into the ground, thus no special equipment is needed.

3.5 Conclusions

- The batch experiment was conducted to investigate the favored conditions for the growth of denitrifying bacteria and the preferred conditions for the progress of the denitrification reaction. Three batches, with ethanol, sodium acetate, and methanol, as electron donors, respectively, were used. The reaction rates and growth rates with ethanol and sodium acetate were similar (nitrate reduction around 9 mmol/day and growth rate around 0.3 OD₆₀₀/day), both of which were considerably faster than the batch with methanol (nitrate reduction around 1.3 mmol/day and growth rate around 0.13 OD₆₀₀/day). Moreover, the batch with methanol showed large amount of permanent nitrite accumulation; while nitrite accumulations with ethanol and sodium acetate were minor and temporary. These results suggest that the composition of soil desaturation solution should be properly selected to avoid the accumulation of nitrite.
- The gas generation process was studied using a small size sand column. The nitrate-N concentrations in the samples were varied from 125~374 mg/L. The reaction was completed in around 3 days and the reaction rates were similar for all the samples. At a nitrate concentration of 374 mg/L KNO₃-N, there were noticeable, but small percentage of residual nitrate and nitrite, while in a lower range of KNO₃-N concentration (125~250 mg/L), residual nitrate and nitrite were minor. The lowest degree of saturation that did not cause nitrite accumulation was 82.7%. This degree of saturation was low enough to considerably mitigate the liquefaction potential according to the conclusions in Chapters 4 and 5. These results prove that microbial denitrification process is effective to generate nitrogen gas in sand.

Chapter 4

BEHAVIOUR OF DESATURATED SAND IN THE TRIAXIAL TESTS

4.1 Introduction

Liquefaction can occur under either cyclic or static loading conditions. This chapter presents a study on the behaviour of desaturated sand under static loading condition using triaxial undrained compression and extension tests. As reviewed in Chapter 2, factors that influence the cyclic and static liquefaction are similar. The study of liquefaction under static condition avoids the complication of dynamic loading, and the element analysis using conventional triaxial tests can provide more insights into the mechanism of soil liquefaction. Thus, the study of the fundamental mechanisms of liquefaction can start from static liquefaction.

Previous studies on undrained behaviour of desaturated sand under monotonic condition, such as Rad and Berre (1994) and Grozic et al. (1999), basically focused on the slope stability due to the presence of gas in pore space (details can be seen in Section 2.6.1). However, the aim of this study is to develop an artificial desaturation technique for liquefaction mitigation. Undrained response of sand was studied under various degrees of saturation and void ratios so as to explore the mechanism of the biogas method and the key control factors for liquefaction mitigation.

4.2 Materials and testing set-up

4.2.1 Materials

The same ASTM graded sand as used in gas generation test in Chapter 3, was used in the tests. ASTM graded sand is clean, round-shaped with minimum and maximum void ratio of 0.5 and 0.8, respectively. Figure 4.1 shows a image of sand particles of ASTM graded sand. The mean size (D_{50}) is 0.4 mm. Typical sand properties are summarized in Table 4.1. Figure 4.2 shows the particle size distribution curve of ASTM graded sand used in the tests.

Table 4.1. Typical engineering properties of ASTM graded sand

Description	Grain shape	Mean size (mm)	Typical specific gravity	Hardness	Maximum void ratio	Minimum void ratio
ASTM graded sand	Round	0.4	2.65	7	0.8	0.5

Figure 4.1. Image of sand particles of ASTM graded sand

Figure 4.2. Particle size distribution

4.2.2 Sample preparation methods

The samples for the compression tests were 100 mm in height and 50 mm in diameter and for the extension tests were 140 mm in height and 70 mm in diameter. Two sample preparation methods were used in the tests. One was the moist tamping (MT) method and the other one was the water sedimentation (WS) method. In this study, the relative densities of the samples prepared by the MT and the WS methods were around 10%~30% and 40%, respectively. The exact value of the void ratio of each sample is shown in Table 4.2. For biogas desaturated samples, medium and bacterial inoculums were pre-mixed with ultra-pure water, and then this solution was used for the preparation of samples. The composition of medium has been described in Chapter 3.

The two sample preparation methods are described as follows.

Moist tamping (MT) method

The procedure of the MT method is as follows:

- Mix a fixed amount of dry sand with 5% of distilled water;

- Assemble the rubber membrane and 3-split sample mould onto the pedestal of the triaxial apparatus;
- Separate the sand into five equal portions, place each portion into the mould, slightly tamp the sand with a flat-bottomed tamper for 5 times, and measure the height of each layer to make sure that all layers had the same height;
- Assemble the top cap, remove the sample mould, assemble the triaxial chamber, and fill the chamber with distilled water;
- Apply a small cell pressure (15 kPa);
- Flush the sample with CO₂ gas for about 20 minutes;
- Flush the sample water (de-aired water was used for saturated samples, and reagent liquid was used for bacterial desaturated samples).

For the biogas samples, the gas generation process can be observed by the water level change in a short tube connected to the sample through the drainage valve. The gas generation completed when the water level in the tube stopped rising. It took less than 5 days to complete the reaction.

Conventionally, saturation of triaxial samples is achieved by applying a high back pressure to ensure that all the gas bubbles are dissolved in pore water. However, this method was not applicable in this study. Because gases are needed to achieve a certain degree of saturation. If the back pressure was too high, gas bubbles would be compressed or dissolved in water. The same back pressure of 100 kPa was applied for all desaturated samples. Back pressure used for each sample can be seen in Table 4.2. Carbon dioxide gas was used for sample saturation. CO₂ gas has a high solubility in water. When flushing water into the sample, CO₂ gas can be easily dissolved in water even at a low back pressure. By using this method, it can be guaranteed that gas bubbles in the sample were produced from the denitrification reaction, and the volume of gas could be controlled.

Water sedimentation (WS) method

The procedure of the WS method is as follows:

- Assemble the rubber membrane onto the pedestal of the triaxial apparatus;
- Pore 2-cm height of water into the mould;
- Slowly deposit dry sand into water through a funnel until the sand level almost reached the water level;
- Repeat the second and third step until the sand level reached the designed height;
- Assemble the top cap and apply a small suction to the sample by lowering the burette connected to the drainage valve;
- Remove the mould, assemble the triaxial chamber, and fill the chamber with distilled water.

Similarly to the MT method, a back pressure of 100 kPa was used for all desaturated WS samples.

4.2.3 Tests with gas-tubes

To assess the desaturation effect using biogas, artificial inclusion of a known volume of gas in a saturated sample was also used. A plastic tube with a given length was sealed at one end and linked to a drainage valve at the other end (as shown in Figure 4.3). Thus, the sample together with a known amount of gas in the tube forms a close system during the test. This method was firstly adopted by Lade and Pradel (1990) to study the effect of desaturation on the instability of sand. This method is referred to as the gas-tube test in the following discussion. The main purpose of this type of tests is to explore the relationship between pore water pressure B -value and degree of saturation.

Figure 4.3. Picture of the gas tube method

4.2.4 Triaxial testing system

A conventional deformation-controlled triaxial apparatus (ELE Digital Tritest) was used. The testing system is shown in Figure 4.4. Two GDS controllers were used to control the cell pressure and back pressure, respectively. The readings of axial load, axial displacement, and pore water pressure were recorded every two seconds by the computer. Pictures of the triaxial testing apparatus for compression and extension tests are shown in Figure 4.5 (a) and (b), respectively.

For extension tests, a specially-designed mechanism as introduced by Bishop and Henkel (1962) was used to interlock the top cap and loading pistol during shear. The mechanism is shown in Figure 4.6. There is a cross-shape latch on the head of the loading pistol and a circular metal plate with cross-shape slot on the top cap. Before the test, the loading pistol moves down through the slot on the top cap and rotates 45 degrees to interlock the top cap and loading pistol.

Figure 4.4. Triaxial testing system

1: Sample; 2: Gas tube (used for gas tube tests); 3: GDS-controller (cell pressure control); 4: GDS-controller (back pressure control); 5: load transducer; 6: LVDT (axial displacement measure); 7: Pore water pressure transducer; 8: Data logger; 9: Computer

(a)

(b)

Figure 4.5. Picture of triaxial test apparatus
(a) compression test; (b) extension test

Figure 4.6. Design of loading pistol and top cap of triaxial extension apparatus

4.2.5 Test procedure and test arrangement

The operation of the experiment followed the standard procedure of a triaxial isotropically-consolidated undrained test (Head and Epps, 2011). The sample was sheared at a strain rate of 0.1% per minute. The axial strain was applied to around 20%, or until deviatoric stress reached zero, or until the sample was seriously distorted.

The testing parameters for all samples are shown in Table 4.2. There were six series of tests. Each series had several tests with various degrees of saturation. Series 1 (Tests MT01 to MT 06) were compression tests. The samples were biogas desaturated and prepared by the MT method. Series 2 (Tests MT01 and MT07 to MT10) were compression tests. The samples were prepared by the MT method and were air-tube desaturated. Series 3 (Tests MT11 to MT14) and Series 4 (Tests 15 to 17) were extension tests. The samples were prepared using the MT method and were bacterial desaturated. Samples in Series 4 were denser than samples in Series 3. Series 5 (Tests WS01 to WS04) were compression tests. The samples were prepared by the WS method and were biogas desaturated. Series 6 (Tests WS05 to WS07) was extensions tests. The samples were prepared by the WS method and were biogas desaturated.

Table 4.2 shows that most of the samples in the triaxial tests were loose sands with relative density from 10% to 30%. Loose sand is the major interest in this chapter, because static liquefaction (strain softening) occurs only in loose sand. Dense sand shows strain hardening and is free from liquefaction in quasi-static loading conditions. In comparison, cyclic liquefaction can potentially occur in dense sand given abundant number of cycles (Yang et al., 2004; Okamura and Soga, 2006).

Table 4.2. Testing programme

Sample preparation method	Sample desaturation method	Loading mode	Void ratio after consolidation	Initial degree of saturation	Cell pressure		Back pressure	Pore pressure coefficient B	Undrained shear strength
					kPa	kPa			
MT01	Moist tamping	Compression	0.769	100.0%	200	100	0.967	24.0	
MT02	Moist tamping	Compression	0.771	99.8%	200	100	0.791	27.9	
MT03	Moist tamping	Compression	0.770	99.2%	200	100	0.685	30.1	
MT04	Moist tamping	Compression	0.773	98.1%	200	100	0.548	39.4	
MT05	Moist tamping	Compression	0.773	96.3%	200	100	0.431	45.2	
MT06	Moist tamping	Compression	0.774	94.5%	200	100	0.351	51.1	
MT07	Moist tamping	Compression	0.772	98.8%	200	100	0.699	28.1	
MT08	Moist tamping	Compression	0.771	97.8%	200	100	0.589	30.3	
MT09	Moist tamping	Compression	0.770	97.0%	200	100	0.485	33.3	
MT10	Moist tamping	Compression	0.772	94.1%	200	100	0.393	36.7	
MT11	Moist tamping	Extension	0.769	100.0%	200	100	0.950	10.4	
MT12	Moist tamping	Extension	0.773	96.4%	200	100	0.592	12.1	
MT13	Moist tamping	Extension	0.774	94.6%	200	100	0.516	13.8	
MT14	Moist tamping	Extension	0.772	87.6%	200	100	0.287	17.4	
MT15	Moist tamping	Extension	0.707	100.0%	200	100	0.986	12.5	
MT16	Moist tamping	Extension	0.706	94.6%	200	100	0.546	17.7	
MT17	Moist tamping	Extension	0.711	87.6%	200	100	0.258	21.6	
WS01	Water sedimentation	Compression	0.692	100.0%	300	200	0.958	332.1	
WS02	Water sedimentation	Compression	0.681	98.1%	200	100	0.661	220.3	
WS03	Water sedimentation	Compression	0.675	94.6%	200	100	0.550	206.6	
WS04	Water sedimentation	Compression	0.657	87.6%	200	100	0.271	162.9	
WS05	Water sedimentation	Extension	0.690	100.0%	200	100	0.947	71.9	
WS06	Water sedimentation	Extension	0.676	94.6%	200	100	0.330	44.2	
WS07	Water sedimentation	Extension	0.684	87.6%	200	100	0.198	40.5	

4.3 Test results

4.3.1 Relationship between pore pressure coefficient B-value and degree of saturation S_r

Skempton (1954) used two parameters A and B to describe the pore pressure response in triaxial tests. Due to the variation of principal stresses, the change of pore pressure can be simply expressed as:

$$\Delta u = B(\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)) \quad (4.1)$$

In the consolidation stage of a triaxial test, there is no deviatoric stress. Then coefficient B becomes:

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad (4.2)$$

Usually, the pore pressure coefficient B is used to evaluate the saturation in triaxial tests. The theoretical relationship of B-value and degree of saturation has been obtained by several workers (Lade and Hernandez, 1977; Tamura et al., 2002; Tsukamoto et al., 2002; Yang, 2002). This relationship can be expressed as (Lade and Hernandez, 1977):

$$B = \frac{1}{1 + nS_r \frac{C_w}{m_v} + \frac{n}{m_v} \frac{1 - S_r}{\bar{u}}} \quad (4.3)$$

in which, n : the porosity of the soil; S_r : degree of saturation; C_w : compressibility of water (approx $0.49 \times 10^{-6} \text{ m}^2/\text{kN}$); m_v : volumetric compressibility coefficient; $\bar{u}$: the absolute pressure after the application of stress increment. By assigning the values of these parameters used in the test ($n=0.44$ for the samples with $e \approx 0.77$ and 0.405 for the samples with $e \approx 0.68$, $m_v = 5 \times 10^{-5} \text{ m}^2/\text{kN}$ for the samples with $e \approx 0.77$ and $7.5 \times 10^{-5} \text{ m}^2/\text{kN}$ for the samples with $e \approx 0.68$, $u_2 = (200 + 100 \times B) \text{ kPa}$) into the equation, theoretical curves can thus be obtained, which are shown as solid lines in Figure 4.7 (a) and (b). Note that Equation 4.3 is not an explicit expression, that is, the parameter $\bar{u}$ on the right hand side is also a function of B-value. In order to solve Equation 4.3,

try and error is needed.

The gas content of biogas samples can be evaluated by converting the initial nitrate concentration to N_2 gas as introduced in Chapter 3. According to Equation 3.1, 2 moles of potassium nitrate produce 1 mole of N_2 gas. However, gas content or degree of saturation cannot be directly measured in the triaxial tests. Because the triaxial samples were held by a flexible membrane, thus the volume replacement method for the determination of gas content is not applicable here. Hence, an indirect method to evaluate the degree of saturation of the biogas samples was used. The method is to use gas tube tests. For gas tube samples as introduced in Section 4.2.3, a known volume of gas was sealed in the gas tube which was linked to the sample. Thus, the measured S_r -B-value relationship of gas tube samples can be used to validate the theoretical relationship (that is, Equation 4.3).

The S_r -B-value relationships are shown in Figure 4.7 (a) and (b) for loose and medium dense samples, respectively. It can be seen that data for both gas tube tests and biogas tests fit the theoretical curve well. Most of the data points are very close to the curve, except for a few points with a little scatter. This proves that the use of pore water pressure coefficient B-value to quantify degree of saturation is valid. This also proves that fixed amount of gas bubbles produced by denitrifying bacteria can be achieved in triaxial samples.

(a)

(b)

Figure 4.7. Relationship between pore pressure coefficient B -value and degree of saturation for (a) loose sand, and (b) medium dense sand

4.3.2 Undrained behaviour of loose sand

4.3.2.1 Compression tests

The undrained responses of biogas desaturated loose sand in the triaxial compression tests are shown in Figure 4.8. From Test MT01 to Test MT06, all the test conditions, such as void ratio, effective confining pressure, sample preparation method, were similar. The only distinct difference among these six tests was degree of saturation. As shown in Figure 4.8(a), with the decrease in B-value, tests manifested an increasing trend in undrained shear strength. The test of fully saturated sand MT01 showed a complete liquefaction behaviour, the deviatoric stress almost reached zero. MT02 to MT05 manifested a strain softening behaviour after the peak strength until critical state was reached at large strain. MT06 experienced a minor quasi-steady state, and then showed a gentle strain hardening manner.

Figure 4.8(b) shows the plot of pore pressure against axial strain. The complete liquefaction of fully saturated test MT01 was also indicated by the observation that the pore pressure increased to the confining pressure (that is, 100 kPa), implying that the effective stress and soil strength decreased to zero. With the decrease in B-value, the pore water pressure generation was gradually reduced. MT06 with B-value of 0.351 had a maximum pore pressure only about half of the effective confining pressure.

The comparison of effective stress paths in Figure 4.8(c) also shows that a little decrease of degree of saturation leads to a much stronger liquefaction resistance for loose sands.

To summarize, with the decrease of degree of saturation, the undrained response of loose sands shows an ascending trend in shear strength and a descending trend in pore pressure generation. For the sand tested, the stress-strain behaviour changed from

complete liquefaction for saturated sample to a non-liquefaction for the sample with $S_r=94.5\%$.

(a)

(b)

(c)

	e_c	S_r	B
MT01	0.769	100.0%	0.967
MT02	0.771	99.8%	0.791
MT03	0.770	99.2%	0.685
MT04	0.773	98.1%	0.548
MT05	0.773	96.3%	0.431
MT06	0.774	94.5%	0.351

(d)

Figure 4.8. Undrained behaviour of desaturated loose sands in triaxial compression (a) stress-strain response; (b) pore pressure response; (c) effective stress path; (d) Testing parameters

4.3.2.2 Extension tests

Figures 4.9 and 4.10 show the undrained response of loose sand in triaxial extension tests. It can be seen that the basic trend in extension tests was similar to that in compression tests: higher strength and lower pore pressure generation were manifested in samples with lower degree of saturation. However, extension tests were much more contractive than compression tests at similar conditions. Samples in Figure 4.9 had similar void ratio as that in compression tests ($e \approx 0.77$). All four

samples in Figure 4.9 manifested strain-softening behaviour. The first three extension tests, MT11, MT12, and MT 13 showed a complete strain softening behaviour: effective stress decreased to zero and pore pressure increased to confining pressure. However, test MT14, with the lowest B-value 0.287, showed a quasi-softening behaviour. When compared with MT06 with the B-value of 0.351 in triaxial compression, the peak strength of MT14 was much lower. Another series of extension tests on loose samples ($e \approx 0.71$) are shown in Figure 4.10. In this series of tests, a complete strain softening behaviour can be turned into strain hardening by merely reduce the S_r from 100% to 87.6%. These two series of tests prove that, although sands are much more contractive in extension tests than in compression tests, the strain-softening (or static liquefaction) can still be prevented by lowering the degree of saturation.

(a)

(b)

(c)

	e_c	S_r	B
MT11	0.769	100.0%	0.950
MT12	0.773	96.4%	0.592
MT13	0.774	94.6%	0.516
MT14	0.772	87.6%	0.287

(d)

Figure 4.9. Undrained behaviour of desaturated loose sands ($e \approx 0.77$) in triaxial extension (a) stress-strain response; (b) pore pressure response; (c) effective stress path; (d) testing parameters

(a)

(b)

(c)

	e_c	S_r	B
MT15	0.707	100.0%	0.986
MT16	0.706	94.6%	0.546
MT17	0.711	87.6%	0.258

(d)

Figure 4.10. Undrained behaviour of desaturated loose sands ($e \approx 0.71$) in triaxial extension (a) stress-strain response; (b) pore pressure response; (c) effective stress path; (d) Testing parameters

4.3.2.3 Gas tube tests

A series of gas tube tests is presented in Figure 4.11. The void ratio and the range of degree of saturation of these tests were similar to the tests presented in Figure 4.8, which were a series of biogas tests. As can be seen in Figure 4.11, the undrained peak strength showed a moderate increasing trend as the degree of saturation reduced. However, the transition from strain softening to strain hardening or the evident reduction in the pore water pressure generation could not be seen, in contrast with the biogas tests in Figure 4.8. Considering that the two series of tests had similar testing parameters, the factor that causes the difference in the undrained behaviour could be different gas distribution patterns: gas bubbles in the biogas samples were relatively uniform, while gases in gas tube samples were heterogeneous. This comparison proves that the undrained behaviour of loose desaturated sand is also affected by the gas distribution: uniform distribution of gases in sand leads to a higher undrained resistance.

(a)

(b)

(c)

	e_c	S_r	B
MT01	0.769	100.0%	0.967
MT07	0.772	98.8%	0.699
MT08	0.771	97.8%	0.589
MT09	0.770	97.0%	0.485
MT10	0.772	94.1%	0.393

(d)

Figure 4.11. Undrained behaviour of gas tube samples (a) stress-strain response; (b) pore pressure response; (c) effective stress path; (d) Testing parameters

4.3.2.4 Comparison between compression and extension test results

The stress-strain curves and effective stress paths of both compression tests and extension tests are compared in Figure 4.12. Note that these two series of test had a similar range of B-value (0.351-0.967 for compression tests and 0.287-0.950 for extension tests). It can be seen that the compression tests showed much higher strengths than the extension tests. Effective stress path curves (Figure 4.12(b)) also showed that the critical state parameter M in extension tests (0.87) was smaller than

that in compression tests (1.22), which agreed with previous experimental studies.

Figure 4.13 shows the normalized undrained shear strength ($= q_u/(2p_0')$, where q_u is the peak deviatoric stress and p_0' is the initial effective confining pressure) of compression and extension tests. For both types of tests, the strength improved significantly with the decrease of B-value. For compression tests, the strength increased from 0.24 to 0.51 as B-value decreases from 0.967 to 0.351, which was a 2.1-fold improvement; for extension tests with $e \approx 0.77$, the strength improved from 0.10 to 0.17 as B-value decrease from 0.950 to 0.287, which was a 1.7-fold improvement; for the other series of extension tests with $e \approx 0.71$, the strength improved from 0.13 to 0.22, which was also a 1.7 fold improvement. The comparison between compression and extension tests shows that, at the same B-value, the normalized undrained compression strength is much larger than that of extension strength.

Besides using strength, liquefaction resistance can also be described by pore water pressure. Pore water pressure ratio r_u is defined as the ratio of the maximum pore water pressure to the initial effective confining stress. The pore water pressure ratio r_u for both compression and extension tests is plotted against B-value in Figure 4.14. The lower the r_u , the stronger the liquefaction resistance. The value of r_u equalling to 1 means that the sample loses all the effective stress and completely liquefies. As shown in Figure 4.14, for compression tests with $e \approx 0.77$, r_u was significantly reduced with the decrease of B-value; for extension tests with $e \approx 0.77$, r_u was close to 1 within the tested range of B-value; for compression tests with $e \approx 0.71$, the trend of r_u was similar to that shown in compression tests. These results prove that both void ratio and degree of saturation are important factors for liquefaction potential of sands: the liquefaction potential can be alleviated by reducing either the void ratio or the degree of saturation.

(a)

(b)

Figure 4.12. Comparison of undrained behaviour of desaturated loose sands ($e \approx 0.77$) in triaxial compression and extension tests (a) stress-strain response; (b) effective stress path

Figure 4.13. Normalized undrained shear strength for loose samples in compression and extension tests

Figure 4.14. Pore water pressure ratio for loose samples in compression and extension tests

4.3.2.5 Instability lines of desaturated sand

It has been discussed in Chapter 2 that, under a load-controlled condition, triaxial samples may collapse in the form of instability. The instability line can be determined as a line that links the origin and peak point in p' - q space as illustrated in Figure 2.7. The instability line (IL) specifies one of the conditions for the occurrence of instability of sand: a stress state located above the instability line is susceptible to instability and a stress state below the instability line is free from instability.

Using three series of effective stress paths obtained from triaxial compression and extension tests as shown in Figure 4.15, the IL can be determined. The slope of IL is denoted as η_{IL} . For both compression tests and extension tests, the value of η_{IL} increased with the decrease of B-value. This implies that sands with lower degree of saturation are less susceptible to instability, that is, a higher q/p' ratio is needed to trigger the occurrence of instability.

(a)

(b)

(c)

Figure 4.15. Instability lines (IL) in compression and extension tests

- (a) compression tests ($e \approx 0.77$); (b) extension tests ($e \approx 0.77$);
 (c) extension tests ($e \approx 0.71$)

4.3.3 Undrained behaviour of medium dense sand

4.3.3.1 Compression tests

A series of undrained compression tests were carried out on medium dense sands as shown in Figure 4.16. The medium dense samples were prepared by water sedimentation method. The relative density of the samples was around 40%.

As shown in Figure 4.16, the typical undrained behaviour in triaxial tests for medium dense sand is different from that for loose sands as presented in Figure 4.8. Medium dense sand showed a much more dilative behaviour than loose sands. Strain hardening behaviour prevailed in all the tests. Pore water pressure change was positive at the initial stage of the test and then became negative with the development of axial strain.

Although all the tests in this series show a similar pattern of the stress strain and the pore water pressure behaviour, there are still some differences for tests with different degrees of saturation. It can be seen in the stress strain curves in Figure 4.16(b) that there was a clear increasing trend in yielding stress as the degree of saturation decreased. Yielding stress here is determined as the deviatoric stress at the point with the maximum curvature on the stress strain curve. Also, the maximum pore water pressure became smaller with the decrease in the degree of saturation. The phase transformation (PT) state, which has been introduced in Chapter 2, corresponds to the maximum pore water pressure (Chu, 1995; Yoshimine and Ishihara, 1998). The PT state was becoming higher with the decrease of the degree of saturation as shown in Figure 4.16(d). The data of yielding stress, pore water pressure ratio r_u and PT state

are summarized in Table 4.3. The relationships of yielding strength and pore water pressure ratio versus B-value are shown in Figures 4.17 and 4.18, respectively. From these trends, it can be concluded that, at small shear strain, the undrained resistance of medium dense sands can also be improved by lowering the degree of saturation. At large shear strain, however, the undrained resistance reduces with the decrease in the degree of saturation, as can be seen in Figure 4.16(a).

(a)

(b)

(c)

(d)

	e_c	S_r	B
WS01	0.692	100.0%	0.958
WS02	0.681	98.1%	0.661
WS03	0.675	94.6%	0.550
WS04	0.657	87.6%	0.271

(e)

Figure 4.16. Undrained behaviour of desaturated medium dense sands in triaxial compression (a) stress-strain response; (b) enlarged stress-strain response; (c) pore pressure response; (d) effective stress path; (e) Testing parameters

Table 4.3. The triaxial test results for medium dense samples

	Test type	B-value	Yielding stress (kPa)	Pore water pressure ratio r_u	Phase transformation	
					p' (kPa)	q (kPa)
WS01	Compression	0.958	58.0	0.500	73.1	78.2
WS02	Compression	0.661	78.0	0.319	108.0	119.6
WS03	Compression	0.550	102.0	0.204	123.2	130.7
WS04	Compression	0.271	132.0	0.088	146.3	165.2
WS05	Extension	0.947	31.0	0.605	31.0	-25.5
WS06	Extension	0.330	39.0	0.344	51.3	-42.9
WS07	Extension	0.198	35.0	0.299	55.7	-43.3

Figure 4.17. Relationship between the yielding strength and the B-value

Figure 4.18. Relationship between pore water pressure ratio r_u and pore pressure coefficient B-value

4.3.3.2 Extension tests

The undrained behaviour for a series of medium dense samples in triaxial undrained extension tests is shown in Figure 4.19. The samples with lower B-value exhibited more dilative response. For two desaturated samples, stress strain curves showed a complete strain hardening response; while the fully saturated sample experienced a quasi-steady state, that is, a reversion from strain-softening to strain-hardening in the stress-strain response as shown in Figure 4.19(a).

Similar to that in undrained compressions tests, extension tests for medium dense sands also showed the trend that, from the saturation to the desaturation state, yielding stress became larger and maximum pore water pressure became smaller, although the improvement in the yielding stress is very small. Moreover, the PT for extension tests increases with the decrease in degree of saturation as shown in Figure 4.19(d). However, within a similar range of B-value, the yielding strength and pore

water pressure ratio for extension tests are much smaller compared with those for compression tests as shown in Figures 4.17 and 4.18. The comparison of the stress-strain behaviour and effective stress path for both compression and extension tests are shown in Figure 4.20. It can be seen that the extension tests showed a much more contractive behaviour than the compression tests.

(a)

(b)

(c)

(d)

	e_c	S_r	B
WS05	0.690	100.0%	0.947
WS06	0.676	94.6%	0.330
WS07	0.684	87.6%	0.198

(e)

Figure 4.19. Undrained behaviour of desaturated medium dense sands in triaxial extension (a) stress-strain response; (b) enlarged stress-strain response; (c) pore pressure response; (d) effective stress path; (e) Testing parameters

(a)

(b)

Figure 4.20. Comparison of undrained behaviour of desaturated medium dense sands in triaxial compression and extension tests (a) stress-strain response; (b) effective stress path

4.4 Conclusions

The following conclusions can be made from the study presented in this chapter:

- For loose sand in the undrained triaxial tests, the undrained shear strength became higher as the degree of saturation became lower. When the pore water pressure coefficient B-value decreased from about 0.95 (saturated state) to around 0.3, the undrained shear strength showed a 2.1-time increase in the compression tests and a 1.7-time increase in the extension tests. Furthermore, the stress strain response changed from complete strain softening to strain hardening with the reduction in the degree of saturation.
- For loose sand in both triaxial undrained compression and extension tests, the reduction in the degree of saturation also led to an increase in the slope of instability line η_{IL} when the void ratio was about the same. This result indicates that desaturation is also an effective way to reduce the susceptibility to prefailure instability.
- For medium dense sand in undrained triaxial compression and extension tests, the reduction in the degree of saturation leads to a higher undrained resistance at small shear strain and a lower undrained resistance at large shear strain. With the decrease in the degree of saturation, samples showed an ascending trend in yielding stress and a descending trend in the maximum pore water pressure generation at relatively small strain. At large strain, the tests showed a strain hardening behaviour.

Chapter 5

CYCLIC BEHAVIOUR OF BIOGAS DESATURATED SAND

5.1 Introduction

This chapter is to explore the cyclic behaviour and liquefaction potential of sand treated with the biogas method using a fully instrumented laminar box mounted on the top of a shaking table.

Laminar box test is a type of soil model test that usually conducted with a shaking table or a centrifuge apparatus. It is widely used to study the dynamic or liquefaction behaviour of soil (Ling et al., 2003; Kagawa et al., 2004; Elgamal et al., 2005; Ueng et al., 2006; Thevanayagam et al., 2009; Sharp et al., 2010; Ueng, 2010; Zhou et al., 2010; Dobry et al., 2011). A laminar box consists of multiple layers of laminates (usually made of metal frames) stacked together. Relative movements are allowed in 1 or 2 directions between two layers, so that soil in the laminar box can deform freely in the lateral direction. There are several types of experiments that can be used in the study of soil liquefaction. However, the cyclic behaviour is largely influenced by different loading conditions and boundary conditions in different experiments. The advantage of laminar box tests over others is that it can reproduce relatively realistic loadings and boundary conditions in an earthquake.

Shaking table tests using other types of model have been carried out to explore the

desaturation effect on soil liquefaction (Okamura and Teraoka, 2006; Yegian et al., 2006, 2007). Okamura and Teraoka (2006) used a rigid sealed box with reduced atmospheric pressure acting on the soil surface. Yegian et al. (2006, 2007) used a strain-controlled model box with flexible wall. The details have been reviewed in Chapter 2. One conclusion that can be drawn from all of these studies is that desaturation of sand can reduce the pore pressure generation and ground subsidence significantly.

5.2 Materials and methods

5.2.1 Denitrifying bacteria and soil desaturation solution

The selection of denitrifying bacteria and the composition of soil desaturation solution have been described in Chapter 3. Different degrees of saturation were obtained by varying the concentration of nitrate added to soil.

5.2.2 The test set-up

A picture of the test set-up is shown in Figure 5.1. It consists of two parts. The upper part is a laminar box and the lower part is a manual shaking table. A metal frame is also installed to surround the laminar box for safety and installation of measuring devices. The details of the design and the test preparation are given below.

Figure 5.1 Model test set-up

5.2.3 Laminar box

A laminar box was made up of 10 layers of aluminum laminates stacked together as shown in Figure 5.2. Interfaces of the laminates were grooved and ball bearings were put in-between, so that the interface friction was very small, which allowed each layer to move freely in the lateral direction during shaking. The laminar box had an inner size of 45 cm $\times$ 30 cm $\times$ 29.7 cm (length $\times$ width $\times$ height) when all the laminates were used. An impervious plastic sheet was lined along the inner boundary before the sand sample was placed as shown in Figure 5.3. The plastic sheet was cut and sewed in a size which was a little bigger than the inner size of the laminar box so that the laminar box could deform freely.

Figure 5.2. Design of the laminar box and instrumentation of transducers (acc: accelerometer; piezo: piezometer; las: laser displacement transducer; lvdt: LVDT displacement transducer)

Figure 5.3. The plastic sheet liner that holds the sand sample

5.2.4 Shaking table

The design of the manual shaking table follows the concept proposed by Prasad et al. (2004) to allow it to provide a horizontal translational seismic loading. A picture of the shaking table is shown in Figure 5.1 and a sketch is given in Figure 5.4. Two pieces of mild steel plates were installed between two wooden boards fixed by angle bars along the transverse direction. The wooden board had a size of 1.4 m × 0.5 m × 2 cm (length × width × thickness). Mild steel plates were 0.3 m in height (distance between tips of angle bars) and 2 mm in thickness. Such a shaking table can be treated as a system with one degree of freedom. The natural frequency can be calculated as:

$$f = \frac{1}{2\pi} \sqrt{\frac{k}{m}} \quad (5.1)$$

where k is the stiffness coefficient of the shaking table and m is the mass of the payload (including the mass of the sand sample, the laminar box, the upper wooden board, and the steel frame). The measured natural frequencies at various payloads are indicated as solid diamond dots in Figure 5.5. The relationship given by Equation 5.1 is also drawn as a curve in Figure 5.5 which shows a good agreement with the measured data. Historical earthquake records showed that most earthquakes that

caused severe damages had frequencies ranging from 1 to 2 Hz (Prasad et al., 2004). A test sample in this study has a mass of around 100 kg. Then the frequency produced by the shaking table can be similar to a real earthquake according to the calculation made using Equation 5.1.

Another important aspect of a seismic event is acceleration. The acceleration in this test was monitored by an accelerometer adhered to the surface of the shaking table. The real-time monitoring data of acceleration were displayed as a graph on a computer monitor. A typical input acceleration chart is shown in Figure 5.6. This test was intended to control the peak acceleration at 1.5 m/s^2 . As can be seen from Figure 5.6, the achieved peak values were in the range of $1.5 \pm 0.1 \text{ m/s}^2$, which was accurate enough for the intended purpose.

Figure 5.4. Design of the manual shaking table

Figure 5.5. Plot of frequency against payload of the shaking table

Figure 5.6. A typical input acceleration (Test 5c)

5.2.5 Sample preparation

The same ASTM graded sand as used in the experiments in Chapters 3 and 4 was also used here. The water sedimentation method was used for the preparation of the sample. The procedure is as follows:

- 1) The laminar box was assembled. After checking the water-tightness of the liner, 2 liters of inoculated bacterial medium (for desaturated samples) or distilled water (for the fully saturated sample) was poured into the laminar box.
- 2) Dry sand was placed into the liquid through a funnel until the sand level almost reached the water level. During deposition, the funnel moved back and forth to allow the sand to be spread uniformly. The sample preparation process was carefully observed to make sure there was no gas bubbles trapped in the sand. The above step was repeated until the sample reached the desired height. The samples prepared were around 27.5 cm in height and the water tables were around 1 cm above the sand surface. The samples prepared in this way gave a relative density around 20%.
- 3) For the tests on biogas sand, the sample was left there for several days until the water level stopped rising. It usually took about 5 days to complete the gas generation process.

In the shaking table tests, the degree of saturation of sand cannot be evaluated by the pore water pressure coefficient B like in triaxial tests. Two other methods were used instead. The first method was to measure the nitrate concentration change by assumption that the reduced nitrate was completely converted to N_2 gas. The second method was to measure the water level rise due to the generation of the gas bubbles in sand. The desired value of S_r (Column 1 of Table 5.1) was calculated according to the

molar ratio in the reaction equation by assuming that nitrate was converted to N_2 gas completely.

Table 5.1. Degree of saturation determined by various methods

	Desired value	Method 1 (nitrate measurement method)	Method 2 (gas volume measurement)
Test 5	80%	79.4%	81%
Test 6	90%	90.7%	92%
Test 7	95%	95.1%	96%

5.2.6 Instrumentations

The layout of the transducers is shown in Figure 5.2. The following five types of transducers were used:

- 1) Two accelerometers (acc1 and acc2) were used to measure the input acceleration of the shaking table and the acceleration of the second top laminate (No. 9), respectively. The model of the accelerometer was TML ARF-10A. The capacity was $10m/s^2$. TML Smart Dynamic Strain Recorder DC-204R was used to record the data.
- 2) Three miniature pore water pressure transducers (pwp1 to pwp3) were used to measure the pore water pressures at the bottom, 2/3 depth, and 1/3 depth of the sample along the centerline, respectively, as shown in Figure 5.2. A small piece of metal was tied to the transducer head and the cable was hung on a bar installed over the laminar box (Figure 5.7), so that the transducer could stay at its original depth even if liquefaction occurred. The model of pore water pressure transducer was Druck PDCR 81 Miniature Pore Water Pressure Transducer. The range of pressure measurement was 3 bar. National Instrument 9219 Module with cDAQ 9178 Chassis was used to record the data of pore water pressure and displacement measured by LVDT (Item 5 as shown below).

Figure 5.7. Installation of pore water pressure transducers and LVDT

- 3) Four laser displacement transducers (las1 to las4) were used to measure the lateral movements of the laminar box at 3 different elevations and the input lateral movement. The model of laser displacement transducers used here and in the next item was Keyence IL-600. The measuring range of this model was 200 to 1000 mm. The data-logger was Keyence NR-600. The transducer heads were clamped on a stand during the test as shown in Figure 5.8.

Figure 5.8. Arrangement of laser displacement transducers for lateral displacement measure of laminar box

- 4) Two laser displacement transducers (las5 and las6) were used to monitor the vertical movements of two concrete blocks placed on the sand surface. The concrete blocks were used to simulate structures resting on the sand. If liquefaction occurred, the structures would sink into the sand. Figure 5.9 shows the placement of the concrete blocks and the arrangement of transducers used to measure them.

Figure 5.9. The arrangement of laser displacement transducers for the measure of structure sinking

- 5) One LVDT was used to measure the subsidence of the sand surface. The model was Wykeham Farrance LSC 50mm Transducer. The installation of LVDT can be seen in Figure 5.7. It was fixed on a bar attached to the metal frame. A piece of foam was attached on the tip of the LVDT with adhesive tape. The bottom of the foam could keep contact with sand surface of during the test. The subsidence could thus be measured.

Table 5.2. Test conditions and test results

	Acceleration (m/s ²)			Relative Density	Volumetric strain	Settlement of structure (mm)		Pore pressure ratio			Cyclic stress ratio			
	Input	soil surface response	amplification coefficient			at center	at adge	Bottom	2/3 depth	1/3 Depth	Bottom	2/3 depth	1/3 Depth	
Test 5(<i>S_r</i> =80%)	Test 5a	0.50	0.57	1.13	20%	0.23%			0.14	0.14	0.13	0.064	0.081	0.147
	Test 5b	1.00	1.18	1.18	21%	0.97%			0.29	0.34	0.23	0.099	0.118	0.186
	Test 5c	1.50	1.80	1.20	27%	1.08%			0.25	0.30	0.34	0.230	0.310	0.626
	Test 5d	1.50	1.68	1.12	33%	0.35%			0.12	0.13	0.19	0.192	0.245	0.459
	Test 5e	2.00	3.00	1.50	35%	1.37%			0.40	0.41	0.59	0.197	0.262	0.462
Test 5f	1.50	1.64	1.09	43%	0.38%			0.11	0.15	0.28	0.230	0.303	0.628	
Test 6(<i>S_r</i> =90%)	Test 6a	0.50	0.54	1.07	17%	0.15%	0.39	0.13	0.08	0.11		0.049	0.057	0.085
	Test 6b	1.00	1.27	1.27	18%	2.70%	0.00	2.14	0.42	0.60		0.153	0.209	0.413
	Test 6c	1.50	1.80	1.20	33%	1.21%	0.77	0.86	0.29	0.42	0.36	0.192	0.250	0.475
	Test 6d	1.50	1.67	1.11	40%	0.31%	0.55	0.94	0.08	0.16	0.17	0.148	0.174	0.280
	Test 6e	1.50	1.62	1.08	42%	0.13%	0.66	0.58	0.03	0.09	0.07	0.229	0.300	0.610
Test 6f	2.00	2.72	1.36	43%	1.03%	4.27	4.02	0.36	0.56	0.59	0.256	0.352	0.719	
Test 7(<i>S_r</i> =95%)	Test 7a	0.50	0.54	1.08	20%	0.21%	0.08	0.00	0.10	0.11	0.06	0.077	0.099	0.191
	Test 7b	1.00	1.44	1.44	22%	5.02%	15.56	6.08	0.55	0.63		0.128	0.179	0.354
	Test 7c	1.50	1.80	1.20	51%	0.66%	2.60	2.22	0.39	0.40	0.47	0.148	0.181	0.313
	Test 7d	1.50	1.64	1.09	54%	0.11%	1.61	0.84	0.14	0.11	0.48	0.231	0.306	0.673
	Test 7e	1.50	1.58	1.05	55%	0.05%	0.83	0.73	0.10	0.06	0.06	0.193	0.243	0.489
Test 7f	2.00	3.56	1.78	55%	0.44%	5.75	6.54	0.54	0.63	0.27	0.198	0.286	0.571	
Test 8(<i>S_r</i> =100%)	Test 8a	0.50	0.54	1.07	21%	5.37%	17.42	15.85	0.55	0.81	0.95	0.064	0.080	0.144
	Test 8b	1.50	2.43	1.62	52%	3.83%	>20	>20	0.86	1.02	1.03	0.148	0.205	0.396
	Test 8c	1.50	2.76	1.84	73%	3.11%	22.77	17.19	0.47	0.70	0.85	0.231	0.388	1.123
	Test 8d	1.50	2.06	1.37	89%	0.29%	2.70	0.38	0.06	0.09	0.12	0.194	0.276	0.746
	Test 8e	2.00	2.86	1.43	91%	0.35%	0.71	1.38	0.09	0.13	0.17	0.198	0.264	0.590

5.3 Test results

5.3.1 Test conditions

Test conditions and some results are listed in Table 5.2. Test 8 was fully saturated sand, and Tests 5, 6, and 7 were bacterial desaturated sand with degree of saturation of 80%, 90%, and 95%, respectively. For each sample prepared, several tests were carried out using different accelerations ranging from 0.05g to 0.2g. The first test was carried out under 0.05g. If liquefaction did not occur and the amount of ground settlement was small, (that is, the change in the density of the sand was small), another test using a higher acceleration would be carried out. A time interval of ten minutes was used between tests for pore water pressure to dissipate fully. If there was no liquefaction and the settlement was small in the second test, a third test at an even higher acceleration would be conducted. This process was repeated until liquefaction or relatively large deformation occurred. For each test, eighteen oscillations at the required acceleration were applied for a duration of around ten seconds as shown in Figure 5.6 as an example. The new relative density was calculated for each test based on the settlement monitored.

5.3.2 Pore water pressure generation

A comparison of the pore water pressure generations in the tests on samples with various degrees of saturation is presented in Figure 5.10. Figures 5.10 (a) to (d) show the pore water pressure generations of loose sands under $a_{max} = 0.5 \text{ m/s}^2$ with $S_r = 100\%$, 95%, 90%, and 80%, respectively. The pore water pressures were measured by the pore water pressure transducers in kPa, but were converted to water heads for the convenience of comparison. Distinct difference can be easily seen between saturated and desaturated samples. The fully saturated sample Test 8a showed a violent pore pressure build-up, followed by a gentle decrease until the end of the loading. However, pore pressures in desaturated samples were all very low as shown in Tests 7a, 6a, and 5a (Figure 5.10 (b), (c) and (d)). Another series of tests at medium dense

state under $a_{max} = 1.5 \text{ m/s}^2$ also clearly shows the effect of desaturation on pore pressure alleviation as shown in Figure 5.10 (e) to (h). A drastic increase in pore water pressure was observed in Figure 5.10(e) for Test 8b with $S_r = 100\%$. The pore water pressure increased to the peak in less than 3 sec and then maintained at almost the peak value for 5 to 10 sec. The pore water pressure generations for Test 7c were substantially smaller than those in Test 8b as shown in (f) even though S_r was reduced to merely 95%. Tests 6e (g) and 5f (h) with $S_r = 90\%$ and 80% , respectively, show a even smaller pore pressure, although densities in these two tests were a little smaller than those in Tests 8b (e) and 7c (f).

The pore water pressure ratio versus relative density of sand curves are plotted in Figure 5.11 using pore water pressure data measured at different elevations. The pore water pressure ratio is defined as the ratio of maximum excess pore water pressure to initial effective overburden stress. It can be seen that at a relative density of 52%, the saturated sample liquefied with a pore water pressure ratio of 1. When the degree of saturation was lowered to 95% for soil with the same relative density, the pore water pressure ratio reduced to be less than 0.5 as shown in Figure 5.11. Similar observations are made in Figure 5.11 for samples with $S_r = 90$ and 80% at lower relative density ranging from 27 to 43%. Thus the use of biogas to reduce the degree of saturation of sand is an effective way to increase the liquefaction resistance of sand. Figure 5.11 also indicates that for the fully saturated sample to attain the same pore water pressure ratio as for partially saturated sand, the relative density of the sample has to be around 90%.

Table 5.2 also shows that the pore water pressure ratio became smaller with increasing depth. There are two possible reasons that can explain this trend. First, the soil acceleration becomes smaller as soil goes deeper (as will be seen Section 5.3.5), causing a smaller shear stress to the soil at deeper location. Second, in a short

duration, excess pore pressure does not have enough time to dissipate, leading to an accumulation of pore pressure along the drainage path from the bottom to the top of the sand sample.

(a) Test 8a

(b) Test 7a

(c) Test 6a

(d) Test 5a

Figure 5.10. Pore water pressure

- (a) Test 8a ($D_r=21\%$, $S_r=100\%$, $a_{max}=0.5 \text{ m/s}^2$);
- (b) Test 7a ($D_r=20\%$, $S_r=95\%$, $a_{max}=0.5 \text{ m/s}^2$);
- (c) Test 6a ($D_r=17\%$, $S_r=90\%$, $a_{max}=0.5 \text{ m/s}^2$);
- (d) Test 5a ($D_r=20\%$, $S_r=80\%$, $a_{max}=0.5 \text{ m/s}^2$);
- (e) Test 8b ($D_r=52\%$, $S_r=100\%$, $a_{max}=1.5 \text{ m/s}^2$);
- (f) Test 7c ($D_r=51\%$, $S_r=95\%$, $a_{max}=1.5 \text{ m/s}^2$);
- (g) Test 6e ($D_r=42\%$, $S_r=90\%$, $a_{max}=1.5 \text{ m/s}^2$);
- (h) Test 5f ($D_r=43\%$, $S_r=80\%$, $a_{max}=1.5 \text{ m/s}^2$)

(a)

(b)

(c)

Figure 5.11. Plots of pore water pressure ratios against relative densities under $a_{max}=1.5 \text{ m/s}^2$ when pore pressure was measured by piezometers at (a) bottom, (b) 2/3 depth and (c) 1/3 depth.

5.3.3 Volumetric strain

Compared with the fully saturated sand, desaturated sand manifested a much smaller volumetric change. The volumetric strain was calculated using the surface settlement by assuming the soil deformed one-dimensionally. Figure 5.12 (a) and (b) show the development of volumetric strains of loose sands under $a_{max}=0.5 \text{ m/s}^2$ and medium dense sands under $a_{max}=1.5 \text{ m/s}^2$, respectively. Fully saturated samples showed substantial increases in volumetric change at both loose state and medium dense state, the corresponding volumetric strains were around 5% and 4%, respectively. However, at similar range of densities and input acceleration, volumetric strains in desaturated samples were by far smaller than those in fully saturated samples. Volumetric strains in all desaturated samples in Figure 5.12 (a) and (b) are smaller than 1%. Tokimatsu and Seed (1987) pointed out that pore water pressure generation and volumetric strain also occurred in a non-liquefiable soil, however, the magnitude was often much smaller, and the volumetric strains observed in non-liquefiable cases were usually less than 1%. As shown in Figure 5.12, the levels of volumetric strains for desaturated soil were mostly within 1% strain, indicating that the desaturated soil had strong resistance to liquefaction and volumetric change. It is also interesting to note that the data shown in Figure 5.13 follow the same pattern as that in Figure 5.11. It implies that when the pore water pressure ratio was higher, the volumetric change would be higher too.

(a)

(b)

Figure 5.12. Volumetric strain for tests with various degrees of saturation (a) Loose sands, $a_{max}=0.5 \text{ m/s}^2$; (b) Medium dense sands, $a_{max}=1.5 \text{ m/s}^2$

Figure 5.13. Plot of volumetric strains against relative densities ($a_{max}=1.5 \text{ m/s}^2$).

5.3.4 Sinking of structures in sand

It is well known that when soil liquefies, a structure on top of the soil deposit will sink into the soil. Two circular concrete blocks of 40 mm in height and 73 mm in diameter were used to simulate the structure resting on the sand surface in the model. One block was placed along the centerline and the other near the edge of the laminar box as shown in Figure 5.2. The base pressure acting on the soil was 4.8 kPa.

Figure 5.14 shows the depth of structure sinking of samples at loose state under $a_{max}=0.5 \text{ m/s}^2$ ((a) and (b)), and at medium dense or dense state under $a_{max}=1.5 \text{ m/s}^2$ ((c) and (d)). Figures 5.14 (a) and (c) were the measured at centerline of the laminar box, and (b) and (d) were measured at edge of the laminar box. The depth of soil settlement has been deducted from the structure sinking data. Thus, the curves in Figure 5.14 are the structure sinking relative to the soil surface. In these four figures, distinct comparison can be seen between saturated and desaturated samples. The depths of structure sinking on fully saturated samples were around 20 mm, which implies that the bearing capacity of soil has completely lost. In contrast, the depths of structure sinking in desaturated samples were only about 1mm. This comparison demonstrates that desaturation is a good way to control structure dislocation during earthquake liquefaction. Note that, in Figure 5.14, the settlements of structures lasted around 5 seconds, which was shorter than the duration of seismic loading applied (around 10 seconds). This is because the buoyant force at that depth is large enough to balance the weight of the structure.

(a) Loose sands, $a_{max}=0.5 \text{ m/s}^2$, at centerline

(b) Loose sands, $a_{max}=0.5 \text{ m/s}^2$, at edge

(c) Medium dense and dense sands, $a_{max}=1.5 \text{ m/s}^2$, at centerline

(d) Medium dense and dense sands, $a_{max}=1.5 \text{ m/s}^2$, at edge

Figure 5.14. Sinking of structures under loose state and $a_{max}=0.5 \text{ m/s}^2$ ((a) and (b)), and under medium dense and dense state and $a_{max}=1.5 \text{ m/s}^2$ ((c) and (d)) ((a) and (c) were the measured at centerline of the laminar box, and (b) and (d) were measured at edge of the laminar box)

5.3.5 Response of the laminar box

During the model tests, the accelerations at the sand surface were amplified to some extent as measured by the accelerometer installed on the second top (or 9th) layer of the laminar box. The acceleration amplification coefficient is defined as the ratio of acceleration measured at the soil surface to that at the base. The acceleration amplification coefficients are plotted versus relative density in Figure 5.15. It can be seen that the acceleration amplification coefficients decreased with the increase in the void ratio and the decrease in the degree of saturation, which is similar to the trends shown in Figures 5.11 and 5.13.

Figure 5.15. Acceleration amplification coefficient versus relative densities ($a_{max}=1.5$ m/s²)

The lateral displacements of the laminar box at different elevations were monitored during tests. As an example, the lateral displacements in a non-liquefied test (Test 5c) and a liquefied test (Test 8c) are shown in Figure 5.16. Figure 5.17 shows lateral displacements at different elevations at one of the peak excursions of shaking wave for Tests 5c and 8c, respectively. It can be seen that the displacements at different elevations are rather random. The largest displacement relative to the base is around

0.2mm. This value corresponds to a shear strain of around 0.2%. This is a small value of shear strain compared with that observed in cyclic triaxial tests. The observations of lateral displacements in other tests were similar to these two tests. This might be because the laminar box was put in the horizontal position and thus there was a lack of a triggering factor to cause a large permanent lateral displacement.

(a)

(b)

(c)

(d)

Figure 5.16. Lateral displacement

(a) input displacement in Test 5c;

(b) displacement of the 9th laminate in Test 5c;

(c) input displacement in Test 8c;

(d) displacement of the 9th laminate in Test 8c.

Figure 5.17. Lateral displacements at one of the peak points of shaking waves (around 8th second) (a) Test 5c; (b) Test 8c.

5.4 Discussions

5.4.1 Pore pressure and volumetric change

Based on the model tests, a relationship between volumetric strain ε_v and pore water pressure ratio r_u is established in Figure 5.18. r_u was determined using the average value measured by pore pressure transducers at three elevations as shown in Figure 5.2. Figure 5.18 contains tests with various input accelerations. It can be seen that the volumetric strain starts to increase in an accelerating rate when the pore water pressure ratio is higher than 0.4. This observation is consistent with those made by other studies (Lee and Albaisa, 1974; Tokimatsu and Seed, 1987). As a single relationship is formed, the trend line appears to be applicable to soil with different densities, degrees of saturation, and input acceleration. At the same value of r_u , denser sand seemed to have a smaller volumetric strain than looser sand, however this trend is not obvious. This observation agrees with Tokimatsu and Seed (1987). Tokimatsu and Seed (1987) also stated that, for practical purposes, the relationship between r_u and ε_v could be represented by a single line when r_u was smaller than 0.6 and relative density was in the range of 30% to 80%. According to the test results in this study, such a relationship seems applicable for desaturated sands. Hence, the pore

water pressure ratio r_u may be a good parameter to use in the design of deformation control.

Figure 5.18 also shows that reduction of the degree of saturation of saturated sand using biogas is an effective method in the mitigation of liquefaction. Most of the pore water pressure ratios for desaturated samples were smaller than 0.4.

Figure 5.18. Plot of volumetric strains against pore pressure ratios

5.4.2 Pore pressure and sinking of structures

Figure 5.19 shows the relationship between the depth of sinking of structures and pore water pressure ratio. Data points in this figure contain tests with different degrees of saturation, densities, and input accelerations. The results show that: When pore water pressure ratio is in the range of 0 to 0.4, the sinking of structure is small irrespective of pore water pressure ratio; whereas when the ratio goes beyond 0.4, the depth of the sinking shows a violent increasing trend as pore water pressure ratio

becomes larger. These results are similar to the relationship between volumetric strain and pore water pressure ratio as shown in Figure 5.18. In both relationships, a pore water pressure ratio of 0.4 seems to be a threshold that separates non-liquefaction and liquefaction.

The effect of desaturation on the control of structure sinking was shown in previous studies. By conducting shaking table soil model tests on both saturated and desaturated sands, Okamura and Teraoka (2006) showed that, during earthquake shaking, the sinking the structure resting the sand surface can be significantly alleviated by introducing small amount of gas bubbles into the sand. Moreover, the magnitude of sinking depth was related to the level of pore water pressure generation, that is, the smaller the sinking depth, the smaller the pore water pressure. These observations agreed with the results in this study.

Figure 5.19. Plot of sinking of structures against pore pressure ratios

5.4.3 Relationship between model test results and field performances

In this study, the small size laminar box has a depth of around 30cm. However, depths of the real liquefiable soil deposits are usually deeper than this. This difference leads the discrepancy in drainage length between two conditions. In turn, the drainage length can affect volumetric strain and pore pressure generation. Figure 5.20 shows the relationship between volumetric strain and pore water pressure ratio in field conditions during true earthquakes (Tokimatsu and Seed, 1987). The Comparison between Figures 5.18 and 5.20 shows that volumetric change in small size model tests are larger than that in field condition given the same level of pore water pressure ratio in the similar range of relative densities. However, the patterns of these two curves are very similar, that is, there is a parabolic increase in volumetric strain when pore water pressure ratio rises larger than around 0.5. Evaluations of ground settlement can use such relationship.

Figure 5.20. Relationship between volumetric strain and induced pore water pressure ratio in the field condition (Lee & Albalsa, 1974, Figure copied from Tokimatsu and Seed, 1987)

5.5 Conclusions

Cyclic behaviour of bacterial desaturated sand has been studied in this chapter. Shaking table tests with a fully instrumented simple shear laminar box were conducted on samples with various degrees of saturation. Four samples had an initial relative density of around 20% and degrees of saturation of 100%, 95%, 90%, and 80%, respectively. Each sample was tested several times with various accelerations, so that the effect of void ratio and acceleration could also be seen in the results. The following conclusions can be made:

- Test results showed that liquefaction occurred for saturated samples at loose states under $a_{max}=0.5 \text{ m/s}^2$ and at medium dense states under $a_{max}=1.5 \text{ m/s}^2$. Liquefaction did not occur for biogas desaturated samples at the same conditions. To achieve a non-liquefaction manner under $a_{max}=1.5 \text{ m/s}^2$ as that in desaturated samples, saturated samples need a relative density as high as around 90%. The occurrence of liquefaction is manifested by several observations simultaneously, including drastic pore water pressure build-up, excessive ground subsidence, sinking of structures resting on the soil surface, and large amplification of acceleration.
- The relationship between the pore pressure ratio and the volumetric strain can be represented by a single line (Figure 5.18). The volumetric strain rises at an accelerating rate with the increase of the pore pressure ratio. The relationship between the pore pressure ratio and the depth of the sinking of structure can also be shown by a single line (Figure 5.19). The sinking of structure becomes deeper as pore pressure ratio increases. In these two relationships, pore pressure ratio r_u of 0.4 seems to be a threshold. When $r_u < 0.4$, The volumetric strain is small and there is almost no sinking of structures, and when $r_u > 0.4$, both the volumetric strain and the sinking of structures starts rising violently. Such relationships can

be used for evaluating the ground and structure performance during liquefaction.

Chapter 6

MORPHOLOGY AND STABILITY OF BIOGAS IN SAND

6.1 Introduction

This chapter discusses two topics: the morphology of gas bubbles in sand; and the stability of gas bubbles at both flow and hydrostatic conditions.

Chapter 4 pointed out that gas distribution pattern can affect the undrained resistance of sands. The biogas samples had relatively uniform gas distribution, and the gas tube samples had a heterogeneous gas distribution pattern. The liquefaction resistance of biogas desaturated samples was much stronger than that of gas tube samples. In this chapter, two direct observation techniques are used to answer the question of how gas bubbles exist in sand. One method is to use a photographic system with a macro lens to observe the sample in a transparent container. The other method is to use computer tomography (CT) technique to observe the inner structure of the soil sample.

Whether gas bubbles can retain in sand for a long duration is a key concern for this technique. However, there was little information on this topic in previous studies. In this study, a specially designed set-up is used to detect the presence of gas bubbles in sand under hydrostatic and water flow conditions.

6.2 Methodologies

6.2.1 Gas observation using the photo imaging method

A single lens reflex camera with a macro lens was used for the observation of gas morphology in sand. The model of the camera is Nikon D3100. The model of the lens is Nikon AF-S VR Micro-Nikkor 105mm f/2.8G IF-ED. The lens has a closest focusing distance of 0.31 m. In order to improve the stability of the images, the camera was installed on a tripod when taking pictures. Adobe Photoshop CS4 was used to process original images, so that solid, liquid and gas phases in the sand could be easily identified. This observation technique was firstly adopted by Eseller-Bayat (2009) and Gokyer (2009). Figure 6.2 shows the comparison of an original image (a) and a software enhanced image (b). The original image is fuzzy and it is difficult to distinguish different phases. While the software enhanced image is much clearer. There are some occluded gas bubbles among sand particles. The difference between gas bubbles and sand particles can be identified by their profiles and surface textures. Gas bubbles have smooth profile, while sand particles have irregular shapes. Gas bubbles have reflective surface texture, which cannot be seen on sand particles.

A rectangular cross-section, transparent flask was used for the preparation of sand samples as shown in Figure 6.1. ASTM graded sand, which was the same sand as used in the experiments shown in previous chapters, was also used here. The sample preparation procedure was the same as described in Chapter 3. As can be seen in the figure, some square grids with side-length of 1 cm was drawn on the wall of the flask, which helped to scale the size of gas bubbles and sand particles.

Figure 6.1. Sample for the observation of gas morphology

(a)

(b)

Figure 6.2. Sample images of sands (a) original image; (b) software enhanced image

Four samples with various degrees of saturation were prepared. The values of degrees of saturation and void ratios for each sample are shown in Table 6.1. For each sample, the volume of the gas produced could be observed through the water level rise in the transparent bottle. Besides, the dry mass, the bulk volume and the specific gravity of the sand, and the volume of the liquid are all known. Degree of saturation and void ratio for each sample could thus be determined through phase diagram.

Table 6.1. Degree of saturation and void ratio of samples for gas observation tests

	Degree of saturation	Void ratio
1	94.4%	0.69
2	90.6%	0.68
3	87.9%	0.69
4	84.1%	0.68

6.2.2 Sand observation using the computer tomography (CT) technique

Computer tomography (CT) technique was used to observe the inner structure of the sand sample. The CT apparatus used was a high-resolution digital radiography and computed tomography system assembled at Center for Non-Destructive Evaluation (CNDE) at Iowa State University. The equipment consists of the following components:

Thermo Kevex model PXS10-16W microfocus x-ray tube (X-ray generation source);
 Varian Paxscan 2520 amorphous silicon x-ray detector with GOS conversion screen (X-ray imager);
 4-axis sample positioner;
 PC interface.

The specifications of the equipment are:

X-ray imager- 1536×1920 0.127 mm pixel, 12- or 16-bit digital images;
 Effective pixel size- $3\mu\text{m}$ to $250\mu\text{m}$ depending on magnification and mode;

Maximum voltage- 130 kV;

Maximum current- 0.36 mA (low magnification);

Maximum power- 16W;

Capable of producing 3-dimensional CT images with voxel size as small as 4 μ m.

A plastic test tube with inner diameter of 1.46 cm was used to hold the sand sample.

Sample preparation method was the same as used in Section 6.2.1. Four samples with $S_r = 100\%$, 97%, 94%, and 88%, respectively, were prepared.

6.2.3 Gas stability test

To check the stability of gas bubbles, a testing set-up as shown in Figure 6.3 was adopted. The set-up is schematically illustrated in Figure 6.4. It consisted of a Plexiglas column with an inner diameter of 7.04 cm, and two water reservoirs linked to the bottom and top of the column to regulate the water head of the flow through the sample. The length of the column was 1.5 m. The length of the sand sample was around 1 m. There was a 25cm-thick layer of gravels placed at the bottom of the column. Samples were tested under either upward or downward seepage flow condition at a water head difference of 10 cm. Thus, the hydraulic gradient was maintained at 0.1. The flow of water was supplied by pumping tap water from the pail using a submersible pump. The sample preparation method was the same as that used in chapter 3. This test set-up was modified from the set-up used by Yegian et al. (2007) and Eseller-Bayat (2009).

Gas stability in sand column could be evaluated by three methods: measuring the permeability of sand, measuring directly the volume of gas coming out of the specimen, and visual observation. For the first method, the coefficient of permeability could be determined by measuring the quantity of water flowing out of the sand column at certain time duration at a constant head. For the second method,

gas bubbles could possibly be taken out of the sand column in either gas phase or dissolved form. If gas bubbles came out in gas phase, these gas bubbles would accumulate at the top of the column. Then, the volume of this portion could be measured. If gas was taken away in dissolved phase, water would occupy the space where gas bubbles were originally existed. The reduction in gas volume in this form could be evaluated by measuring the increase of the weight of the column. This was done by an electronic balance as shown in Figures 6.3 and 6.4. For the third method, gas bubbles could be visually observed, because the Plexiglas column was transparent.

Figure 6.3. Set-up of gas stability test

Figure 6.4. Illustration of the set-up of gas stability test

Three samples were conducted under hydrostatic, upflow, and downflow conditions, respectively. The initial conditions of these three samples are shown in Table 6.2.

Table 6.2. Initial conditions of samples in gas stability test

	Degree of saturation	Void ratio
Upward seepage	88.7%	0.694
Downward seepage	88.8%	0.692
Hydrostatic	88.6%	0.693

6.3 Results and discussions

6.3.1 Observation of gas using the photo imaging

Figures 6.5 to 6.8 show the images of samples with S_r of 94.4%, 90.6%, 87.9%, and 84.1%, respectively. After the water level rise stopped (the gas generation process was completed), images of the samples were taken. Two images are shown for each sample. Gas bubbles in the sample are circled by red lines. The size, shape, and distribution of the bubbles can be easily seen in these images. Note that ASTM graded sand used here has a mean size of 0.4 mm, which can be used to scale and compare the size the gas bubbles in the same image.

Figure 6.5 shows two images of the sample with $S_r = 94.4\%$. The size of gas bubbles in these two images is around 0.2 mm, which is similar or a little smaller than the size of sand particles. Most of the bubbles were single pocket, although it was possible that the bubbles might have been the result of aggregating of small gas bubbles to form larger gas bubbles. There were around 20 gas bubbles in a 1-cm² size image. The bubbles are distributed randomly in the sand.

The sample with $S_r = 90.6\%$ is shown in Figure 6.6. The size of gas bubbles was around 0.3 mm, which is similar or a little larger than those in the sample with $S_r = 94.4\%$. The gas bubbles were randomly distributed in the image. There were around 25 gas bubbles in a 1-cm² size image.

The sample with $S_r = 87.9\%$ is shown in Figure 6.7. Some bubbles were in the single form, but some others were aggregated together. Single bubbles have sizes of around 0.3 mm. Aggregated bubbles have irregular shapes. The size of the throats of aggregated bubbles is around 0.6 mm. The number of gas bubbles in a 1-cm² size image was s around 25.

The sample with $S_r = 84.1\%$ is shown in Figure 6.8. The size and shape of gas bubbles were similar to those in the sample with $S_r = 87.9\%$. Some bubbles were single with a size of around 0.3 mm, and some others were aggregated. The throat of aggregated bubbles was around 0.6 mm. There were around 30 bubbles in 1 cm² area.

By comparing these images, it can be concluded that: (1) with the decrease in the degree of saturation, the size of gas bubbles becomes larger and the number of gas bubbles becomes more; (2) gas bubbles are relatively randomly distributed in sand; (3) gas bubbles in sands with S_r higher than 90% are mostly single, while more bubbles are aggregated when $S_r < 90\%$; (4) when S_r is in the range of 84.1% to 94.4%, gas bubbles are basically in insular condition (i.e. discontinuous gas phase and continuous liquid phase). These results agreed with Fredlund and Rahardjo (1993), which stated that gas bubbles in soils with S_r larger than 85% were usually in the insular form, and gas phase gradually became continuous as S_r reduced to around 85% or lower.

(a) Image 94B2

(b) Image 94C6

Figure 6.5. Images of sand with $S_r = 94.4\%$ and $e = 0.69$ (a) Image 94B2; (b) Image 94C6

(a) Image 91B2

(b) Image 91C1

Figure 6.6. Images of sand with $S_r = 90.6\%$ and $e = 0.68$ (a) Image 91B2; (b) Image 91C1

(a) Image 88A4

(b) Image 88B1

Figure 6.7. Images of sand with $S_r = 87.9\%$ and $e = 0.69$ (a) Image 88A4; (b) Image 88B1

(a) Image 84A5

(b) Image 84B4

Figure 6.8. Images of sand with $S_r = 84.1\%$ and $e = 0.68$ (a) Image 84A5; (b) Image 84B4

Gokyer (2009) provided a method to calculate degree of saturation of sand using images. The equation is:

$$S_r = 1 - \frac{R}{n} \quad (6.1)$$

where R is the ratio of area of gas bubbles to the total area of the image; n is the porosity of sand. The deduction of this equation is introduced here. Firstly, it should be assumed that the ratio of the area occupied by gas bubbles to the total area of the image equals to the ratio of gas volume to the total volume of the sample (It is statistically true if the size of gas bubbles are randomly distributed and small enough compared with the size of the image). Then the following relationship is correct:

$$R = \frac{V_{gas}}{V_{gas} + V_{liquid} + V_{solid}} \quad (6.2)$$

The porosity of sand is:

$$n = \frac{V_{gas} + V_{liquid}}{V_{gas} + V_{liquid} + V_{solid}} \quad (6.3)$$

Divide Equation 6.2 by Equation 6.3, we can get:

$$\frac{R}{n} = \frac{V_{gas}}{V_{gas} + V_{liquid}} = 1 - \frac{V_{liquid}}{V_{gas} + V_{liquid}} = 1 - S_r \quad (6.4)$$

Rearrange Equation 6.4, then we can have Equation 6.1.

This calculation method was used here. For each sample, three areas are selected for the determination of degree of saturation. The size of each area is $2 \times 2 \text{ cm}^2$. The results are plotted in Figure 6.9 against degree of saturation calculated by volume replacement method as introduced previously. This figure shows that degrees of saturation determined by two methods are similar. Imaging method provides local degree of saturation. Volume change method gives overall degree of saturation. Thus, a maximum of 4% difference is seen between local S_r and overall S_r . For each sample, although there is a little scatter in S_r at different locations of a sample, relatively

uniform distribution of gas bubbles throughout the sample can still be assured.

Figure 6.9. Plot of degree of saturation determined by imaging method and volume change method

6.3.2 Computer tomography (CT) observation of sand samples

The mechanism of the CT technique is to use the relationship between X-ray attenuation and material density so as to map the density distribution of the material(s). The product of CT analysis is volumetric maps of variation of material density. Denser materials have brighter pixels, and lighter materials have darker pixels. Figures 6.11 to 6.14 are CT images on samples with $S_r = 100\%$, 97%, 94%, and 88%, respectively. From the brightness contrast of the images, we can see that there are basically three brightness levels in the images, which correspond to three phases in the samples. The brightest pixels are sand particles which have a unit weight of around 2.65. Medium bright pixels occupying most of the pore space represent liquid phase, which has a unit weight of around 1. In desaturated samples, there are some dark spots which are gas bubbles. These images show the size, shape,

and distribution of different phases in the sample. These observations are also accordant with the commonsense what these phases are supposed to look like.

For each sample, there are 500 slides of images in XY direction (horizontal cross-section) and 436 slides of images in Z direction (vertical cross-section), as illustrated in Figure 6.10. The distance between two consecutive slides is $36\text{ }\mu\text{m}$.

(a)

(b)

Figure 6.10. Illustration of CT images

(a) horizontal slides; (b) vertical slides

The CT images of the saturated sample are shown in Figure 6.11 for both vertical and horizontal cross sections. It can be seen from the images that the distribution of sand particles and pore voids were quite uniform.

The CT images of a desaturated sample with the degree of saturation of 97% are shown in Figure 6.12. It can be seen that there were some relatively large pockets marked by red arrows in Figure 6.12 (a) and (b). These large pockets indicate the formation of the metastable structure of unsaturated sand samples. However, a closer examination of these pockets reveals that the pockets were occupied by clusters of water and gas bubbles, as shown in a locally enlarged photo in Figure 6.12(f). This figure is a zoom-in photo of the red box area of Horizontal Slide 200 (Figure 6.12(a)) and processed by increasing the contrast and brightness.

The CT images of the samples with the degrees of saturation of 94% and 88% are shown in Figures 6.13 and 6.14, respectively. Similar observations were made. There were also pockets in these images. The pockets have more or less similar size to those in the sample with the degree of saturation of 97%. However, the numbers of pockets appeared to increase with the reduction in degree of saturation.

It is interesting to compare the gas morphologies and distributions in sand induced by different gas desaturation methods. Eseller-Bayat et al. (2012) adopted a chemical sodium perborate to generate gases and desaturate sand. The photo images showed that gas bubbles in sand were small (0.1-0.3 mm) and uniform. Okamura et al. (2011) reported that the degree of saturation ranged from 68%-98% in an in-situ gas injection test. Based on these experimental results, it can be seen that chemical and microbial methods for the gas generation result in a uniform distribution of gases in comparison to the direct gas injection method. The reason could be that the liquid-phase medium can flow easily in soil pores which can help form a uniform

distribution of the medium, and once the gas bubbles are generated from soil pores, they are relatively stable.

(a) Horizontal Slide No. 227

(b) Horizontal Slide No. 255

(c) Vertical Slide No. 188

(d) Vertical Slide No. 244

(e) The enlarged image of Horizontal Slide No. 255

Figure 6.11. CT images of the sample with $S_r=100\%$ and $e=0.63$

Figure 6.12. CT images of the sample with $S_r=97\%$ and $e=0.64$

Figure 6.13. CT images of the sample with $S_r=94\%$ and $e=0.66$

(a) Horizontal Slide 247

(b) Horizontal Slide 270

(c) Vertical Slide 172

(d) Vertical Slide 221

(e) The enlarged image of Horizontal Slide No. 247

(f) The software enhanced enlarged image of Horizontal Slide No. 247

Figure 6.14. CT images of the sample with $S_r=88\%$ and $e=0.66$.

6.3.3 Gas stability in a sand column

Gas stability tests on sand samples were carried out at hydrostatic, upflow, and down flow conditions, respectively. Observations and measurements have been done on the variations of gas content, permeability, void ratio in each sample. The brief results are: Under the hydrostatic condition, there was almost no variation in gas content, permeability and void ratio in a duration of 10 days; However, under flow condition with a hydraulic gradient of about 0.1, a change in degree of saturation from 89% to 100% was observed in about 4 day's time. Details of the results and discussions are presented here.

Visual observation

Gas bubbles in the sand column can be visually observed during the test. The growth of saturation zone for three tests is shown in Figure 6.15. Under hydrostatic condition, there was no variation in the distribution of gas bubbles in the sample during the test. Under water flow conditions, gas bubbles gradually disappeared from the upstream to the downstream and saturation zone gradually expanded according to the results of visual observation as shown in Figure 6.15. Under upward flow condition, saturation zone was observed in the bottom part of the column initially, and then the saturation zone gradually expanded from the bottom up, the sand column became completely saturated in about 6 days. Under downward flow condition, saturation zone started at the top of the column and expanded from the top down, the sand became fully saturated at the third day of the test. The rate of gas erosion under upflow condition is a little faster than that under downflow condition. Both upflow and downflow conditions demonstrate that saturation zone grows from upstream to downstream of the sand column.

Gas bubbles in sand under flow condition can potentially be taken away in either gas phase or dissolved phase. In the test condition used here, no gas bubbles was

observed to be taken away in gas phase (if gas bubbles came out of sand, they would accumulated at the top of the Plexiglas column as shown in Figure 6.4). Thus the erosion of gas bubbles is only because of dissolving and water flow. This observation also agreed with previous study (Matyas, 1967), which pointed out that gas flow through pore water was due to dissolving in the condition when gas phase is discontinuous.

Figure 6.15. Change in the visual observation of saturated zone

Overall degree of saturation

The variation in degree of saturation can also be manifested by an increase in the weight of Plexiglas column due to the replacement of gas bubbles by water. The converted overall degree of saturation is shown in Figure 6.16. The sand column became saturated in about 5 day under upflow conditions and about 3 days under downflow conditions. Note that curves in Figures 6.15 and 6.16 are very similar, which implies that the degrees of saturation (or gas content) measured by these two methods are coincident.

Figure 6.16. Change in overall degree of saturation

Permeability

The variation in coefficient of permeability k is shown in Figure 6.17. The permeability of sand was obtained by measuring the volume of water flowing out of the sand column in a fixed time duration. As mentioned previously, a water head of 10 cm was maintained for samples under flow conditions. There was almost no change in k under hydrostatic condition. For sands under either upward or downward flow condition, there was an increase in k from 0.4 cm/s to about 0.54 cm/s in 4 and 2 days, respectively. Also note that Figure 6.17 shows a very similar pattern to that in Figure 6.15. This similarity can be explained by relationship between degree of saturation and permeability. Brooks and Corey (1964) proposed a relationship as:

$$k = k_s S_e^\delta \quad (6.5)$$

where k_s is the coefficient of permeability at fully saturated condition, S_e is the effective degree of saturation, δ is an empirical constant, and a value of 3 can be used for uniform sand. The effective degree of saturation is defined as:

$$S_e = \frac{S - S_r}{1 - S_r} \quad (6.6)$$

where S_r is residual degree of saturation at which the increase of matric suction does not lead to significant change in k .

Equation 6.5 manifests that permeability increases as degree of saturation becomes larger. A simple calculation using Equation 6.5 shows that the value of k at $S_r = 89\%$ is around 70% of that at fully saturated condition, which agrees with the test result shown in Figure 6.17 (coefficient of permeability when $S_r = 89\%$ is around 74% of that when sample became fully saturated).

Figure 6.17. Change in the coefficient of permeability

Void ratio

Figure 6.18 shows that the variation in void ratio basically coincides with the variation in degree of saturation as shown in Figures 6.15, 6.16 and 6.17. There was almost no change in void ratio under hydrostatic condition. Under water flow conditions, There was a small decrease in void ratio, from about 0.695 to 0.675 (the corresponding relative density was from 35% to 42%). The variation in void ratio

was not significant. This reduction in the void ratio could be due to the collapse of the metastable structure of gas pockets as shown in the CT images.

Figure 6.18. Change in the void ratio

Erosion of gas against flow quantity

The test results shown above prove that the major reason for loss of gas bubbles in sand is water flow. Thus, it is natural to show the relationship between erosion of gas and flow quantity. Such a plot is shown in Figure 6.19. The initial degree of saturation is around 89%, that is, around 11% pore space is occupied by gas bubbles. As water flow flushes the pore space, the gas content gradually decreases. After about 90 times of replacement of pore volume at upward flow condition and about 60 times at downward flow condition, gas bubbles are completely lost. This relationship can be used to quantitatively evaluate the variation in degree of saturation under flow condition.

Figure 6.19. Plot of gas content against flow quantity

Gas flow in unsaturated soil at insular gas phase is due to water flow when there is water head difference, and is due to diffusion when there is concentration difference of dissolved gas (Fredlund and Rahardjo, 1993). Here are some simple evaluations in these two conditions.

Under flow condition, we have seen the rate of gas erosion. That is, about 11% gas volume of pore space can be completely eroded in about 4 day's time.

Under hydrostatic condition, gas diffusion is governed by Fick's law, which states that the diffusive flux (the amount of substance goes through unit area per unit time) is proportional to the concentration gradient under the assumption of steady flow. At one dimensional condition, Fick's law can be expressed as:

$$J = D \frac{\partial C}{\partial x} \quad (6.7)$$

where J is the diffusive flux, D is the coefficient of diffusion, $\partial C / \partial x$ is the gradient of concentration. Coefficient of diffusion D is a very small value for N_2 gas, $2 \times 10^{-9} \text{ m}^2/\text{s}$.

A simple calculation is conducted here. Assuming that the concentration gradient is 0.015 (this value is equivalent to the condition where dissolved gas concentration head from 1.5% (saturated with dissolved N₂ gas) to 0 (no dissolved N₂ gas) at 1 meter distance. Then diffusion rate of gas is:

$$J = D \frac{\partial C}{\partial x} = 2 \times 10^{-9} \cdot 0.015 \text{ m/s} = 3 \times 10^{-11} \text{ m/s} \approx 1 \text{ mm/year} \quad (6.8)$$

This rate is orders of magnitude smaller than the rate under water flow condition. This comparison explains the difference that gas bubbles can retain in the sample for much longer time at the hydrostatic condition than at the water flow condition.

As water flow through sand can erode the gas bubbles inside, measures should be taken to control the water flow. One possible way is to induce bioclogging effect together with the biogas generation. As introduced in Chapter 2, microbial denitrification can also be used as a biocement (van Paassen et al., 2010a). Therefore, we can use denitrification process to exert both effects: the generation of gases to mitigate liquefaction potential and the formation of bioclogging to control water flow. This could be a possible follow-up research from this study.

6.4 Conclusions

Observations of biogas desaturated sand samples were carried out by two techniques, a photo imaging technique and a computer tomography (CT) technique. Gas stability test was conducted using a sand column model to evaluate how long gas bubbles can stay in the sand. Major conclusions of this chapter are made here:

- In the photo imaging observation test, gas bubbles were found to be randomly distributed in the sample. With the decrease in the degree of saturation, there were more gas bubbles observed in the sample. In the tested range of degree of saturation (84.1% ~94.4%), gas bubbles were in the insular form (i.e.

discontinuous gas phase and continuous liquid phase).

- Computer tomography (CT) technique is able to observe the inner structure of materials. For saturated sand, the distribution of sand particles and pore voids were uniform. For desaturated sands with the degrees of saturation in a range of 88%~97%, there were some small single gas bubbles in the pore voids. There were also some relatively large pockets formed by some small bubbles aggregation, which indicated the formation of the metastable structure of unsaturated sand.
- The gas stability test was conducted under hydrostatic, upward flow, and downward flow conditions. Under hydrostatic condition, there was almost no change in the degree of saturation. Under both upward and downward flow conditions with a hydraulic gradient of 0.1, desaturated sand with a degree of saturation of around 89% could gradually become saturated in 3 to 5 days. The gas was taken away in the dissolved form. These results suggest that gas bubbles in sand are stable at hydrostatic condition, while water flow could affect the stability of gas bubbles in sand. Thus water flow should be controlled to minimize erosion of gas bubbles when the biogas technique is applied.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

A new technique of using biogas for the mitigation of soil liquefaction has been developed in this study. Microbial denitrification process is used for the production of N_2 gas bubbles in sand. The mechanical behaviour and liquefaction potential of biogas desaturated sand have been studied using triaxial tests. The effectiveness of the biogas method for the mitigation of sand liquefaction under cyclic load has been verified using a laminar box and shaking table testing system. The gas morphology and stability in sand are also experimentally studied. The following conclusions have been derived from this study.

- Batch experiments were conducted to investigate the favored conditions for the growth of denitrifying bacteria and the preferred conditions for the completion of the denitrification reaction. Three batches, with ethanol, sodium acetate, and methanol, as electron donors, respectively, were used. The reaction rates and growth rates with ethanol and sodium acetate were similar (nitrate reduction around 9 mmol/day and growth rate around 0.3 OD₆₀₀/day), both of which were considerably faster than the batch with methanol (nitrate reduction around 1.3 mmol/day and growth rate around 0.13 OD₆₀₀/day). Moreover, the batch with methanol showed a large amount of permanent nitrite accumulation; while the amount of nitrite accumulations with ethanol and sodium acetate was small and

temporary. These results suggest that the composition of soil desaturation solution should be properly selected to avoid the accumulation of nitrite.

- The gas generation process was studied using a small size sand column. The nitrate-N concentrations in the samples were varied from 125~374 mg/L. The reaction was completed in around 3 days and the reaction rates were similar for all the samples. At a nitrate concentration of 374 mg/L $\text{KNO}_3\text{-N}$, there were noticeable, but small percentage of residual nitrate and nitrite, while in a lower range of $\text{KNO}_3\text{-N}$ concentration (125~250 mg/L), residual nitrate and nitrite were minor. The lowest degree of saturation that did not cause nitrite accumulation was 82.7%, which was low enough to considerably mitigate the liquefaction potential. These results prove that the microbial denitrification process is effective for the purpose of generating nitrogen gas in sand.
- For loose sand in the undrained triaxial tests, the undrained shear strength became higher when the degree of saturation became lower. When the pore water pressure coefficient B-value decreased from about 0.95 (saturated state) to around 0.3, the undrained shear strength showed a 2.1-time increase in the compression tests and a 1.7-time increase in the extension tests. Furthermore, the stress strain response changed from complete strain softening to strain hardening with the reduction in the degree of saturation.
- Prefailure instability may occur for loose sand in the form of sudden increase in shear strain and loss of strength. For loose sand in both triaxial undrained compression and extension tests, the reduction in the degree of saturation also led to an increase in the slope of instability line η_{IL} when the void ratios were about the same. This result indicates that biogas desaturation is also an effective way to reduce the susceptibility to prefailure instability.

- For medium dense sand in triaxial undrained compression and extension tests, there is an increase in the yielding stress and a decrease in the maximum pore water pressure generation with the decrease in the degree of saturation. The undrained resistance of medium dense sand improves at small shear strain and reduces at large shear strain.
- A fully instrumented laminar box and shaking table testing system was designed to study the liquefaction responses of biogas desaturated sands. Test results showed that liquefaction occurred for saturated samples at loose states under $a_{max}=0.5 \text{ m/s}^2$ and at medium dense states under $a_{max}=1.5 \text{ m/s}^2$. Liquefaction did not occur for biogas desaturated samples with the degree of saturation in the range of 80% to 95% given the other conditions the same. To achieve a non-liquefaction response under $a_{max}=1.5 \text{ m/s}^2$ for saturated samples, the relative density of the sand has to be as high as around 90%. The occurrence of liquefaction is manifested by several observations simultaneously, including drastic pore water pressure build-up, excessive ground subsidence, sinking of structures resting on the soil surface, and large amplification of acceleration.
- The relationship between the pore pressure ratio and the volumetric strain can be represented by a single line (Figure 5.18). The volumetric strain rises at an accelerating rate with the increase of the pore pressure ratio. The relationship between the pore pressure ratio and the depth of the sinking of structure can also be shown by a single line (Figure 5.19). The sinking of structure becomes deeper as pore pressure ratio increases. In these two relationships, pore pressure ratio r_u of 0.4 seems to be a threshold. When $r_u < 0.4$, The volumetric strain is small and there is almost no sinking of structures, and when $r_u > 0.4$, both the volumetric strain and the sinking of structures starts rising violently. Such relationships can

be used for evaluating the ground and structure performance during liquefaction.

- A photographic system with a single lens reflex camera and micro lens was used for the observation of gas morphology in biogas sand. Gas bubbles were found to be randomly distributed in the sample. With the decrease in the degree of saturation, there were more gas bubbles observed in the sample. In the tested range of degree of saturation (84.1% ~94.4%), gas bubbles were in the insular form (i.e. discontinuous gas phase and continuous liquid phase).
- Computer tomography (CT) technique was adopted to observe the inner structure and gas distribution of the biogas desaturated sand samples. For saturated sand, the distribution of sand particles and pore voids were relatively uniform. For desaturated sands with the degrees of saturation in a range of 88%~97%, there were small single gas bubbles in the pore voids. However, there were also some pockets of gas/water conglomerates formed by aggregated bubbles in water.
- A sand column was used to evaluate the stability of gas bubbles in sand under both hydrostatic and flow conditions. Under hydrostatic condition, there was almost no change in the degree of saturation. Under both upward and downward flow conditions with a hydraulic gradient of 0.1, desaturated sand with a degree of saturation of around 89% could gradually become saturated in 3 to 5 days. The gas was taken away in the dissolved form. These results suggest that gas bubbles in sand are stable at hydrostatic condition, while water flow could affect the stability of gas bubbles in sand. Thus water flow should be controlled to minimize erosion of gas bubbles when the biogas technique is applied.

7.2 Recommendations

The following recommendations can be made for future studies:

- The study of the biogas method should be scaled up. Large-scale shaking table tests or in-situ tests can be carried out to investigate the effect of spatial variations of gas distribution and related soil behaviour.
- Instability is an important aspect of mechanical behaviour of soil. Load-controlled triaxial tests can be carried out to study the instability of sand. Such work can expand the scope of application of biogas technique.
- Effect of fines content is an important factor related to liquefaction of sand. Sand with fines content has not been tested in this thesis. Future experimental studies can take this factor into consideration.
- Wave velocity measurement can be used to detect gas content in sand. It is easy to conduct and the cost is low. It provides an effective way for the monitoring of biogas, especially in large scale tests. Future studies should employ this technique for different experiments and practical applications.
- The gas stability tests in Chapter 6 show that gases in sand can be taken away by water flow. As reviewed in Chapter 2, denitrification process can also be used as a biocement (van Paassen et al., 2010a). Therefore, a possible way is to use microbial denitrification process to achieve both objectives: to generate gases for the liquefaction mitigation and to form bioclogging for the control of water flow. This could be an interesting topic for future studies.

REFERENCES

Alarcon-Guzman, A., Leonards, G. A. and Chameau, J. L. (1988), "Undrained Monotonic and Cyclic Strength of Sands", Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 10, pp. 1089-1109.

Altun, S. and Goktepe, B. (2006), "Cyclic Stress-Strain Behavior of Partially Saturated Soils", Proceedings of the Fourth International Conference on Unsaturated Soils (GSP 147), pp. 497-507.

APHA (2005). Standard Methods for the Examination of Water and Wastewater, American Public Health Association, Washington, D.C., USA.

Arthur, J. R. F. and Menzies, B. K. (1972), "Inherent Anisotropy in a Sand", Geotechnique, Vol. 22, No. 1, pp. 115-128.

Barak, Y., Tal, Y. and van Rijn, J. (1998), "Light-Mediated Nitrite Accumulation During Denitrification by *Pseudomonas sp.* Strain jr12", Applied and Environmental Microbiology, Vol. 64, No. 3, pp. 813-817.

Baveye, P., Vandevivere, P., Hoyle, B. L., DeLeo, P. C. and de Lozada, D. S. (1998), "Environmental Impact and Mechanisms of the Biological Clogging of Saturated Soils and Aquifer Materials", Critical Reviews in Environmental Science and Technology, Vol. 28, No. 2, pp. 123-191.

Been, K. and Jefferies, M. G. (1985), "A State Parameter for Sands", Geotechnique, Vol. 35, No. 2, pp. 99-112.

Been, K., Jefferies, M. G. and Hachey, J. (1991), "The Critical State of Sands", Geotechnique, Vol. 41, No. 3, pp. 365-381.

Betlach, M. R. and Tiedje, J. M. (1981), "Kinetic Explanation for Accumulation of Nitrite, Nitric-Oxide, and Nitrous-Oxide During Bacterial Denitrification", Applied and Environmental Microbiology, Vol. 42, No. 6, pp. 1074-1084.

- Bishop, A. W. (1966), "The Strength of Soils as Engineering Materials", Geotechnique, Vol. 16, No. 2, pp. 91-130.
- Bishop, A. W. (1971), "Shear Strength Parameters for Undisturbed and Remoulded Soil Specimens", Roscoe Memorial Symposium, Cambridge, pp. 3-58.
- Bishop, A. W. (1973), "The Stability of Tips and Spoil Heaps", Quarterly Journal of Engineering Geology, Vol. 6, pp. 335-376.
- Bishop, A. W. and Henkel, D. J. (1962), The Measurement of Soil Properties in the Triaxial Test, Edward Arnold Ltd., London.
- Blaszczyk, M. (1993), "Effect of Medium Composition on the Denitrification of Nitrate by *Paracoccus denitrificans*", Applied and Environmental Microbiology, Vol. 59, No. 11, pp. 3951-3953.
- Blaszczyk, M., Galka, E., Sakowicz, E. and Mycielski, R. (1985), "Denitrification of High-Concentrations of Nitrites and Nitrates in Synthetic Medium with Different Sources of Organic-Carbon .3. Methanol", Acta Microbiologica Polonica, Vol. 34, No. 2, pp. 195-205.
- Bouferra, R., Benseddiq, N. and Shahrour, I. (2007), "Saturation and Preloading Effects on the Cyclic Behavior of Sand", International Journal of Geomechanics, Vol. 7, No. 5, pp. 396-401.
- Brooks, R. H. and Corey, A. T. (1964), "Hydraulic Properties of Porous Media", Colorado State University Hydrology Papers, Vol. 27, No. 3, pp. 1-15.
- Carlson, C. A. and Ingraham, J. L. (1983), "Comparison of Denitrification by *Pseudomonas stutzeri*, *Pseudomonas aeruginosa*, and *Paracoccus denitrificans*", Applied and Environmental Microbiology, Vol. 45, No. 4, pp. 1247-1253.
- Casagrande, A. (1975), "Liquefaction and Cyclic Deformation of Sands, a Critical Review", 5th Pan-American Conference on Soil Mechanics and Foundation Engineering, Buenos Aires, pp. 79-133.
- Castro, G. (1987). "On the Behaviour of Soil During Earthquakes - Liquefaction", Soil Dynamics and Liquefaction. Amsterdam, Elsevier.
- Catney, S. F. and Lynch, R. J. (2001), "Experimental Investigations of Air Sparging to Control Contaminated Groundwater", Proceedings of the Institution of Civil Engineers-Geotechnical Engineering, Vol. 149, No. 4, pp. 253-258.

- Chaney, R. (1978), "Saturation Effects on the Cyclic Strength of Sand", Proceeding of ASCE Special Conference on Earthquake Engineering and Soil Dynamics, New York, pp. 342-359.
- Chang, W. J., Rathje, E. M., Stokoe, K. H. and Hazirbaba, K. (2007), "In Situ Pore-Pressure Generation Behavior of Liquefiable Sand", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 133, No. 8, pp. 921-931.
- Chen, Y. C. and Liao, T. S. (1999), "Studies of the State Parameter and Liquefaction Resistance of Sands", Earthquake Geotechnical Engineering, Seco e Pinto (ed.), pp. 513-518.
- Cho, G. C., Dodds, J. and Santamarina, J. C. (2006), "Particle Shape Effects on Packing Density, Stiffness, and Strength: Natural and Crushed Sands", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 5, pp. 591-602.
- Christian, H. A. and Cranston, R. E. (1997), "A Methodology for Detecting Free Gas in Marine Sediments", Canadian Geotechnical Journal, Vol. 34, No. 2, pp. 293-304.
- Chu, J. (1995), "An Experimental Examination of the Critical State and Other Similar Concepts for Granular Soils", Canadian Geotechnical Journal, Vol. 32, No. 6, pp. 1065-1075.
- Chu, J. (2007), "What Is Post-Liquefaction Strength for Sand?", 13th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Kolkata.
- Chu, J., Ivanov, V., He, J., Naeimi, M., Li, B. and Stabnikov, V. (2011), "Development of Microbial Geotechnology in Singapore", Geo-Frontiers 2011: Advances in Geotechnical Engineering.
- Chu, J. and Leong, W. K. (2001), "Pre-Failure Strain Softening and Pre-Failure Instability of Sand: A Comparative Study", Geotechnique, Vol. 51, No. 4, pp. 311-321.
- Chu, J. and Leong, W. K. (2002), "Effect of Fines on Instability Behaviour of Loose Sand", Geotechnique, Vol. 52, No. 10, pp. 751-755.
- Chu, J., Leong, W. K., Loke, W. L. and Wanatowski, D. (2012a), "Instability of Loose Sand under Drained Conditions", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 138, No. 2, pp. 207-216.
- Chu, J., Leroueil, S. and Leong, W. K. (2003), "Unstable Behaviour of Sand and Its

- Implication for Slope Instability", Canadian Geotechnical Journal, Vol. 40, No. 5, pp. 873-885.
- Chu, J., Lo, S. C. R. and Lee, I. K. (1993), "Instability of Granular Soils under Strain Path Testing", Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 5, pp. 874-892.
- Chu, J., Stabnikov, V. and Ivanov, V. (2012b), "Microbially Induced Calcium Carbonate Precipitation on Surface or in the Bulk of Soil", Geomicrobiology Journal, Vol. 29, No. 6, pp. 544-549.
- Chu, J., Varaksin, S., Klotz, U. and Menge, P. (2009), "State of the Art Report: Construction Process", 17th International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt.
- Chu, J. and Wanatowski, D. (2008), "Instability Conditions of Loose Sand in Plane-strain", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 134, No. 1, pp. 136-142.
- Chu, J. and Wanatowski, D. (2009), "Effect of Loading Mode on Strain Softening and Instability Behavior of Sand in Plane-strain Tests", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 135, No. 1, pp. 108-120.
- Cornforth, D. H. (1964), "Some Experiments on the Effect of Strain Condition on The Strength of Sand", Geotechnique, Vol. 14, No., pp. 143-167.
- Cuhel, J., Simek, M., Laughlin, R. J., Bru, D., Cheneby, D., Watson, C. J. and Philippot, L. (2010), "Insights into the Effect of Soil pH on N(2)O and N(2) Emissions and Denitrifier Community Size and Activity", Applied and Environmental Microbiology, Vol. 76, No. 6, pp. 1870-1878.
- De Muynck, W., De Belie, N. and Verstraete, W. (2010a), "Microbial Carbonate Precipitation in Construction Materials: A Review", Ecological Engineering, Vol. 36, No. 2, pp. 118-136.
- De Muynck, W., Verbeken, K., De Belie, N. and Verstraete, W. (2010b), "Influence of Urea and Calcium Dosage on the Effectiveness of Bacterially Induced Carbonate Precipitation on Limestone", Ecological Engineering, Vol. 36, No. 2, pp. 99-111.
- Dealba, P. A., Seed, H. B., Retamal, E. and Seed, R. B. (1988), "Analyses of Dam Failures in 1985 Chilean Earthquake", Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 12, pp. 1414-1434.

- DeJong, J. T., Fritzges, M. B. and Nusslein, K. (2006), "Microbially Induced Cementation to Control Sand Response to Undrained Shear", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 11, pp. 1381-1392.
- DeJong, J. T., Martinez, B. C., Martinez, B. C., Nelson, D. C., Waller, J. T., Weil, T. R., Ginn, T. R., Weather, T., Barkouki, T., Fujita, Y., Redden, G., Hunt, C., Major, D. and Yanyu, B. (2009), "Upscaling of Bio-Mediated Soil Improvement", The 17th International Conference on Soil Mechanics and Geotechnical Engineering, pp. 2300-2303.
- DeJong, J. T., Mortensen, B. M., Martinez, B. C. and Nelson, D. C. (2010), "Bio-Mediated Soil Improvement", Ecological Engineering, Vol. 36, No. 2, pp. 197-210.
- DeJong, J. T., Soga, K., Banwart, S. A., Whalley, W. R., Ginn, T. R., Nelson, D. C., Mortensen, B. M., Martinez, B. C. and Barkouki, T. (2011), "Soil Engineering in Vivo: Harnessing Natural Biogeochemical Systems for Sustainable, Multi-Functional Engineering Solutions", Journal of the Royal Society Interface, Vol. 8, No. 54, pp. 1-15.
- Dobry, R., Ladd, R. S., Yokel, F. Y., Chung, R. M. and Powell, D. (1982), "Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method", NBS building science series 138, National Bureau of Standards.
- Dobry, R., Thevanayagam, S., Medina, C., Bethapudi, R., Elgamal, A., Bennett, V., Abdoun, T., Zeghal, M., El Shamy, U. and Mercado, V. M. (2011), "Mechanics of Lateral Spreading Observed in a Full-Scale Shake Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 137, No. 2, pp. 115-129.
- Dobry, R., Vasquez-Herrera, A., Mohamad, R. and Vucetic., M. (1985), "Liquefaction Flow Failure of Silty Sand by Torsional Cyclic Tests", Advances in the Art of Testing Soils Under Cyclic Conditions, Detroit, Mich., pp. 29-50.
- Elgamal, A., Yang, Z. H., Lai, T., Kutter, B. L. and Wilson, D. W. (2005), "Dynamic Response of Saturated Dense Sand in Laminated Centrifuge Container", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 131, No. 5, pp. 598-609.
- Eseller-Bayat, E. (2009), "Seismic Response and Prevention of Liquefaction Failure of Sands Partially Saturated through Introduction of Gas Bubbles", Ph.D. Thesis, Northeastern University.

- Eseller-Bayat, E., Gokyer, S. and Yegian, M. K., (2012), "Liquefaction Response of Partially Saturated Sands: Experimental Results", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Preview Manuscript, (doi: [http://dx.doi.org/10.1061/\(ASCE\)GT.1943-5606.0000815](http://dx.doi.org/10.1061/(ASCE)GT.1943-5606.0000815)).
- Eseller-Bayat, E., Yegian, M. K., Alshawabkeh, A. and Gokyer, S. (2009), "A New Mitigation Technique for Preventing Liquefaction Induced Building Damages During Earthquakes", Earthquake & Tsunami WCCE-ECCE –TCCE Joint Conference, Istanbul, Turkey.
- Finno, R. J. and Rechenmacher, A. L. (2003), "Effects of Consolidation History on Critical State of Sand", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 129, No. 4, pp. 350-360.
- Fischer, E. N. and Whalen, S. C. (2005), "Rates and Controls on Denitrification in an Agricultural Soil Fertilized with Liquid Lagoonal Swine Waste", Nutrient Cycling in Agroecosystems, Vol. 71, No. 3, pp. 271-287.
- Fredlund, D. G. and Rahardjo, H. (1993). "Soil Mechanics for Unsaturated Soils". New York, Wiley.
- Glass, C. and Silverstein, J. (1998), "Denitrification Kinetics of High Nitrate Concentration Water: pH Effect on Inhibition and Nitrite Accumulation", Water Research, Vol. 32, No. 3, pp. 831-839.
- Gokyer, S. (2009), "Inducing and Imaging Partial Degree of Saturation in Laboratory Sand Specimens", Master Thesis, Northeastern University.
- Grozic, J. L., Robertson, P. K. and Morgenstern, N. R. (1999), "The Behavior of Loose Gassy Sand", Canadian Geotechnical Journal, Vol. 36, No. 3, pp. 482-492.
- Harkes, M. P., van Paassen, L. A., Booster, J. L., Whiffin, V. S. and van Loosdrecht, M. C. M. (2010), "Fixation and Distribution of Bacterial Activity in Sand to Induce Carbonate Precipitation for Ground Reinforcement", Ecological Engineering, Vol. 36, No. 2, pp. 112-117.
- He, J., Chu, J., Ivanov, V. (2013), "Mitigation of Liquefaction of Saturated Sand Using Biogas", Geotechnique, Vol. 63, No. 4, pp. 267-275.
- Head, K. H. And Epps, R. (2011), "Manual of Soil Laboratory Testing, Volume 2, 3rd Edition", Whittles Publishing.
- Hird, C. C. and Hassona, F. A. K. (1990), "Some Factors Affecting the Liquefaction

and Flow of Saturated Sands in Laboratory Tests", Engineering Geology, Vol. 28, No. 1-2, pp. 149-170.

Huang, Y., Tsuchiya, C. and Ishihara, K. (1999), "Estimation of Partial Saturation Effect on Liquefaction Strength of Unsaturated Sandy Soil", JGS Symposium.

Idriss, I. M. (1991), "Earthquake Ground Motions at Soft Soil Sites", Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, pp. 2265-2272.

Ishihara, K. (1993), "Liquefaction and Flow Failure During Earthquakes", Geotechnique, Vol. 43, No. 3, pp. 351-415.

Ishihara, K., Cubrinovski, M. and Novaka, T. (1998), "Characterization of Undrained Behaviour of Soils in the Reclaimed Area of Kobe", Special Issue of Soils and Foundations, pp. 33-46.

Ishihara, K., Tatsuoka, F. and Yasuda, S. (1975), "Undrained Deformation and Liquefaction of Sand under Cyclic Stresses", Soils and Foundations, Vol. 15, No. 1, pp. 29-44.

Ishihara, K., Tsuchiya, H., Huang, Y. and Kamada, K. (2001), "Recent Studies on Liquefaction Resistance of Sand: Effect of Saturation", ASCE Special Conference on Earthquake Engineering and Soil Dynamics, New York, pp. 342-359.

Ivanov, V. and Chu, J. (2008), "Applications of Microorganisms to Geotechnical Engineering for Bioclogging and Biocementation of Soil in Situ", Reviews in Environmental Science and Biotechnology, Vol. 7, No. 2, pp. 139-153.

Japanese Geotechnical Society, the (1998), Remedial Measures against Soil Liquefaction : From Investigation and Design to Implementation, A.A. Balkema, Rotterdam.

Japanese Port and Harbour Research Institute (1997), Handbook on Liquefaction Remediation of Reclaimed Land, A.A. Balkema, Rotterdam.

Jefferies, M. G. and Been, K. (2000), "Implications for Critical State Theory from Isotropic Compression of Sand", Geotechnique, Vol. 50, No. 4, pp. 419-429.

Jefferies, M. G. and Been, K. (2006). Soil Liquefaction : A Critical State Approach, Taylor & Francis, London.

Jefferies, M. G. and Shuttle, D. A. (2002), "Dilatancy in General Cambridge-Type

Models", Geotechnique, Vol. 52, No. 9, pp. 625-638.

Kagawa, T., Sato, M., Minowa, C., Abe, A. and Tazoh, T. (2004), "Centrifuge Simulations of Large-Scale Shaking Table Tests: Case Studies", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 7, pp. 663-672.

Kester, R. A., deBoer, W. and Laanbroek, H. J. (1997), "Production of NO and N₂O by Pure Cultures of Nitrifying and Denitrifying Bacteria During Changes in Aeration", Applied and Environmental Microbiology, Vol. 63, No. 10, pp. 3872-3877.

Kokusho, T., Hara, T. and Hiraoka, R. (2004), "Undrained Shear Strength of Granular Soils with Different Particle Gradations", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 6, pp. 621-629.

Konrad, J. M. (1990a), "Minimum Undrained Strength of Two Sands", Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 6, pp. 932-947.

Konrad, J. M. (1990b), "Minimum Undrained Strength Versus Steady State Strength of Sands", Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 6, pp. 948-963.

Konrad, J. M. (1993), "Undrained Response of Loosely Compacted Sands During Monotonic and Cyclic Compression Tests", Geotechnique, Vol. 43, No. 1, pp. 69-89.

Kuerbis, R. H. and Vaid, Y. P. (1988), "Sand Sample Preparation - the Slurry Deposition Method", Soils and Foundations, Vol. 28, No. 4, pp. 107-118.

Lade, P. V. (1993), "Initiation of Static Instability in the Submarine Nerlerk Berm", Canadian Geotechnical Journal, Vol. 30, No. 6, pp. 895-904.

Lade, P. V. and Hernandez, S. B. (1977), "Membrane Penetration Effects in Undrained Tests", Journal of Geotechnical Engineering Division, ASCE, Vol. 116, No. 11, pp. 2532-2550.

Lade, P. V., Nelson, R. B. and Ito, Y. M. (1987), "Nonassociated Flow and Stability of Granular Materials", Journal of Engineering Mechanics, ASCE, Vol. 113, No. 9, pp. 1302-1308.

Lade, P. V., Nelson, R. B. and Ito, Y. M. (1988), "Instability of Granular-Materials with Nonassociated Flow", Journal of Engineering Mechanics, ASCE, Vol. 114, No. 12, pp. 2173-2191.

- Lade, P. V. and Pradel, D. (1990), "Instability and Plastic Flows. I: Experimental Observations.", Journal of Engineering Mechanics, ASCE, Vol. 116, No. 11, pp. 2532-2550.
- Lade, P. V. and Yamamuro, J. A. (1996), "Undrained Sand Behavior in Axisymmetric Tests at High Pressures", Journal of Geotechnical Engineering, ASCE, Vol. 122, No. 2, pp. 120-129.
- Lane, D. J. (1991), "Nucleic Acid Techniques in Bacterial Systematics", Stackebrandt, E. & Goodfellow, M. (eds). Chichester: Wiley and Sons, pp.115–175.
- Lee, J. S. and Santamarina, J. C. (2007), "Seismic Monitoring Short-Duration Events: Liquefaction in 1g Models", Canadian Geotechnical Journal, Vol. 44, No. 6, pp. 659-672.
- Lee, K. L. and Albaisa, A. (1974), "Earthquake Induced Settlements in Saturated Sands", Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. 4, pp. 387-406.
- Ling, H. I., Mohri, Y., Kawabata, T., Liu, H., Burke, C. and Sun, L. (2003), "Centrifugal Modeling of Seismic Behavior of Large-Diameter Pipe in Liquefiable Soil", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 129, No. 12, pp. 1092-1101.
- Lu, W. J., Qian, C. X. and Wang, R. X. (2010), "Study on Soil Solidification Based on Microbiological Precipitation of CaCO_3 ", Science China-Technological Sciences, Vol. 53, No. 9, pp. 2372-2377.
- Marcuson, I., W. F., Hynes, M. E. and Franklin, A. G. (1990), "Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments", Earthquake Spectra, Vol. 6, No. 3, pp. 529-572.
- Martin, G. R., Seed, H. B. and Finn, D. L. (1975), "Fundamentals of Liquefaction under Cyclic Loading", Journal of Geotechnical Engineering, ASCE, Vol. 101, No. 5, pp. 423-438.
- Matyas, E. L. (1967), "Air and Water Peameability of Compacted Soils", ASTM STP 417, American Society of Testing and Materials.
- Miura, S., Toki, S. and Tanizawa, S. (1984), "Cone Penetration Characteristics and Its Correction to Static and Cyclic Deformation-Strength Behaviours of Anisotropic Sand", Soils and Foundations, Vol. 24, No. 2, pp. 58-74.

- Murthy, T. G., Loukidis, D., Carraroy, J. A. H., Prezzi, M. and Salgado, R. (2007), "Undrained Monotonic Response of Clean and Silty Sands", Geotechnique, Vol. 57, No. 3, pp. 273-288.
- Negussey, D. and Islam, M. S. (1994), "Uniqueness of Steady-State and Liquefaction Potential", Canadian Geotechnical Journal, Vol. 31, No. 1, pp. 132-139.
- Ng, C. W. W. N., Fung, W. T., Cheuk, C. Y. and Zhang, L. M. (2004), "Influence of Stress Ratio and Stress Path on Behavior of Loose Decomposed Granite", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 1, pp. 36-44.
- Oda, M. (1972), "Initial Fabrics and Their Relations to Mechanical Properties of Granular Materials", Soils and Foundations, Vol. 12, No. 1, pp. 17-36.
- Oda, M., Konishi, J. and Nemat-Nasser, S. (1980), "Some Experimentally Based Fundamental Results on the Mechanical-Behavior of Granular-Materials", Geotechnique, Vol. 30, No. 4, pp. 479-495.
- Okamura, M., Ishihara, M. and Oshita, T. (2003), "Liquefaction Resistance of Sand Deposit Improved with Sand Compaction Piles", Soils and Foundations, Vol. 43, No. 5, pp. 175-187.
- Okamura, M., Ishihara, M. and Tamura, K. (2006), "Degree of Saturation and Liquefaction Resistances of Sand Improved with Sand Compaction Pile", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 2, pp. 258-264.
- Okamura, M. and Soga, Y. (2006), "Effects of Pore Fluid Compressibility on Liquefaction Resistance of Partially Saturated Sand", Soils and Foundations, Vol. 46, No. 5, pp. 695-700.
- Okamura, M., Takebayashi, M., Nishida, K., Fujii, N., Jinguji, M., Imasato, T., Yasuhara, H. and Nakagawa, E. (2011), "In-Situ Desaturation Test by Air Injection and Its Evaluation through Field Monitoring and Multiphase Flow Simulation", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 137, No. 7, pp. 643-652.
- Okamura, M. and Teraoka, T. (2006), "Shaking Table Tests to Investigate Soil Desaturation as a Liquefaction Countermeasure", Seismic Performance and Simulation of Pile Foundations in Liquefied and Laterally Spreading Ground (GSP 145), pp. 282-293.
- Okwadha, G. D. O. and Li, J. (2010), "Optimum Conditions for Microbial Carbonate

- Precipitation", Chemosphere, Vol. 81, No. 9, pp. 1143-1148.
- Parkin, T. B., Sexstone, A. J. and Tiedje, J. M. (1985), "Adaptation of Denitrifying Populations to Low Soil-pH", Applied and Environmental Microbiology, Vol. 49, No. 5, pp. 1053-1056.
- Pietruszczak, S., Pande, G. N. and Oulapour, M. (2003), "A Hypothesis for Mitigation of Risk of Liquefaction", Geotechnique, Vol. 53, No. 9, pp. 833-838.
- Pitman, T. D., Robertson, P. K. and Sego, D. C. (1994), "Influence of Fines on the Collapse of Loose Sands", Canadian Geotechnical Journal, Vol. 31, No. 5, pp. 728-739.
- Poulos, S. J. (1981), "The Steady-State of Deformation", Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. 5, pp. 553-562.
- Poulos, S. J., Castro, G. and France, J. W. (1988), "Liquefaction Evaluation Procedure - Closure", Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 2, pp. 251-259.
- Pradel, D. and Lade, P. V. (1990), "Instability and Plastic-Flow of Soils .2. Analytical Investigation", Journal of Engineering Mechanics, ASCE, Vol. 116, No. 11, pp. 2551-2566.
- Prasad, S. K., Towhata, I., Chandradhara, G. P. and Nanjundaswamy, P. (2004), "Shaking Table Tests in Earthquake Geotechnical Engineering", Current science, Vol. 87, No. 10, pp. 1398-1404.
- Rad, N. S. and Lunne, T. (1994), "Gas in Soil .1. Detection and Tau-Profiling", Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 4, pp. 697-717.
- Rad, N. S., Vianna, A. J. D. and Berre, T. (1994), "Gas in Soils .2. Effect of Gas on Undrained Static and Cyclic Strength of Sand", Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 4, pp. 716-736.
- Rebata-Landa, V. and Santamarina, J. C. (2012), "Mechanical Effects of Biogenic Nitrogen Gas Bubbles in Soils", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 138, No. 2, pp. 128-137.
- Saleh-Lakha, S., Shannon, K. E., Henderson, S. L., Goyer, C., Trevors, J. T., Zebarth, B. J. and Burton, D. L. (2009), "Effect of pH and Temperature on Denitrification Gene Expression and Activity in *Pseudomonas mandelii*", Applied and Environmental Microbiology, Vol. 75, No. 12, pp. 3903-3911.

- Sandven, R., Husby, E., Husby, J. E., Jonland, J., Roksvag, K. O., Staehli, F. and Tellugen, R. (2007), "Development of a Sampler for Measurement of Gas Content in Soils", Journal of Waterway Port Coastal and Ocean Engineering, ASCE, Vol. 133, No. 1, pp. 3-13.
- Santamarina, J. C. and Cho, G. C. (2004), "Soil Behaviour: The Role of Particle Shape", Proc. Skempton Conf. London.
- Schofield, A. and Wroth, C. P. (1968). "Critical State Soil Mechanics", McGraw-Hill, London.
- Seed, H. B. and Idriss, I. M. (1971), "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. 9, pp. 1249-1273.
- Seed, H. B., Martin, P. P. and Lysmer, J. (1975), "The Generation and Dissipation of Pore Pressure During Soil Liquefaction", UCB/EERC-75/26, Earthquake Engineering Research Center, University of California.
- Shapiro, S. and Yamamuro, J. A. (2003), "Effects of Silt on Three-Dimensional Stress-Strain Behavior of Loose Sand", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 129, No. 1, pp. 1-11.
- Sharp, M. K., Dobry, R. and Phillips, R. (2010), "CPT-Based Evaluation of Liquefaction and Lateral Spreading in Centrifuge", Journal of Geotechnical & Geoenvironmental Engineering, ASCE, Vol. 136, No. 10, pp. 1334-1346.
- Sherif, M. A., Tsuchiya, C. and Ishibashi, I. (1977), "Saturation Effect on Initial Soil Liquefaction", Journal of Geotechnical Engineering, ASCE, Vol. 103, No. 8, pp. 914-917.
- Simek, M., Jisova, L. and Hopkins, D. W. (2002), "What Is the So-Called Optimum pH for Denitrification in Soil?", Soil Biology & Biochemistry, Vol. 34, No. 9, pp. 1227-1234.
- Skempton, A. W. (1954), "The Pore Pressure Coefficient A and B", Geotechnique, Vol. 4, No. 4, pp. 143-147.
- Sladen, J. A., Dhollander, R. D. and Krahn, J. (1985), "The Liquefaction of Sands, a Collapse Surface Approach", Canadian Geotechnical Journal, Vol. 22, No. 4, pp. 564-578.
- Sobkowicz, J. C. (1984), "The Undrained Equilibrium Behaviour of Gassy

- Sediments", Canadian Geotechnical Journal, Vol. 21, No., pp. 439-448.
- Stabnikov, V., Naeimi, M., Ivanov, V. and Chu, J. (2011), "Formation of Water-Impermeable Crust on Sand Surface Using Biocement", Cement and Concrete Research, Vol. 41, No. 11, pp. 1143-1149.
- Stanford, G., Dzienia, S. and Vanderpol, R. A. (1975), "Effect of Temperature on Denitrification Rate in Soils", Soil Science Society of America Journal, Vol. 39, No. 5, pp. 867-870.
- Tamura, S., Tokimatsu, K., Abe, A. and Sato, M. (2002), "Effects of Air Bubbles on B-Value and P-Wave Velocity of a Partly Saturated Sand", Soils and Foundations, Vol. 42, No. 1, pp. 121-129.
- Thevanayagam, S. (1998), "Effect of Fines and Confining Stress on Undrained Shear Strength of Silty Sands", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 6, pp. 479-491.
- Thevanayagam, S., Kanagalingam, T., Reinhorn, A., Tharmendhira, R., Dobry, R., Pitman, M., Abdoun, T., Elgamal, A., Zeghal, M., Ecemis, N. and El Shamy, U. (2009), "Laminar Box System for 1-g Physical Modeling of Liquefaction and Lateral Spreading", Geotechnical Testing Journal, Vol. 32, No. 5, pp. 1-12.
- Tokimatsu, K. and Seed, H. B. (1987), "Evaluation of Settlements in Sands Due to Earthquake Shaking", Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.
- Topp, G. C., Davis, J. L. and Annan, A. P. (1980), "Electromagnetic Determination of Soil Water Content: Measurements in Coaxial Triaxial Lines", Water Resources Research, Vol. 16, No. 3, pp. 574-582.
- Tsukamoto, Y., Ishihara, K., Nakazawa, H., Kamada, K. and Huang, Y. N. (2002), "Resistance of Partly Saturated Sand to Liquefaction with Reference to Longitudinal and Shear Wave Velocities", Soils and Foundations, Vol. 42, No. 6, pp. 93-104.
- Tsukamoto, Y., Ishihara, K. and Nonaka, K. (1998), "Undrained Deformation and Strength Characteristics of Soils from Reclaimed Deposits in Kobe", Special Issue of Soils and Foundations, Vol., No., pp. 47-55.
- Ueng, T. S. (2010), "Shaking Table Test for Studies of Soil Liquefaction and Soil-Pile Interaction", Geotechnical Engineering Journal of the SEAGS & AGSSEA, Vol. 41, No. 1, pp. 1-10.

- Ueng, T. S., Wang, M. H., Chen, M. H., Chen, C. H. and Peng, L. H. (2006), "A Large Biaxial Shear Box for Shaking Table Test on Saturated Sand", Geotechnical Testing Journal, Vol. 29, No. 1, pp. 1-8.
- Vaid, Y. P., Sayao, A., Hou, E. H. and Negussey, D. (1990), "Generalized Stress-Path-Dependent Soil Behavior with a New Hollow Cylinder Torsional Apparatus", Canadian Geotechnical Journal, Vol. 27, No. 5, pp. 601-616.
- Vaid, Y. P. and Thomas, J. (1995), "Liquefaction and Postliquefaction Behavior of Sand", Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 2, pp. 163-173.
- Van der Ruyt, M. and van der Zon, W. (2009), "Biological in Situ Reinforcement of Sand in near-Shore Areas", Proceedings of the Institution of Civil Engineers-Geotechnical Engineering, Vol. 162, No. 1, pp. 81-83.
- van Paassen, L. A., Daza, C. M., Staal, M., Sorokin, D. Y., van der Zon, W. and van Loosdrecht, M. C. M. (2010a), "Potential Soil Reinforcement by Biological Denitrification", Ecological Engineering, Vol. 36, No. 2, pp. 168-175.
- van Paassen, L. A., Ghose, R., van der Linden, T. J. M., van der Star, W. R. L. and van Loosdrecht, M. C. M. (2010b), "Quantifying Biomediated Ground Improvement by Ureolysis: Large-Scale Biogrout Experiment", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 136, No. 12, pp. 1721-1728.
- van Paassen, L. A., Harkes, M. P., van Zwieten, G. A., van der zon, W. H., van der Star, W. R. L. and van Loosdrecht, M. C. M. (2009), "Scale up of Biogrout: A Biological Ground Reinforcement Method", The 17th International Conference on Soil Mechanics and Geotechnical Engineering, pp. 2328-2333.
- Verdugo, R. and Ishihara, K. (1996), "The Steady State of Sandy Soils", Soils and Foundations, Vol. 36, No. 2, pp. 81-91.
- Wan, R. G. and Guo, P. J. (2004), "Stress Dilatancy and Fabric Dependencies on Sand Behavior", Journal of Engineering Mechanics, ASCE, Vol. 130, No. 6, pp. 635-645.
- Wanatowski, D. (2005), "Strain Softening and Instability of Sand under Plane-Strain Conditions", Ph.D. Thesis, Nanyang Technological University.
- Wang, Z. L., Dafalias, Y. F., Li, X. S. and Makdisi, F. I. (2002), "State Pressure Index for Modeling Sand Behavior", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 128, No. 6, pp. 511-519.
- Weaver, T. J., Burbank, M., Lewis, A., Lewis, R., Crawford, R. and Williams, B.

- (2011), "Bio-Induced Calcite, Iron, and Manganese Precipitation for Geotechnical Engineering Applications", Geo-Frontiers 2011: Advances in Geotechnical Engineering.
- Whiffin, V. S., van Paassen, L. A. and Harkes, M. P. (2007), "Microbial Carbonate Precipitation as a Soil Improvement Technique", Geomicrobiology Journal, Vol. 24, No. 5, pp. 417-423.
- Wroth, C. P. (1984), "Twenty Fourth Rankine Lecture: The Interpretation of in Situ Soil Tests", Geotechnique, Vol. 36, No. 5, pp. 907-933.
- Xia, H. and Hu, T. (1991), "Effects of Saturation and Back Pressure on Sand Liquefaction", Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 9, pp. 1347-1362.
- Yamamuro, J. A. and Covert, K. M. (2001), "Monotonic and Cyclic Liquefaction of Very Loose Sands with High Silt Content", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 4, pp. 314-324.
- Yamamuro, J. A. and Lade, P. V. (1996), "Drained Sand Behavior in Axisymmetric Tests at High Pressures", Journal of Geotechnical Engineering, ASCE, Vol. 122, No. 2, pp. 109-119.
- Yamamuro, J. A. and Lade, P. V. (1997a), "Instability of Granular Materials at High Pressures", Soils and Foundations, Vol. 37, No. 1, pp. 41-52.
- Yamamuro, J. A. and Lade, P. V. (1997b), "Static Liquefaction of Very Loose Sands", Canadian Geotechnical Journal, Vol. 34, No. 6, pp. 905-917.
- Yamamuro, J. A. and Lade, P. V. (1998), "Steady-State Concepts and Static Liquefaction of Silty Sands", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 9, pp. 868-877.
- Yang, J. (2002), "Liquefaction Resistance of Sand in Relation to P-Wave Velocity", Geotechnique, Vol. 52, No. 4, pp. 295-298.
- Yang, J. and Sato, T. (2001), "Analytical Study of Saturation Effects on Seismic Vertical Amplification of a Soil Layer", Geotechnique, Vol. 51, No. 2, pp. 161-165.
- Yang, J., Savidis, S. and Roemer, M. (2004), "Evaluating Liquefaction Strength of Partially Saturated Sand", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 9, pp. 975-979.

- Yasuda, S., Kobayashi, T., Fukushima, Y., Kohari, M. and Simazaki, T. (1999), "Effect of Degree of Saturation on the Liquefaction Strength of Masa", 34th Japanese National Conference of Geotechnical Engineering, pp. 2071-2072.
- Yegian, M. K., Eseller-Bayat, E., Alshawabkeh, A. and Ali, S. (2007), "Induced-Partial Saturation for Liquefaction Mitigation: Experimental Investigation", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 133, No. 4, pp. 372-380.
- Yegian, M. K., Eseller, E. and Alshawabkeh, A. (2006), "Preparation and Cyclic Testing of Partially Saturated Sands", Unsaturated Soils 2006 (GSP 147), pp. 508-518.
- Yoshimi, Y., Yanaka, K. and Tokimatsu, K. (1989), "Liquefaction Resistance of Partially Saturated Sand", Soils and Foundations Vol. 29, No. 2, pp. 157-162.
- Yoshimine, M. and Ishihara, K. (1998), "Flow Potential of Sand During Liquefaction", Soils and Foundations, Vol. 38, No. 3, pp. 189-198.
- Yoshimine, M., Ishihara, K. and Vargas, W. (1998), "Effects of Principal Stress Direction and Intermediate Principal Stress on Undrained Shear Behavior of Sand", Soils and Foundations, Vol. 38, No. 3, pp. 179-188.
- Yoshimine, M., Robertson, P. K. and Wride, C. E. (1999), "Undrained Shear Strength of Clean Sands to Trigger Flow Liquefaction", Canadian Geotechnical Journal, Vol. 36, No. 5, pp. 891-906.
- Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B. and Stokoe, K. H. (2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 Nceer and 1998 Nceer/Nsf Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 10, pp. 817-833.