ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY OF FINE GRAINED SOILS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

MENZER PEHLİVAN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

JULY 2009

Approval of the thesis:

ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY OF FINE GRAINED SOILS

submitted by **MENZER PEHLİVAN** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Canan Özgen _____________________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Güney Özcebe _____________________
Head of Department, **Civil Engineering**

Assoc. Prof Dr. Kemal Önder Çetin _____________________
Supervisor, **Civil Engineering Dept., METU**

Examining Committee Members:

Prof. Dr. M. Yener Özkan _____________________
Civil Engineering Dept., METU

Assoc. Prof. Dr. Kemal Önder Çetin _____________________
Civil Engineering Dept., METU

Inst. Dr. Berna Unutmaz _____________________
Civil Engineering Dept., Kocaeli University.

Prof. Dr. Ufuk Ergun _____________________
Civil Engineering Dept., METU

Inst. Dr. N. Kartal Toker _____________________
Civil Engineering Dept., METU

Date: _____________________

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name : MENZER PEHLİVAN

Signature :

ABSTRACT

ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY OF FINE GRAINED SOILS

Pehlivan, Menzer

M.S., Department of Civil Engineering

Supervisor: Assoc. Prof. Dr. K. Önder Çetin

July 2009, 167 pages

Recent ground failure case histories after 1994 Northridge, 1999 Kocaeli and 1999 Chi-Chi earthquakes revealed that low-plasticity silt-clay mixtures generate significant cyclic pore pressures and can exhibit a strain-softening response, which may cause significant damage to overlying structural systems. These observations accelerated research studies on liquefaction susceptibility of fine-grained soils. Alternative approaches to Chinese Criteria were proposed by several researchers (Seed et al. 2003, Bray and Sancio 2006, Boulanger and Idriss 2006) most of which assess liquefaction triggering potential based on cyclic test results compared on the basis of index properties of soils (such as LL, PI, LI, w_c/LL). Although these new methodologies are judged to be major improvements over Chinese Criteria, still there exist unclear issues regarding if and how reliably these methods can be used for the assessment of liquefaction triggering potential of fine grained soils. In this study, results of cyclic tests performed on undisturbed specimens

(ML, CL, MH and CH) were used to study cyclic shear strain and excess pore water pressure generation response of fine-grained soils. Based on comparisons with the cyclic response of saturated clean sands, a shift in pore pressure ratio (r_u) vs. shear strain response is observed, which is identified to be a function of PI, LL and (w_c/LL). Within the confines of this study, i) probabilistically based boundary curves identifying liquefaction triggering potential in the r_u vs. shear strain domain were proposed as a function of PI, LL and (w_c/LL), ii) these boundaries were then mapped on to the normalized net tip resistance ($q_{t,1,net}$) vs. friction ratio (F_R) domain, consistent with the work of Cetin and Ozan (2009). The proposed framework enabled both Atterberg limits and CPT based assessment of liquefaction triggering potential of fine grained low plasticity soils, differentiating clearly both cyclic mobility and liquefaction responses.

Keywords: Liquefaction Susceptibility, CPT, Fine-grained Soils, Cyclic Tests.

ÖZ

İNCE DANELİ MALZEMELERİN SIVILAŞABİLİRLİĞİNİN DEĞERLENDİRİLMESİ

Pehlivan, Menzer

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Doç. Dr. K. Önder Çetin

Temmuz 2009, 167 sayfa

1994 Northridge, 1999 Kocaeli ve 1999 Chi-Chi depremleri sonrasında oluşan zemin yenilmeleri düşük plastisiteli silt-kil karşımlarının üzerlerinde bulunan yapıya ciddi hasar verebilecek miktarda boşluk suyu basıncı üretebildiğini ve bu karışımların birim deformasyon altında yumuşama davranışı sergilediğini göstermiştir. Bu gözlemler ince daneli malzemelerin sıvılaşabilirliği üzerine yapılan çalışmalara hız kazandırmıştır. Çin kriterlerine alternatif olarak bazı araştırmacılar tarafından (Seed vd. 2003, Bray ve Sancio 2006, Boulanger ve Idriss 2006) önerilen yeni yaklaşımların büyük bir kısmı zeminin sıvılaşabilme potansiyelinin zeminin dinamik deney sonuçlarının indeks özellikleri (PI, LL, LI, w_c/LL gibi) kapsamında karşılaştırılmasına dayandırılmıştır. Söz konusu olan yeni yöntemler Çin kriterlerinden sonra büyük gelişmeler olarak kabul edilmiş olsalar dahi, bu yöntemlerin ince daneli malzemelerin sıvılaşabilirlik potansiyellerinin belirlenmesi amacıyla kullanımları ve güvenilirlikleri konusunda bazı belirsizlikler bulunmaktadır. Bu çalışmada, örselenmemiş numuneler (ML, CL, MH ve CH) üzerinde yapılan dinamik deney sonuçlarından faydalanılarak ince daneli malzemelerin devirsel birim deformasyon ve aşırı boşluk suyu basıncı oluşum tepkileri araştırılmıştır. Doymuş temiz kumların dinamik tepkileriyle yapılan karşılaştırmalara dayanarak, aşırı boşluk suyu basıncı oranına (r_u) karşılık oluşan birim kesme deformasyonlarının PI, LL ve (w_c/LL) parametrelerinin bir fonksiyonu olduğu belirlenmiştir. Bu çalışma kapsamında, i) sıvılaşma tetiklenme potansiyeli olasılıksal limit eğrileri aşırı boşluk suyu basıncına karşılık birim kesme deformasyonu alanında PI, LL ve (w_c/LL)'nin bir fonksiyonu olarak tanımlanmıştır, ii) bu eğriler daha sonra normalize edilmiş net uç direncine ($q_{t,1,net}$) karşılık sürtünme oranı (F_R) alanına, Cetin ve Ozan (2009) tarafından sunulan çalışmayla tutarlı olarak aktarılmıştır. Bu öneri kayma birim deformasyon birikimi ve zemin sıvılaşmasını birbirinden net olarak ayırarak, düşük plastisiteli ince daneli malzemelerin sıvılaşma tetiklenme olasılığının hem Atterberg limitlerine hem de CPT'ye dayalı olarak değerlendirilmesine olanak sağlamaktadır.

Anahtar Kelimeler: Sıvılaşma yatkınlığı, CPT, İnce daneli zeminler, dinamik deney.

To My Family...

ACKNOWLEDGEMENTS

First of all, I would like to express my gratitude to my grandfather Osman Gelişen, my mother Ülkü Pehlivan, my father Hayati Pehlivan and my lovely brother Tanzer, for their invaluable support, endless and unconditional love, consistent encouragement and trust on my abilities. I would not have been succeeded without them.

Then, I would like to express my sincere thanks and appreciation to my dear supervisor Assoc. Prof. Dr. Kemal Önder Çetin for his never ending support, continuous understanding and invaluable patience throughout this study. Above all, he never doubted my abilities and helped me clarify and realize my goals which I will remember forever.

I owe further thanks to H. Tolga Bilge for his invaluable contribution and support on this study.

I would also like to thank to Tolga Tonguç Değer for his understanding, patience, support and encouragement.

Last but not the least; I would like to express my gratitude to Onur Arı, Ömer Karamanlı and Sevinç Ünsal Oral for not leaving me alone during this study and for their support and sympathy.

TABLE OF CONTENTS

LIST OF FIGURES

FIGURES:

LIST OF TABLES

TABLES:

LIST OF ABBREVIATIONS

PI	Plasticity Index
LL	Liquid Limit
$\mathbf{w_c/LL}$	Water Content to Liquid Limit Ratio
FC	Fines Content
$\mathbf{D_R}$	Relative Density
$\mathbf{r_u}$	Excess Pore Pressure Ratio
γ_{max}	Double Amplitude Maximum Shear Strain
γ_c	Cyclic Shear Strain Amplitude
γ_r	Reference Shear Strain
G	Secant Modulus
λ	Damping Ratio
$\mathbf{M_w}$	Moment Magnitude
N	Number of Loading Cycles Applied in the Test
$\sigma'_{v,0}$	Initial Vertical Effective Stress
σ'_m	Mean Effective Stress
τ_{max}	Shear Stress at Failure

CSR	Cyclic Stress Ratio (= τ / σ'_v)
CRR	Cyclic Resistance Ratio
K_0	Coefficent of Lateral Stress at Rest
SPT	Standard Penetration Test
CPT	Cone Penetration Test
CTX	Cyclic Triaxial (Test)
CSS	Cyclic Simple Shear (Test)
$N_{1,60}$	Overburden- and Procedure-corrected SPT blow counts
$N_{1,60,CS}$	Overburden-, Procedure- and Fines corrected SPT blow counts
$q_{t,1,net}$	Normalized Net Cone Tip Resistance Ratio
f_s	Sleeve Friction Resistance
F_R	Friction Ratio
I_c	Soil Behavior Type Index

CHAPTER 1

INTRODUCTION

1.1 Research Statement

This study aims to develop a unified system for the assessment of liquefaction triggering potential of coarse and fine grained soils based on excess pore pressure ratio (r_u) and cyclic shear strain (γ_{max}) responses. For this purpose, results of cyclic tests performed on various types of soils, ranging from laboratory-reconstituted clean sands to “undisturbed” cohesive samples, were compiled. Based on the observed behavioral trends and test data, probabilistically-based boundary curves were developed in r_u vs. γ_{max} domain as a function of soil index and state parameters such as (i) plasticity index (PI), (ii) water content to liquid limit ratio (w_c/LL) and (iii) liquid limit (LL). Then, these boundary curves were mapped on to the normalized net cone tip resistance ($q_{t,1,net}$) vs. friction ratio (F_R) domain by using previous correlations relating soil index parameters with $q_{t,1,net}$ and F_R. This framework enables both Atterberg limits and CPT based assessment of liquefaction triggering potential of fine grained low-plasticity soils, differentiating clearly both cyclic mobility and liquefaction responses.

1.2 Problem Significance and Limitations of Previous Studies

Liquefaction of soils, which is defined as significant loss in shear strength and stiffness due to increase in pore pressure, has been one of the major reasons for damage and life loss during the earthquakes. After dramatically-severe losses due to liquefaction-induced damages observed during and after 1964 Niigata and Great Alaska earthquakes, numerous researches have been carried out to better understand this phenomenon. Almost five decades have passed after the pioneer studies, and meanwhile the number of both case histories and high-quality test data have increased. Yet, there are still some unknowns waiting to be resolved.

Until Haicheng (1975) and Tangshan (1976) earthquakes, it was believed that only “clean sandy soils” with few amount of fines do liquefy and cohesive soils were considered to be resistant to cyclic loading due to cohesional component of shear strength. However, those earthquakes showed that even cohesive soils could liquefy (Wang, 1979, 1981, 1984). 1994 Northridge, 1999 Adapazarı and 1999 Chi-Chi earthquakes further illustrated that silty and clayey soils may exhibit soil liquefaction response. In compliance with these observations, liquefaction susceptibility criteria for fine grained soils were improved by various researchers (e.g. Chinese Criteria 1979, Seed et al. 1983, Finn et al. 1994, Andrews and Martin 2000, Seed et al. 2003, Bray and Sancio 2006, Boulanger and Idriss 2006).

Although these studies are important to better understand the response, they have some limitations and suffer from one or more of the following issues; i) use of significantly different liquefaction definitions: some of which are based on field surface manifestations in the form of sand boils, excessive settlements or lateral spreading, and others are based on laboratory based exceedence of a threshold shear or axial strain (e.g.: $\gamma >$ 5 %) or excess pore pressure ratio, r_u level (e.g.: $r_u > 0.85$), ii) both field or laboratory based liquefaction triggering definitions require robust definition of the threshold performance levels, which may exhibit a significant variability depending on stress and state of the susceptible soil, iii) strain or r_u based liquefaction definitions require the determination of CSR levels and duration of the excitation.

More explicitly, the major limitation of previous criteria arises from the differences in the definition of liquefaction. There is no unique definition of liquefaction in literature, and therefore each of the previous studies set their own definition. Some of them are based on directly the field case histories (e.g. Chinese Criteria). Ground surface investigations are used to assess whether the site is liquefied or not in these studies. However, these investigations do not provide any insight regarding the mechanical behavior of the soil, other than the observation of large soil deformations. On the other hand, some others were developed based on laboratory test data, which enables the examination of the soil mechanics, yet even for these methods there is no consensus over the definition of liquefaction. Each study have used different definitions, such as fulfillment of r_u=1.00 condition, development of 5% double amplitude axial or 7.5% double amplitude shear strains, etc.

Another limitation arises from the ambiguity in loading amplitudes. Almost none of the previous studies have defined a loading amplitude condition such as cyclic stress ratio (CSR), under which their criteria is valid. Consequently, it is not clear under which cyclic loading levels their criteria are reliable.

For the purpose of eliminating some of these concerns, it is aimed to develop new unified liquefaction susceptibility criteria for fine-grained soils. The proposed criteria provide liquefaction susceptibility boundary curves as a function of soil index parameters (PI, LL, w_c/LL). The boundary curves developed in r_u vs. γ_{max} domain are then mapped on to CPT domain ($q_{t,1,net}$ vs. F_R), consistent with the recent study of Cetin and Ozan (2009).

1.3 Scope of the Thesis

Following this introduction, Chapter 2 presents an overview of previous studies on the assessment of liquefaction susceptibility by providing brief information about how these criteria were developed. Moreover, the shortcomings of each criterion were discussed separately.

Chapter 3 presents database compilation efforts. Data sources are introduced in detail along with the information about testing procedures and sample properties. Discrepancies in the database are also stated by using clear examples. This chapter is concluded by a summary of the compiled database.

Chapter 4 begins with a discussion on how liquefaction is defined in the literature and also introduces the definitions used in this study. The performance of the available criteria is also evaluated in view of different definitions by using the compiled database. The results of this evaluation study are also presented in this chapter and the deficiency of each criterion is discussed at the end.

Before introducing the proposed methodology, brief background information which will strengthen its validity is presented in Chapter 5, and in Chapter 6 the development of the proposed probabilistic models is discussed. Alternative models are introduced and selected correlations are presented. Then, new liquefaction susceptibility criteria developed based on selected correlationand its comparision with available liquefaction criteria is presented as the conclusion of this chapter.

Chapter 7 discusses estimation of liquefaction susceptibility margins in the CPT domain. The methodology followed is clearly described prior to generation of boundary curves according to the selected correlations provided in Chapter 6.

Finally, Chapter 8 summarizes the studies performed throughout this research and presents major conclusions.

CHAPTER 2

AN OVERVIEW OF PREVIOUS STUDIES ON LIQUEFACTION SUSCEPTIBILITY

2.1 Introduction

Liquefaction has been one of the major causes of damage and life loss during the earthquakes in the past. Thus, many researchers studied on this topic in order to understand the cyclic soil response and mechanisms leading to liquefaction. 1964 Niigata, Japan and 1964 Great Alaskan earthquakes played a major role in the development of modern liquefaction engineering. During these earthquakes seismically-induced liquefaction caused devastating effects, which drew the attention of researchers on this topic. Since then, many research projects have been carried out to assess the likelihood of liquefaction triggering of sandy soils. Additionally, every new earthquake produced value insight which helped to improve the state of knowledge and create new research areas. Figure 2.1 summarizes major components of liquefaction engineering (Seed et al., 2003). As illustrated in Figure 2.1, starting point of liquefaction engineering is to decide whether soil is susceptible to liquefaction or not, since assessment of likelihood of liquefaction triggering would be meaningless for nonliquefiable soils.

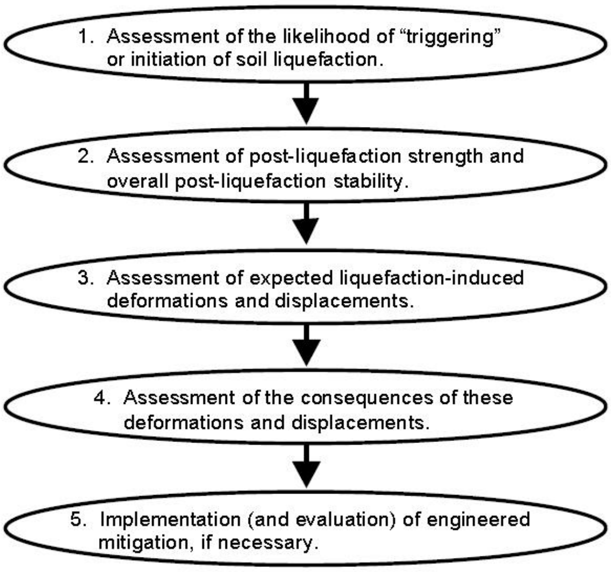

Figure 2.1 Key elements of soil liquefaction engineering (Seed et al., 2003)

Until Haicheng (1975) and Tangshan (1976) earthquakes, it was believed that only "clean sandy soils" with few amount of fines do liquefy and all techniques related to liquefaction triggering assessment had been developed based on this assumption. These earthquakes showed that cohesive soils could also liquefy (Wang, 1979, 1981, 1984). 1994 Northridge, 1999 Adapazarı and 1999 Chi-Chi earthquakes further illustrated that silty and clayey soils may exhibit cyclic soil liquefaction. Based on these observations and laboratory test results, various liquefaction susceptibility criteria for fine grained soils were recommended.

This chapter discusses widely used liquefaction susceptibility criteria proposed previously in the literature.

2.2 An Overview of Available Liquefaction Susceptibility Criteria

Numerous researches have focused on liquefaction susceptibility of fine grained soils. Major improvements were established by every new lesson gathered through every new earthquake. This section will attempt to summarize the journey of the evaluation of liquefaction susceptibility of fine grained soils starting with discussion of Chinese criteria, followed by Andrews and Martin (2001). Then, it continues with the recent criteria suggested by Seed et al. (2003), Bray and Sancio (2006) and Boulanger and Idriss (2004, 2006). Lastly, other criteria suggested by various researchers are summarized.

2.2.1 Chinese Criteria

Liquefaction susceptibility criteria proposed by Wang (1979), known as Chinese criteria, is based on the observed failures due to liquefaction in fine-grained soil profiles after 1975 Haicheng and 1976 Tangshan earthquakes occurred in China. A database was compiled from sites where liquefaction was observed or not. This database was composed of cohesive soils, whose clay fraction is less than 20%, LL is between 21 and 35, PI is between 4 and 14 and $w_c/LL > 0.90$ (Wang, 1979). Most of the fine grained soils were classified as low plasticity clays (CL) or silty clays (CL-ML) according to Unified Soil Classification System. According to Chinese criteria, clayey soils are susceptible to liquefaction if they contain 15-20 % of particles by weight smaller than 0.005 mm and if they have w_c/LL ratio greater than 0.90.

Chinese criteria were later modified by Seed and Idriss (1982) and named as “Modified Chinese Criteria”. It states that “clayey soils”, soils lying above A-line, are vulnerable to liquefaction only if they satisfy all following three conditions, (i) percent of particles less than 0.005 mm should be less than 15%, (ii) LL < 35 and (iii) w_c/LL > 0.90 (Seed and Idriss, 1982). Modified Chinese Criteria are illustrated in Figure 2.2. Later, Koester (1992) indicated that fall cone apparatus, used in Chinese practice, determined LL values about 3% to 4% greater than the values obtained when standard Casagrande percussion device is used. Hence a reduction in LL criteria was proposed.

Main limitations of this criterion can be summarized as follows; (i) data utilized in these studies were obtained from only two earthquakes which produced only a limited range of peak ground acceleration or corollary CSR levels, (ii) Chinese liquid limit and percent fine definitions do not comply with widely used standard definitions. Hence, their use for i) earthquake excitations producing significantly different level or duration of shaking and ii) for sites which exhibit significant different characters than the regional fine grain soils is questionable.

Following case histories also supported the fact that Chinese Criteria cannot satisfactorily identify potentially liquefiable soils. Based on their observations after 1989 Loma Prieta earthquake, Boulanger et al. (1998) proposed that the ground failure observed in Moss Landing Site can be due to cohesive materials and stated that use of Modified Chinese criteria without laboratory testing should be avoided.

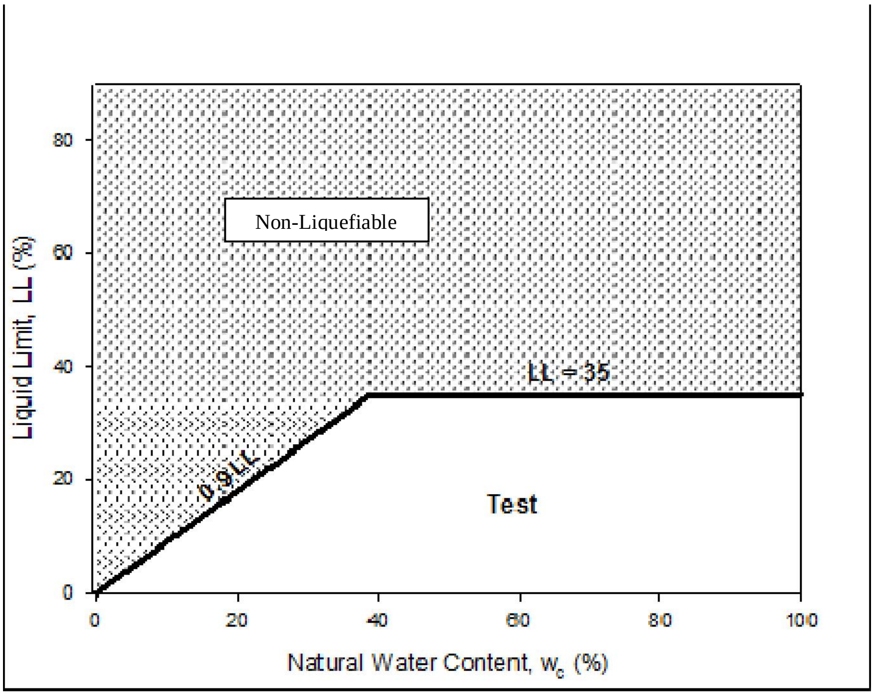

Figure 2.2 Modified Chinese criteria (Seed and Idriss, 1982)

Similarly, after 1994 Northridge Earthquake, Holzer et al. (1999) reported that the ground deformations were primarily due to liquefaction at Malden Street Site. This site was composed of “low strength lean clays” which were not considered to be susceptible to liquefaction according to Modified Chinese criteria. Holzer et al. stated that these observations were consistent with the findings after 1971 San Fernando earthquake.

During 1999 Adapazari and 1999 Chi-Chi earthquakes, sites which are classified as not susceptible to liquefaction by Modified Chinese criteria, experienced significant bearing capacity failures and settlements because of liquefaction of the fine grained (or cohesive) soils (Bray and Sancio, 2006).

2.2.2 Andrews and Martin (2000)

Andrews and Martin (2000) re-evaluated Modified Chinese criteria after re-examining the database provided by Wang (1979) and data obtained from earthquakes occurred in succeeding years. Authors suggested that fine grained soils have liquefaction triggering potential if (i) percent of fines by weight smaller than 0.002 mm is less than 10% and (ii) LL < 32. According to these criteria, soils satisfying just one of these conditions require further testing and ones that do not meet any of these conditions are not vulnerable to liquefaction. The proposed criteria are presented in Figure 2.3.

Andrews and Martin (2000) criteria further decrease the upper limit for LL and re-defined the boundary between silt-size and clay-size particles as 0.002 mm. Additionally, they do not consider w_c/LL ratio as a screening tool for the assessment of liquefaction susceptibility. However, w_c/LL parameter of Chinese criteria is stated as the ratio expressing soil sensitivity and differentiating criterion of Chinese criteria by Bray and Sancio (2006). The criteria proposed by Andrews and Martin (2000) also suffer from the same limitations of Chinese criteria, since they mainly use the same database.

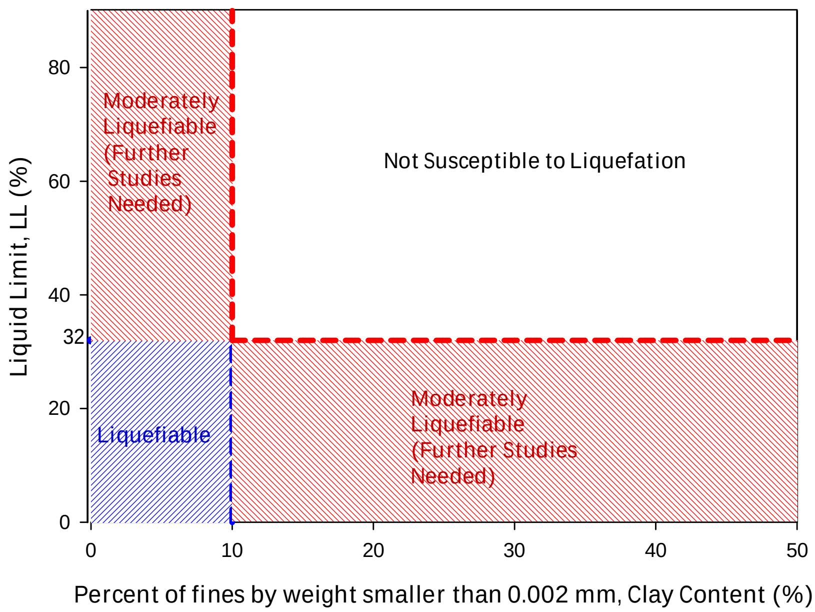

Figure 2.3 Liquefaction susceptibility criteria proposed by Andrews and Martin (2000)

2.2.3 Seed et al. (2003)

Based on both field observations after 1999 Adapazarı and Chi Chi earthquakes and laboratory test data, Seed et al. (2003) pointed out that soils of higher plasticity may be susceptible to significant excess pore water pressure increase and consequent loss of strength than it is determined by using the Modified Chinese criteria. It is also mentioned that there is a gradual transition in response of soils while plasticity is increasing and even high plasticity soils suffer from cyclic shearing to some degree. Authors added that with increasing plasticity, the level of shear strain required to trigger liquefaction also increases. Contrary to Chinese Criteria and Andrews and Martin (2000), this study stated that the important factor is the activity of clay particles rather than the percent of clay-size particles and using the latter parameter as a criterion may lead to unconservative conclusions regarding liquefaction susceptibility.

Based on these discussions, Seed et al. (2003) proposed new criteria for the assessment of liquefaction susceptibility of fine grained soils. These criteria classify soils based on soil index parameters as shown in Figure 2.4. Soils, which satisfy all three following conditions: (i) PI < 12, (ii) LL < 37 and (iii) w_c/LL > 0.8 fall into Zone A and considered to be potentially liquefiable. Soils lie in Zone B, i.e. satisfying the following conditions: (i) 12 < PI <20, (ii) 37 < LL < 47 and (iii) w_c/LL > 0.85, are classified to be moderately susceptible to liquefaction and need further testing. Soils lie out of these boundaries (named as Zone C) are not considered to be susceptible to "classical" liquefaction.

When compared with the Modified Chinese criteria and Andrews and Martin (2000) criteria, this methodology is much more reliable since it considers mineralogy rather than grain size by setting PI as a parameter for liquefaction. Moreover, Seed et al. (2003) accounts for sensitivity of fine-grained soils by considering w_c/LL ratio as a screening tool. However, authors do not clearly address how these criteria were developed. Instead, these criteria were proposed as a summary of their knowledge up to that date, gained from experimental researches and field case histories. Thus, it is not clear under which cyclic loading level these criteria are valid.

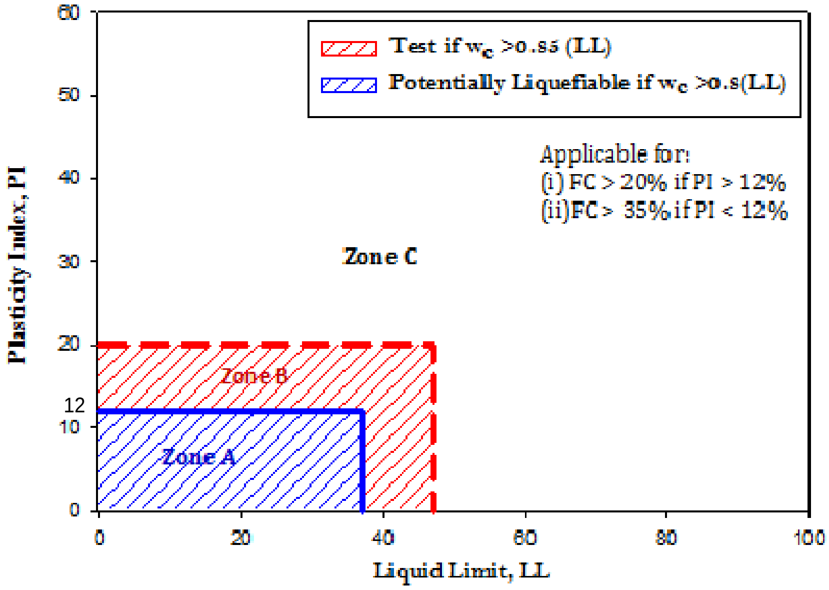

Figure 2.4 Recommended liquefaction susceptibility boundaries for fine-grained soils by Seed et al. (2003)

2.2.4 Bray et al. (2004b)

Bray et al. (2004b) criticized Modified Chinese criteria and methodology proposed by Andrews and Martin (2000). The criteria proposed by Bray et al. (2004b) were developed based on more than 100 cyclic triaxial tests (CTX), 19 static strength tests, 24 consolidation tests and numerous index tests performed on undisturbed specimens obtained from 7 different liquefied sites at Adapazari after 1999 Kocaeli earthquake. Specimens mostly have more than 15% of particles by weight smaller than 0.005 mm and more than 35% of particle smaller than 0.075mm, covering a large range on plasticity index scale (0 < PI <40). CTX tests are performed under undrained conditions at a frequency of either 1 Hz or 0.005 Hz. During CTX tests, one of three CSRs (0.3, 0.4 or 0.5) was applied to specimens. Bray et al (2004b) set number of cycles necessary to reach 3% single amplitude (5% double amplitude) axial strain as a criterion for liquefaction. According to liquefaction susceptibility criteria proposed by Bray et al. (2004b), soils with (i) $w_c/LL \geq 0.85$ and (ii) $PI \leq 12$ are considered to be vulnerable to liquefaction, soils having (i) $w_c/LL \geq 0.80$ and (ii) $12 < PI < 20$ are considered to be moderately susceptible to liquefaction or cyclic mobility and they require further laboratory testing for determination of their actual liquefaction potential. Soils having PI > 20 are considered to be non-liquefiable due to high clay content. Figure 2.5 summarizes the criteria proposed by Bray et al. (2004b).

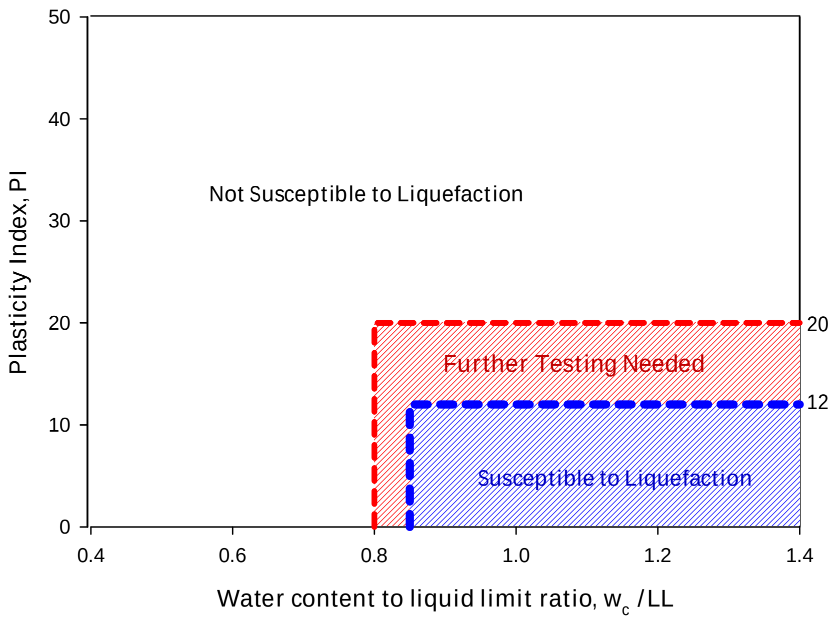

Figure 2.5 Liquefaction susceptibility margins defined by Bray et al. (2004b)

There are similarities between the criteria of Bray et al. and Seed et al. (2003), except LL parameter which is a common parameter of almost all of the previous studies. Bray et al. (2004b) dropped LL parameter since they observed that a number of specimens with LL > 35 found to be moderately susceptible to liquefaction. Bray et al. (2004b) provides more information regarding specimens used in their study and methodology of testing such as amplitude of loading (one of three different CSRs of 0.3, 0.4 and 0.5) applied during cyclic tests. However, problems like the ambiguity in the definition of liquefaction and amplitude of loading also exist in this study. Rather than a specific CSR level, a range of nominal CSR's is provided in this study, and it gives rise to the same question again: under which loading amplitudes these materials are liquefiable?

2.2.5 Bray and Sancio (2006)

Bray and Sancio (2006) further developed the criteria proposed by Bray et al. (2004b). In addition to CTX tests, 10 cyclic simple shear (CSS) tests were performed on undisturbed samples of silty and clayey soils obtained from the same sites. These fine-grained soils cover a relatively smaller range of PI (0< PI< 25) and have fines content (FC) generally greater than 70%. Moreover, the standard penetration test (SPT) blow counts, $(N_1)_{60}$, of these soils are in the range of 3 to 8.

According to Bray and Sancio (2006) soils with (i) w_c/LL > 0.85 and (ii) PI < 12 are vulnerable to liquefaction, soils having (i) w_c/LL > 0.80 and (ii) 12 < PI < 18 are moderately susceptible to liquefaction and they propose further laboratory testing for fine-grained soils located in this range; whereas, soils having PI > 18 are considered to be non-liquefiable under low effective stress levels due to their high clay content. Figure 2.6

illustrates the criteria proposed by Bray and Sancio (2006). This study adopted the same strain-based liquefaction criterion as proposed by Bray et al. (2004b).

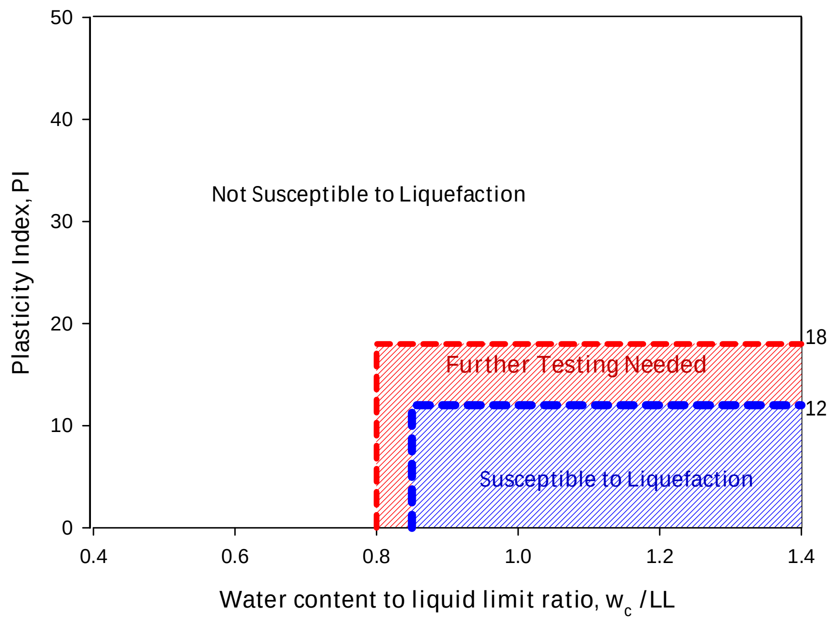

Figure 2.6 Liquefaction susceptibility margins proposed by Bray and Sancio (2006)

As shown in Figure 2.6, only the upper PI limit of Bray et al. (2004b) is modified for moderately liquefiable soils such that it is lowered from 20 to 18. These criteria are developed by using the same database used by Bray et al. (2004b). Therefore, these criteria also have the same shortcomings as previously discussed in Section 2. 2. 4.

2.2.6 Boulanger and Idriss (2004, 2006)

Boulanger and Idriss (2004, 2006) discussed the complications observed for the development of liquefaction susceptibility criteria due to difficulties associated with the definition of liquefaction. Consequently, based on an extensive literature survey; they have proposed new liquefaction susceptibility criteria in which fine-grained soils are classified into categories as, “sand-like” and “clay-like soils”. Sand-like soils are defined as fine-grained soils, which undergo cyclic liquefaction by exhibiting a response similar to sands; whereas clay-like soils are defined as fine-grained soils which undergo cyclic mobility rather than cyclic liquefaction. According to Boulanger and Idriss (2004, 2006) there exists a smooth transition from sand-like behavior to clay like behavior across a range of Atterberg Limits. Fine-grained soils with PI ≥ 7 (PI ≥ 5 for CL-ML soils) are considered to exhibit clay-like behavior and therefore they are vulnerable to cyclic mobility. On the other hand, fine grained soils with PI values between 3 and 6 are considered to exhibit transient behavior and therefore further testing is required for soils lying in this range. Soils that do not satisfy any of these conditions are considered to be liquefiable according to Boulanger and Idriss (2006).

Boulanger and Idriss discussed the validity of other parameters such as; Atterberg Limits, water content and clay content used in prior criteria for liquefaction susceptibility. In this regard, similar to Seed et al. (2003) and Bray et al. (2004a, b), Boulanger and Idriss state that clay content criterion, which was previously used in Chinese Criteria and Andrews and Martin (2000), is not proved to correlate well with the engineering properties to reflect liquefaction potential of fine-grained soils. They reported that w_c relative to Atterberg Limit value (LL) is a good indicator of cyclic failures in clay-like soils; however Boulanger and Idriss do not considered w_c/LL as a good screening tool to identify soil behavior (sand-like or clay-like) since soils can have high or low ratios of w_c/LL depending on the depositional environment. They recommend use of “liquefaction” term for sand-like soils and “cyclic failure” term for clay-like soils.

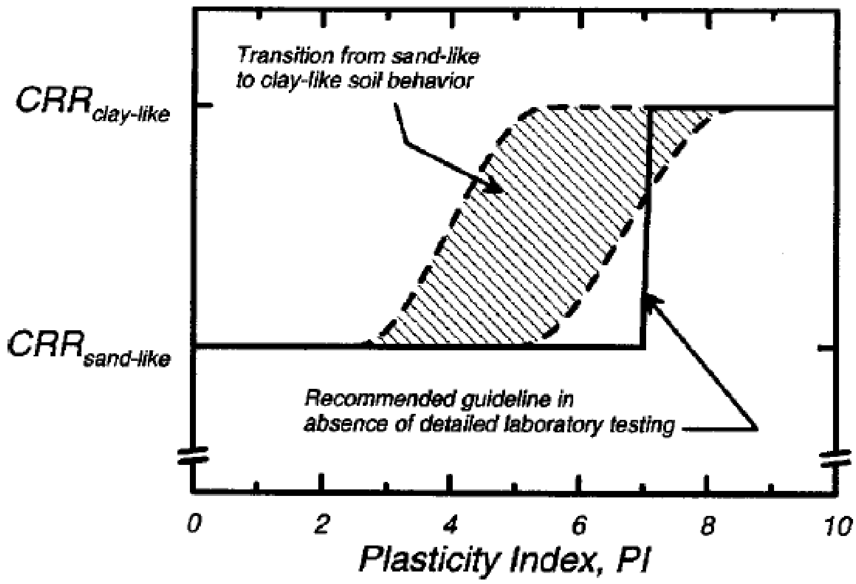

Figure 2.7 Schematic Illustration of the transition from sand-like to clay-like behavior for fine-grained soils with increasing PI, and recommended guidline for practice. (after Boulanger & Idriss, 2006)

Boulanger and Idriss provide a schematic illustration of liquefaction susceptibility boundary on cyclic resistance ratio (CRR) vs. PI domain (Figure 2.7). However, it should be noted that, the boundary curve, as illustrated in Figure 2.7, is not drawn to scale. Even though this study gives an insight regarding the change in liquefaction potential with amplitude of loading, i.e. CSR, still it does not propose a solution for its estimation.

2.2.7 Other Studies

In addition to previously mentioned studies, there have been numerous efforts to characterize liquefaction susceptibility of fine-grained soils which will be discussed in this section

Youd (1998) proposed that soils having "C" descriptor according to Unified Soil Classification System (USCS) are not vulnerable to liquefaction. Moreover, Youd also provides a screening tool for liquefaction such that fine grained soils lying below A-line with LL < 35 or soils having PI < 7 are considered to be susceptible to liquefaction.

Polito (2001) later proposed a liquefaction susceptibility criteria, illustrated in Figure 2.8, based on soil plasticity in which he suggested that soils are liquefiable if they have (i) PI < 7 and (ii) LL < 25, they are moderately liquefiable if they have (i) 7 < PI <10 and (ii) 25 < LL < 35, and soils are vulnerable to cyclic mobility if they have (i) 10 < PI < 15 and (ii) 35 < LL < 50; fine-grained soils lying out of these ranges are considered to be non-liquefiable. Even though Polito (2001) used parameters to account for the activity of clayey particles, he did not specify the cyclic stress ratio level of the cyclic tests performed. Thus, it is not clear under which seismic loading conditions these criteria are valid. On the other hand, similar to Andrews and Martin (2000), Polito (2001) does not consider w_c/LL, which reflects the sensitivity of soils as stated by Bray and Sancio (2006), as a screening parameter to identify liquefiable soils

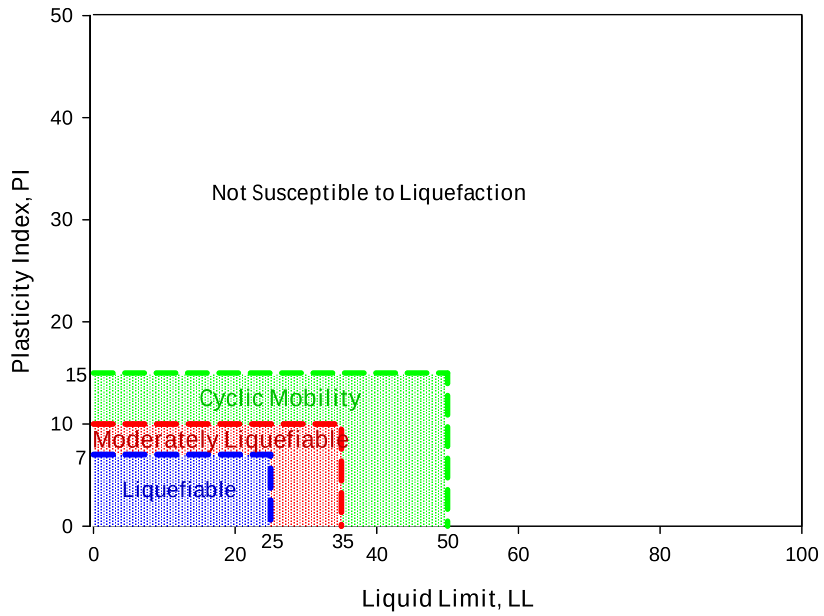

Figure 2.8 Liquefaction Susceptibility criteria proposed by Polito (2001)

Gratchev et al. (2006) examined the validity of PI as a screening tool for liquefaction susceptibility by performing undrained cyclic stress-controlled ring-shear tests on artificial mixtures of saturated soils with varying PI. They found that the liquefaction potential of fine-grained soils without high ion concentrations in their pore-water can be related to PI, such that an increase in PI causes a decrease in liquefaction potential of soil and for PI >15 soils are considered to be nonliquefiable.

Robertson and Wride (1998) proposed a criterion according to soil type behavior index (I_c) which can be determined using CPT parameters: normalized tip resistance, Q, and friction ratio, F_R. Authors suggest that soils with $I_c > 2.6$ are considered to be nonliquefiable; whereas soils with

$I_c < 2.6$ and $F_R \leq 1.0\%$ are considered to be very sensitive and vulnerable to liquefaction. However, the proposed cut-off value of I_c criticized by some researchers (Gilstrap, 1998; Zhang et al., 2002). They stated that 2.6 is too conservative and this boundary should be lowered. Later Youd et al. (2001) lowered I_c value to 2.4 and stated that soils with $I_c > 2.4$ should be further tested while soils with $I_c < 2.4$ are considered to be liquefiable.

Recently, Li et al. (2007) pointed out the deficiencies of the previous I_c -based classification and proposed using a modified soil behavior type index $I_{c,m}$, which depends also on pore pressure ratio (B_q). Using a similar database, Hayati and Andrus (2007) also studied the liquefaction susceptibility of fine-grained soils based on CPT data. However, they found that using $I_{c,m}$ with the boundaries given by Youd et al. (2001) is not consistent with the PI-based criteria of Bray and Sancio (2006). Thus, they recommended new criteria depending on both I_c and B_q such that soils with $I_c > 2.6$ or $B_q > 0.5$ are too clay rich to be liquefiable, soils with $I_c < 2.4$ and $B_q < 0.4$ are vulnerable to liquefaction and soils that lie in between these limits are moderately susceptible to liquefaction and need further testing.

2.3 Conclusions

Existing liquefaction susceptibility criteria for fine-grained soils were discussed within the confines of this chapter. Major limitations of the existing liquefaction susceptibility criteria can be summarized as; (i) there is no unique definition of liquefaction and hence, each criterion is developed based on different understandings regarding what liquefaction behaviour is; (ii) the amplitude of cyclic loading is not specifically defined in strain or r_u based exceedance of threshold definition, as a consequence there exist ambiguity under which cyclic stress conditions these criteria were applicable.

CHAPTER 3

DATABASE COMPILATION

3.1 Introduction

Efforts aiming to develop a semi-empirical or empirical model naturally require the compilation of a high quality database. Due to its nature, research studies focusing on seismic soil liquefaction triggering problem frequently need either case histories or laboratory test data. For this purpose a comprehensive database has been compiled and discussed within the confines of this chapter.

As introduced in the previous sections, numerous research studies have been performed on identification of soils susceptible to liquefaction. Most of these studies try to correlate liquefaction susceptibility with index (or sometimes gradational) properties of soils rather than focusing on the mechanisms leading to liquefaction, which includes excess pore water pressure generation and cyclic shear strain accumulation. Considering this limitation of previous efforts, this thesis focuses on r_u vs. γ_{max} response of fine-grained soils to develop probabilistically-based liquefaction susceptibility criteria. Moreover, to better understand the variations in behavioral patterns, cyclic tests performed on cohesionless soil specimens were also included into the compiled database.

The databases studied and compiled within the confines of this study can be grouped into two, based on type of soils tested, as: i) laboratory reconstituted clean sands (from Wu et al., 2003 and Bilge, 2005 databases) and ii) “undisturbed” fine-grained soils (Pekcan, 2001; Sancio, 2003; and Bilge, 2009). The compiled database includes information regarding the important problem descriptors such as r_u vs. γ_{max} histories, Atterberg limits along with moisture content of tested specimens and consolidation and applied cyclic shear stress conditions. The details of these databases will be introduced in the remaining sections of this chapter.

3.2 Database Compilation

As a result of a comprehensive database compilation study, a total number of 234 tests have been studied from different data sources as summarized in Table 3.1. After a brief introduction of these data sources, details of data processing will be discussed.

Table 3.1 Summary of data sources of companion study.

<table>
<tr><th></th><th>Reference</th><th>Number of cyclic tests</th></tr>
<tr><td rowspan="2">Coarse-Grained</td><td>Wu et al. (2003)</td><td>50</td></tr>
<tr><td>Bilge (2005)</td><td>36</td></tr>
<tr><td rowspan="3">Fine-Grained</td><td>Bilge (2009)</td><td>50</td></tr>
<tr><td>Pekcan (2001)</td><td>7</td></tr>
<tr><td>Sancio (2003)</td><td>91</td></tr>
<tr><td></td><td>Total</td><td>234</td></tr>
</table>

3.2.1 Data Sources Corresponding To Coarse-Grained Soil Specimens

For the purpose of differentiating the transition to cyclic response of fine-grained soils, tests performed on clean sand specimens were also studied. Databases presented by Wu et al. (2003) and Bilge (2005) were used for the purpose. Detailed information regarding these data sources and data processing is provided in following sections.

3.2.1.1 Wu et al. (2003)

Wu et al. (2003) performed stress controlled cyclic simple shear (CSS) tests on wet pluviated Monterey No.0/30 sand. Specimens were first K_0 consolidated to effective vertical stresses of 40, 80 or 180 kPa and then subjected to cyclic shearing with a frequency of 0.1 Hz.

A total of 50 cyclic simple shear (CSS) test were performed on Monterey No.0/30 sand. Relative density of specimens varies from 31 to 85%, and specimens were consolidated under initial effective vertical stresses ($\sigma'_{v,0}$) ranging from 32 to 182 kPa. Applied cyclic stress ratio (CSR) varies from 0.06 to 0.435. Tested specimens can be classified as clean sands (SW-SP) according to Unified Soil Classification System (USCS). A summary of individual tests performed by Wu et al. (2003) is summarized in Table 3.2 which presents relative density (D_R), initial effective vertical stress ($\sigma'_{v,0}$), fines content (FC), Overburden-, fines-, and the procedure-corrected Standard Penetration Test (SPT) blow counts ($N_{1,60,cs}$), cyclic stress ratio value corresponding to field conditions (CSR_{field}) and cyclic resistance ratio (CRR). Additionally, r_u and γ_{max} pairs are presented in Figure 3.1

Points in Figure 3.1 represent the excess pore pressures generated at the corresponding shear strain levels. Trend of clean sands can be observed easily after a careful examination of this plot. Considering the data quality and completeness of the criteria, all of the data compiled from Wu et al. (2003) database are used in further analysis.

Table 3.2 Database of Wu et al. (2003)

Specimen No	D_R	$\sigma'_{v,0}$	FC	$N_{1,60,CS}$	CSR_{field}	CRR
7j	58	33	0	15.5	0.396	0.177
19j	55	80	0	13.9	0.153	0.123
20j	45	85	0	9.3	0.205	0.086
23j	43	81	0	8.5	0.163	0.082
25j	56	85	0	14.4	0.310	0.126
28j	56	85	0	14.4	0.226	0.126
30j	77	77	0	27.3	0.398	0.339
31j	79	79	0	28.7	0.445	0.375
33j	71	80	0	23.2	0.353	0.247
35j	75	85	0	25.9	0.465	0.297
38j	50	75	0	11.5	0.197	0.104
41j	44	81	0	8.9	0.124	0.084
43j	31	82	0	4.4	0.123	0.060
46j	61	32	0	17.1	0.302	0.202
47j	45	32	0	9.3	0.295	0.112
48j	33	39	0	5.0	0.180	0.077
49j	33	34	0	5.0	0.140	0.080
59j	58	98	0	15.5	0.199	0.131
60j	56	40	0	14.4	0.291	0.155
61j	66	36	0	20.0	0.314	0.243
62j	50	36	0	11.5	0.216	0.128
63j	40	43	0	7.4	0.154	0.089
66j	67	37	0	20.6	0.460	0.253
73j	46	43	0	9.7	0.211	0.107
78j	63	42	0	18.3	0.333	0.204
79j	60	79	0	16.6	0.225	0.151

Table 3.2 Database of Wu et al. (2003) (continued)

Specimen No	D_R	$\sigma'_{v,0}$	FC	$N_{1,60,CS}$	CSR_{field}	CRR
80j	38	34	0	6.6	0.141	0.090
82j	47	45	0	10.2	0.212	0.109
83j	63	43	0	18.3	0.487	0.203
84j	63	173	0	18.3	0.170	0.138
85j	64	182	0	18.8	0.188	0.142
86j	65	178	0	19.4	0.229	0.149
88j	45	180	0	9.3	0.109	0.070
90j	54	182	0	13.4	0.131	0.094
91j	85	180	0	33.2	0.283	0.419
92j	55	180	0	13.9	0.135	0.098
93j	60	182	0	16.6	0.135	0.119
94j	84	178	0	32.5	0.284	0.396
96	81	179	0	30.2	0.279	0.334
99	81	178	0	30.2	0.294	0.334
100j	83	177	0	31.7	0.349	0.375
102j	51	180	0	12.0	0.132	0.085
103j	54	180	0	13.4	0.146	0.095
104j	50	176	0	11.5	0.159	0.082
106j	81	178	0	30.2	0.142	0.334
107j	78	38	0	28.0	0.638	0.435
108j	74	44	0	25.2	0.522	0.339
110j	81	83	0	30.2	0.461	0.413
124j	49	81	0	11.0	0.271	0.099
125j	64	79	0	18.8	0.427	0.179

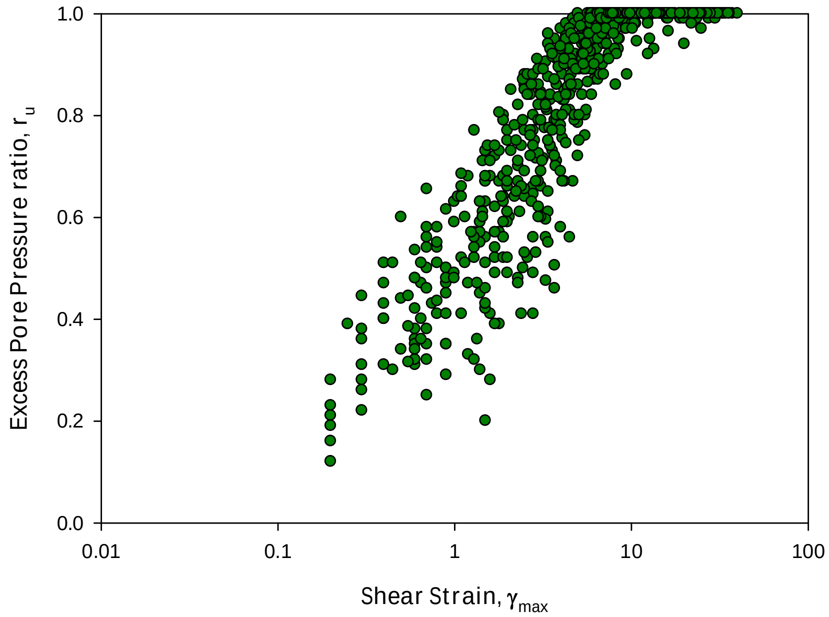

Figure 3.1 Database of Wu et al. (2003) plotted on r_u vs. γ_{max} domain.

3.2.1.2 Bilge (2005)

Bilge (2005) performed stress-controlled cyclic triaxial (CTX) tests on poorly graded Kizilirmak River sand. Depending on the target relative density (D_R), either dry pluviation or moist tamping method was used to reconstitute samples in the laboratory and then specimens were consolidated isotropically under a confinement pressure of 100 kPa. The frequency of cyclic loading was selected as 1 Hz. and specimens were subjected to 20 loading cycles which simulated duration of an earthquake having M_w of 7.5 (Liu et al. 2001). During cyclic loading excess pore water pressure and axial strains were recorded. Shear strains were calculated by simply multiplying recorded axial strains with a strain conversion factor. Cetin et al. (2009) provided a detailed discussion on strain conversion factors. In this study, it was stated that these factors varied in the range of 1.5 (recommended by Ishihara and Yoshmine (1992) for undrained conditions) to $\sqrt{3}$ ($\approx$1.73) (recommended by Vucetic and Dobry (1988) for marine clays straining in the plastic range). Based on these previous recommendations, Cetin et al. performed a discriminant analyses and their results indicated that a value of 1.5 would eliminate the variability in the recorded strain values of a triaxial test, and this value was also adopted in this study.

Data from a total number of 36 CTX tests were gathered from Bilge (2005) database to be used in further analysis. As summarized in Table 3.3, relative density (D_R) of reconstituted specimens vary from 35 to 85%, and all of them were isotropically consolidated under an initial vertical effective stress (σ'_{v0}) of 100 kPa. Specimens can be classified as SW-SP according to USCS. Applied cyclic stress ratio (CSR) varies from 0.10 to 0.58. The compiled test data is also presented in Figure 3.2. Each point in Figure 3.2 represents a r_u - (γ_{max}) pair recorded at the end of a loading cycle.

Points in Figure 3.2 represent the excess pore pressures generated at the corresponding shear strain level. Trend of clean sands can be observed easily after a careful examination of this plot. In this regards, all of the data provided by Bilge (2005), similar to Wu et al. (2003), is selected for further use in analysis.

Table 3.3 Database of Bilge (2005)

Specimen No	D_R	$\sigma'_{v,0}$	FC	$N_{1,60,CS}$	CSR_{field}	CRR
1	75	100	0	46.0	0.382	0.284
3	85	100	0	46.0	0.512	0.493
4	80	100	0	46.0	0.451	0.371
5	82	100	0	46.0	0.557	0.415
6	75	100	0	46.0	0.490	0.284
7	80	100	0	46.0	0.401	0.371
8	66	100	0	46.0	0.541	0.183
9	77	100	0	46.0	0.208	0.315
10	60	100	0	46.0	0.315	0.141
11	69	100	0	46.0	0.190	0.211
12	65	100	0	46.0	0.093	0.175
13	71	100	0	46.0	0.461	0.232
14	75	100	0	46.0	0.245	0.284
15	75	100	0	46.0	0.196	0.284
16	75	100	0	46.0	0.196	0.284
17	65	100	0	46.0	0.510	0.175
18	60	100	0	46.0	0.360	0.141
19	74	100	0	46.0	0.146	0.270
21	35	100	0	46.0	0.141	0.062
23	65	100	0	46.0	0.278	0.175
24	57	100	0	46.0	0.308	0.125
30	75	100	0	46.0	0.137	0.284
32	72	100	0	46.0	0.386	0.244
34	64	100	0	46.0	0.387	0.167
35	46	100	0	46.0	0.241	0.084
37	82	100	0	46.0	0.314	0.415
38	64	100	0	46.0	0.323	0.167
40	55	100	0	46.0	0.174	0.116
41	60	100	0	46.0	0.450	0.141
42	75	100	0	46.0	0.490	0.284
43	59	100	0	46.0	0.214	0.135
44	45	100	0	46.0	0.318	0.082
45	59	100	0	46.0	0.250	0.135
46	85	100	0	46.0	0.256	0.493
47	75	100	0	46.0	0.392	0.284
48	84	100	0	46.0	0.122	0.465

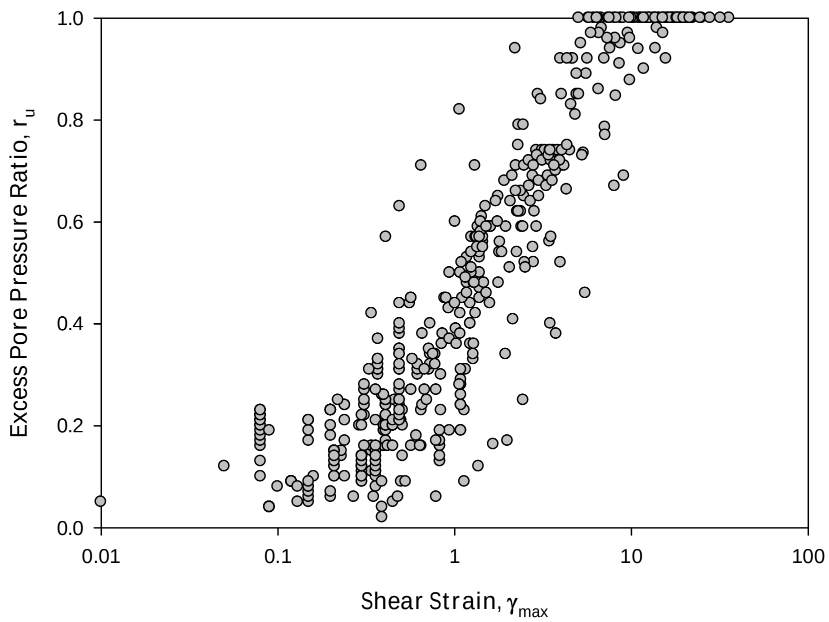

Figure 3.2 Database of Bilge (2005) plotted on r_u vs. γ_{max} domain.

3.2.2 Data Sources Corresponding to Fine-Grained Soil Specimens

The main motivation of this thesis is to develop criteria for the assessment liquefaction susceptibility of fine-grained soils. For this purpose, databases of Pekcan (2001), Sancio (2003) and Bilge (2009) were studied. A brief overview of data sources and data processing efforts will be presented next.

3.2.2.1 Pekcan (2001)

In this study, series of stress-controlled cyclic triaxial tests were performed to investigate the cyclic response of Adapazari silt and clay mixtures, which exhibited either cyclic mobility or cyclic liquefaction type responses during the 1999 Adapazari earthquake. “Undisturbed” specimens were retrieved from 5 selected sites where significant foundation displacements were observed during this earthquake. Pekcan (2001) reported a total of 11 cyclic test results and 7 of these tests were used in this study due to data quality considerations. For these 7 specimens, PI values vary in the range of 1 to 35; whereas LL values vary between 6 and 61. w_c/LL ratio was calculated based on the reported moisture contents and this ratio varies from 0.7 to 1.07. Samples can be classified as CH, CL-ML and ML according to USCS and their corresponding locations on plasticity chart are presented in Figure 3.3. Table 3.4 also summarize index properties of the tested specimens

In the followed testing procedure, first “undisturbed” specimens were consolidated to 50 kPa isotropically and saturated by back-pressure until a B-value of minimum 0.95 is achieved. Loading frequency was selected as 1 Hz and during loading phase, both axial strain and excess pore pressure measurements were made. Axial strain to shear strain conversion was performed by again using a strain conversion factor of 1.5 for the sake of consistency. Figure 3.4 presents the CTX test results compiled from Pekcan (2001) database in r_u vs. γ_{max} domain.

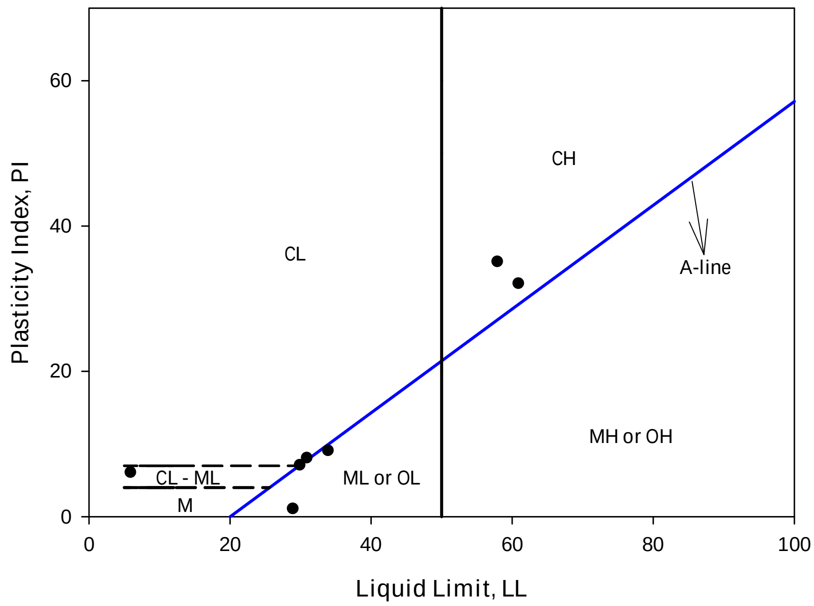

Figure 3.3 Plasticity chart showing specimens constituting Pekcan (2001) database

Table 3.4 Soil index properties of specimens tested (Pekcan, 2001)

Specimen No	**PI**	**w_c/LL**	**LL**
C1-1	35	0.67	58
C1-3	9	1.06	34
D2-1	7	1.07	30
D2-2	8	0.94	31
E1-2	32	0.64	61
J3-2	6	1.00	6
J3-3	1	1.00	29

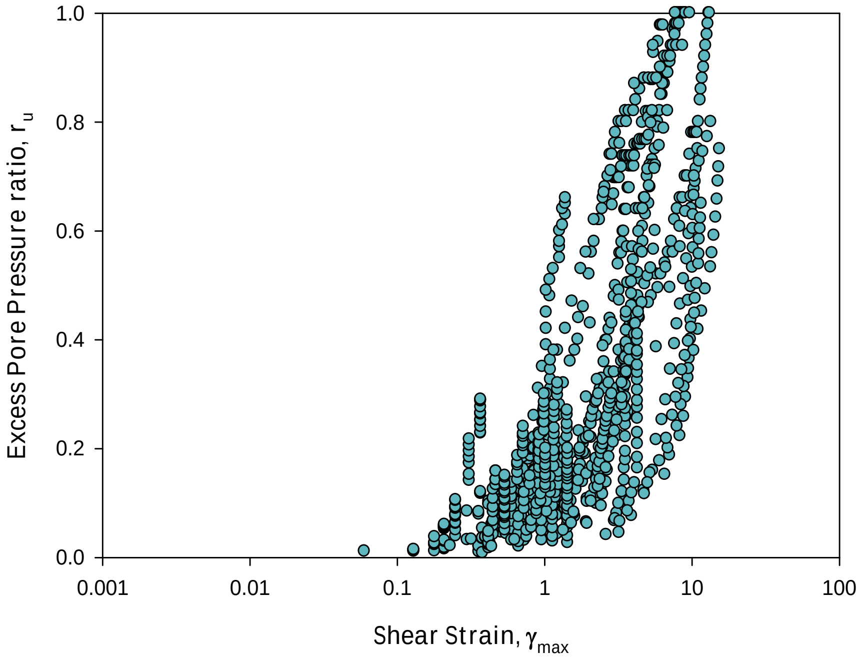

Figure 3.4 Graphical representation of CTX test results of Pekcan (2001) on r_u vs. γ_{max} domain

Figure 3.4 clearly illustrates the trend of clayey silt and silty clay mixtures on r_u vs. γ_{max} domain which indicates that the level of shear strain required to generate a certain value of r_u is greater for fine-grained soils compared to clean saturated sands. This observation is consistent with the previous findings of Matasovic and Vucetic (1995) and Hsu and Vucetic (2006).

All the test results were examined carefully to eliminate any inconsistencies. Even though only 7 CTX test results were compiled from Pekcan (2001), it serves quite valuable data to model the behaviour of clayey silt and silty clay mixtures.

3.2.2.2 Bilge (2009)

Bilge (2009) have focused on the cyclic straining response of fine-grained soils and performed series of stress-controlled cyclic triaxial tests on “undisturbed” soil specimens retrieved from several locations of Turkey, such as cities of Adapazari, Duzce and Ordu.

From Bilge (2009) database, a total number of 50 tests have been selected and studied. These tests were performed mainly on clayey soils having PI, LL and w_c/LL ratio vary in the ranges of 4.9 to 58.9, 27 to 94, and 0.41 to 1.08, respectively. Figure 3.5 presents the locations of these specimens on plasticity chart, and Table 3.5 summarizes the index parameters of the specimens which can be classified as either CL or CH according to USCS.

In the test program followed by Bilge (2009) specimens were consolidated either isotropically or anisotropically under effective vertical stresses varying in the range of 60 to 150 kPa. Samples were subjected to 20 cycles of loading having frequency of 1 Hz. During loading phase, axial strain and generated excess pore pressures were recorded. It was observed that after the end of cyclic loading, pore water pressures increased before reaching a steady value. Low permeability and high plasticity is considered to be responsible from this delayed pore water pressure generation response. The final value of excess pore water pressure was recorded and pore water pressure measurements were adjusted according to this value. Maximum double amplitude shear strains were obtained again by simply multiplying double amplitude axial strain values with a strain conversion factor of 1.5 as discussed before.

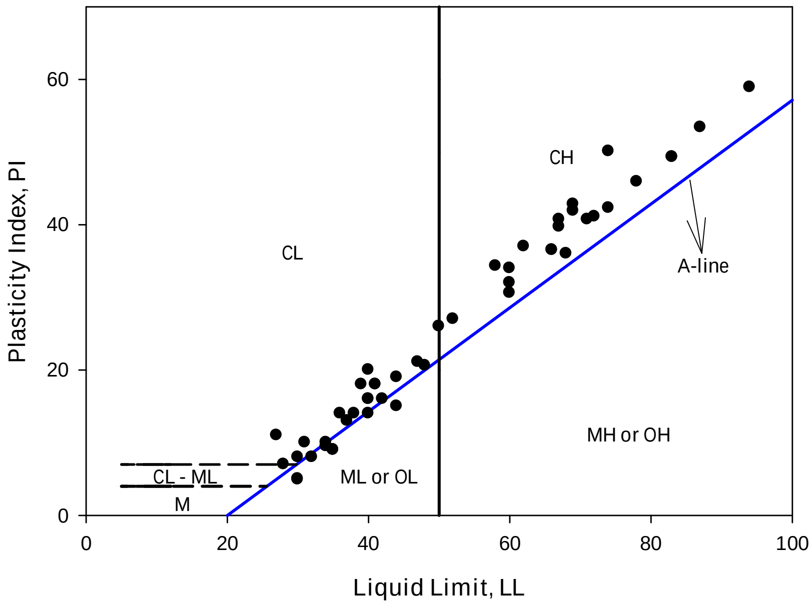

Figure 3.5 Plasticity chart showing specimens constituting Bilge (2009) database

Figure 3.6 presents the compiled data is presented in r_u vs. γ_{max} domain and the observed trends are consistent with the expectations. As the PI of specimens increases, r_u - γ_{max} data pairs shift to the right.

Table 3.5 Soil index properties of specimens tested in Bilge (2009)

Specimen No	**PI**	**w_c/LL**	**LL**
CTXT32	9.5	0.7	34
CTXT34	4.9	0.7	30
CTXT16	10	0.75	31
CTXT28	8	0.95	30
CTXT15	7	0.83	28
CTXT29	5	0.84	30
CTXT13	8	0.93	32
CTXT5	9	0.91	35
CTXT11	9	0.81	35
CTXT12	9	0.73	35
CTXT14	10	0.69	34
CTXT20	16	0.77	40
CTXT18	16	0.58	42
CTXT30	16	0.62	40
CTXT25	11	0.42	27
CTXT21	14	0.8	40
CTXT31	14	0.72	38
CTXT22	15	0.74	44
CTXT1	14	1.08	36
CTXT3	13	0.98	37
CTXT2	18	1	39
CTXT9	18	0.75	41
CTXT10	18	0.7	41
CTXT6	19	0.83	44
CTXT7	20	0.75	40
CTXT63	20.6	0.72	48
CTXT64	21.1	0.73	47
CTXT4	26	0.74	50
CTXT27	27	0.72	52
CTXT44	30.6	0.85	60
CTXT36	32	0.54	60
CTXT35	34	0.41	60
CTXT53	34.3	0.52	58
CTXT45	36	0.76	68
CTXT61	36.5	0.64	66
CTXT62	36.5	0.54	66
CTXT51	37	0.77	62
CTXT38	39.7	0.49	67
CTXT58	40.7	0.62	67
CTXT59	40.7	0.61	71
CTXT56	41.1	0.74	72
CTXT37	41.9	0.48	69
CTXT55	42.3	0.63	74
CTXT50	42.8	0.62	69

Table 3.5 Soil index properties of specimens tested in Bilge (2009). (continued.)

Specimen No	PI	w_c/LL	LL
CTXT48	45.9	0.67	78
CTXT60	49.3	0.56	83
CTXT47	49.3	0.61	83
CTXT49	50.1	0.6	74
CTXT46	53.4	0.58	87
CTXT52	58.9	0.49	94

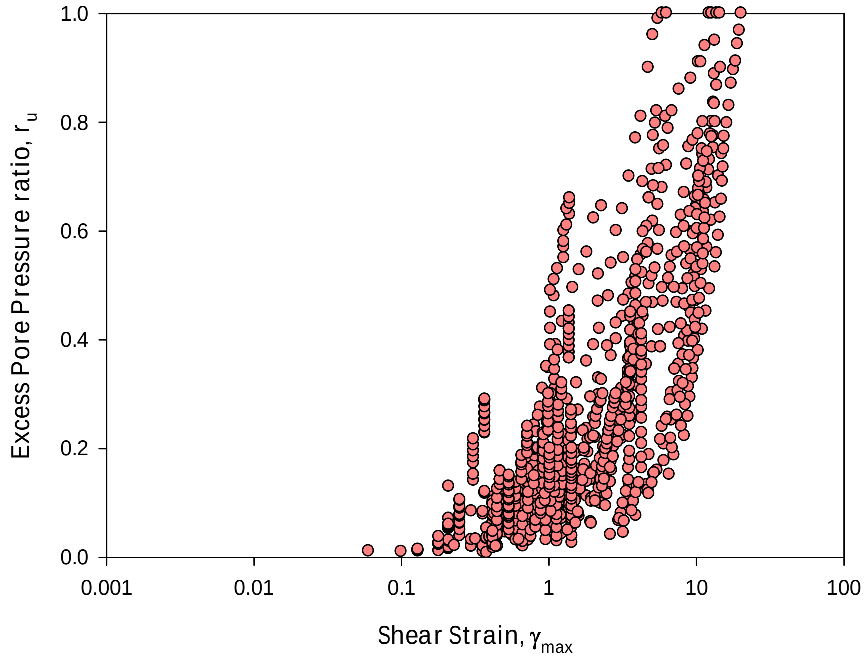

Figure 3.6 Graphical representation of CTX test results of Bilge (2009) on r_u vs. γ_{max} domain

3.2.2.3 Sancio (2003)

Sancio (2003) performed an extensive experimental study on silt and clay mixtures from 7 different liquefied sites of Adapazari, Turkey after 1999 Adapazari earthquake. Over 100 CTX, 10 CSS, 19 anisotropically consolidated monotonic triaxial and 24 consolidation tests along with numerous index tests were performed on these samples. Considering the effects of loading frequency on cyclic response of plastic fine-grained soils (Zavoral and Campanella 1994), only 91 of these tests, in which frequency of loading was 1 Hz, were studied in this study.

Figure 3.7 presents the locations of these specimens on plasticity chart. The PI, LL and w_c/LL ratio of the tested specimens vary between 0 and 40, 0 and 71, 0.50 and 1.50, respectively. Tested specimens can be classified as CL, CH, M and ML according to USCS and their index properties were summarized in Table 3.6.

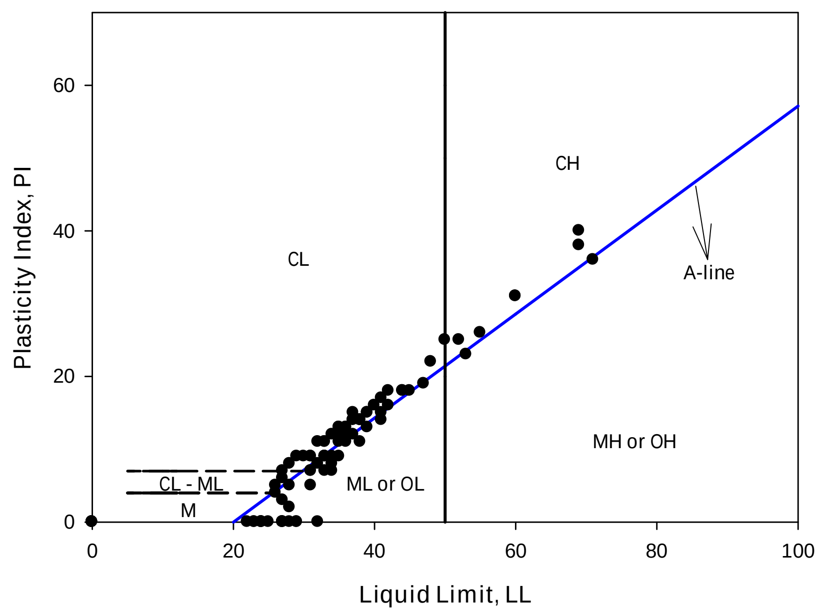

Figure 3.7 Plasticity chart showing specimens constituting Sancio (2003) database

Table 3.6 Soil index properties of specimens tested (Sancio, 2003)

Specimen No.	FC (%)	PI	LL	w_c/LL
F5-P2B	78.00	5.00	28.00	1.10
F7-P1B	81.00	7.00	31.00	1.00
J5-P4A	87.00	7.00	31.00	1.00
C11-P2A	87.00	11.00	32.00	1.10
I2-P7B	82.00	0.00	32.00	1.10
F6-P3B	68.00	2.00	28.00	1.10
F7-P4A	77.00	9.00	33.00	1.00
F7-P3B	61.00	0.00	24.00	1.30
F6-P4A	92.00	5.00	31.00	1.00
F8-P3A	71.00	4.00	26.00	1.20
G5-P1A	67.00	5.00	26.00	1.20
G5-P2B	75.00	0.00	27.00	1.20
C12-P2A	79.00	0.00	24.00	1.20
C12-P2B	74.00	9.00	30.00	1.10
A5-P2A	51.00	0.00	27.00	1.20
D5-P2A	68.00	0.00	25.00	1.20
D5-P2B	70.00	8.00	28.00	1.10
D4-P2A	75.00	6.00	27.00	1.00
D4-P2B	84.00	11.00	33.00	0.90
J5-P3A	70.00	7.00	27.00	1.00
J5-P3B	57.00	0.00	23.00	1.30
J5-P2A	56.00	0.00	24.00	1.40
J5-P2B	84.00	12.00	34.00	1.00
I6-P4	90.00	9.00	31.00	1.00
I6-P6	80.00	7.00	34.00	1.10
I6-P5	82.00	11.00	35.00	1.10
I8-P1B	47.00	0.00	23.00	1.30
I4-P5B	87.00	12.00	37.00	0.90
A5-P6A	-	9.00	34.00	0.90
A5-P6B	84.00	11.00	36.00	0.90
A6-P6A	95.00	11.00	38.00	0.90
A6-P9A	95.00	12.00	35.00	1.10
F4-P7A	93.00	7.00	33.00	1.10
I8-P3A	68.00	0.00	28.00	1.30
F4-P2A	67.00	0.00	24.00	1.30
A6-P5A	81.00	9.00	31.00	1.10
A6-P1A	84.00	3.00	27.00	1.30
F9-P2A	81.00	0.00	29.00	1.00
F4-P2B	61.00	0.00	22.00	1.50
F9-P2B	-	0.00	0.00	-
F7-P1A	88.00	8.00	34.00	0.90
F7-P3A	77.00	0.00	27.00	1.00
F6-P4B	99.00	9.00	35.00	1.00
F8-P3B	58.00	0.00	24.00	1.10
F4-P7B	69.00	8.00	32.00	1.00
A6-P6B	-	12.00	36.00	1.00
I6-P7	90.00	14.00	41.00	0.90
C14-P2B	96.00	13.00	36.00	1.10

Table 3.6 Soil index properties of specimens tested (Sancio, 2003) (continued)

Specimen No.	FC (%)	PI	LL	w_c/LL
D4-P4A	92.00	14.00	37.00	1.00
C14-P2A	98.00	14.00	38.00	1.10
C12-P3A	95.00	16.00	40.00	1.00
C10-P3B	97.00	14.00	38.00	1.10
C10-P3A	100.00	19.00	47.00	0.90
C11-P4B	99.00	14.00	38.00	1.00
G4-P2B	91.00	13.00	36.00	1.00
A6-P5B	90.00	15.00	39.00	1.00
A6-P8B	93.00	16.00	42.00	1.00
A6-P10A	97.00	18.00	44.00	0.90
A5-P9A	96.00	17.00	41.00	0.90
F4-P6A	97.00	18.00	45.00	0.80
A6-P9B	91.00	15.00	39.00	1.10
I8-P1A	83.00	13.00	35.00	0.90
I8-P2A	75.00	13.00	35.00	1.10
I8-P2B	89.00	18.00	42.00	0.90
A6-P10B	90.00	14.00	38.00	1.10
I7-P1	100.00	36.00	71.00	0.50
A6-P2B	99.00	23.00	53.00	0.70
A6-P3A	100.00	40.00	69.00	0.60
C10-P4A	100.00	31.00	60.00	0.70
C10-P4A	100.00	31.00	60.00	0.70
C11-P4A	99.00	22.00	48.00	0.90
C12-P4A	98.00	25.00	50.00	0.90
C10-P4B	100.00	38.00	69.00	0.70
J5-P6A	100.00	25.00	52.00	0.80
A6-P8A	99.00	26.00	55.00	0.70
F5-P2A	80.00	9.00	33.00	0.00
F7-P4B	87.00	9.00	33.00	1.00
D4-P3A	79.00	9.00	29.00	1.00
D4-P3B	89.00	11.00	33.00	1.10
A5-P5B	94.00	13.00	39.00	0.90
A6-P7A	79.00	0.00	27.00	1.00
C12-P3B	89.00	15.00	37.00	1.00
C11-P2B	99.00	18.00	44.00	0.90
C10-P8A	56.00	0.00	0.00	-
C10-P8B	83.00	0.00	27.00	1.30
I8-P5A	98.00	15.00	41.00	0.90
I8-P5B	78.00	0.00	29.00	1.10
G4-P4A	47.00	0.00	0.00	-
G4-P4B	64.00	0.00	0.00	-
G4-P5B	56.00	0.00	0.00	-
G4-P5A	89.00	14.00	37.00	-

Sancio (2003) presented his data using 4 way plots as: i) deviatoric stress vs. number of cycles, ii) axial strain vs. number of cycles, iii) excess pore water pressure vs. number of cycles, iv) axial strain vs. deviatoric stress. A sample test result is presented in Figure 3.8 which belongs to specimen F7-P21B. From these plots, it is possible obtain the complete excess pore water pressure vs. double amplitude axial strain history for each test.

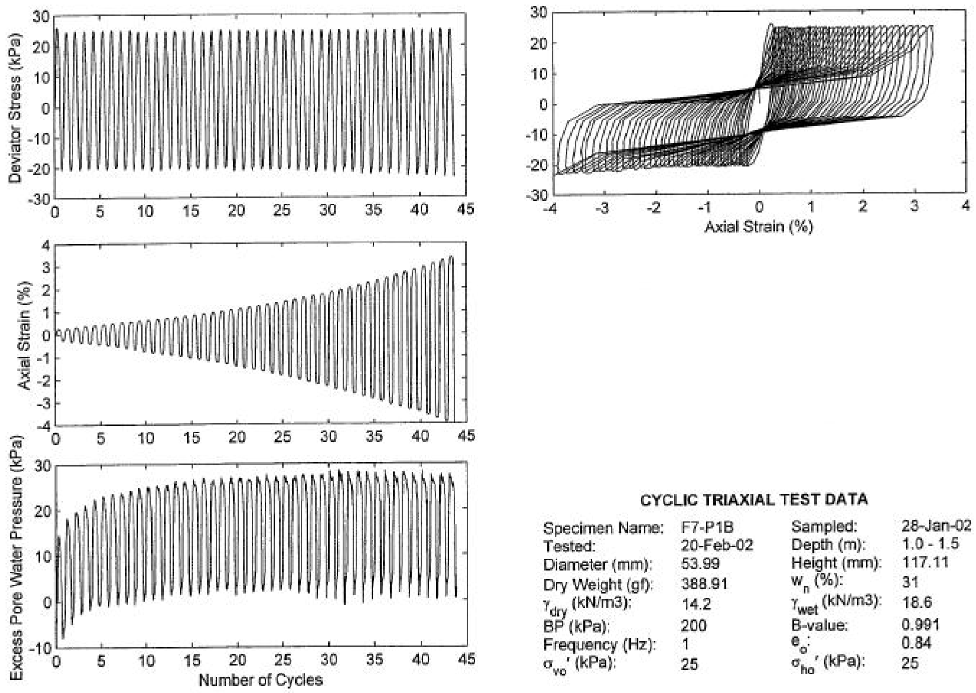

Figure 3.8 4-way plot for F7-P1B (Sancio, 2003)

After obtaining excess pore pressure and axial strain for each uniform loading cycle, excess pore pressure ratio (r_u) is calculated for each uniform loading cycle by simply dividing the measured excess pore pressure values to the reported vertical effective stress (σ'_{vo}) and axial strains were converted into shear strains by simply multiplying with 1.5

as discussed in previous sections. Figure 3.9 illustrates the CTX test results of Sancio (2003) on r_u vs. γ_{max} domain. This figure reveals that most of the r_u values exceed 1.00 which is an indication of a measurement error in either pore water pressure readings or reported effective stresses. For this reason, for the tests, where banana loops were observed, excess pore pressure values were scaled to produce r_u values of maximum 1.0. Figure 3.10 shows CTX test results in the r_u vs. γ_{max} domain after these adjustments .

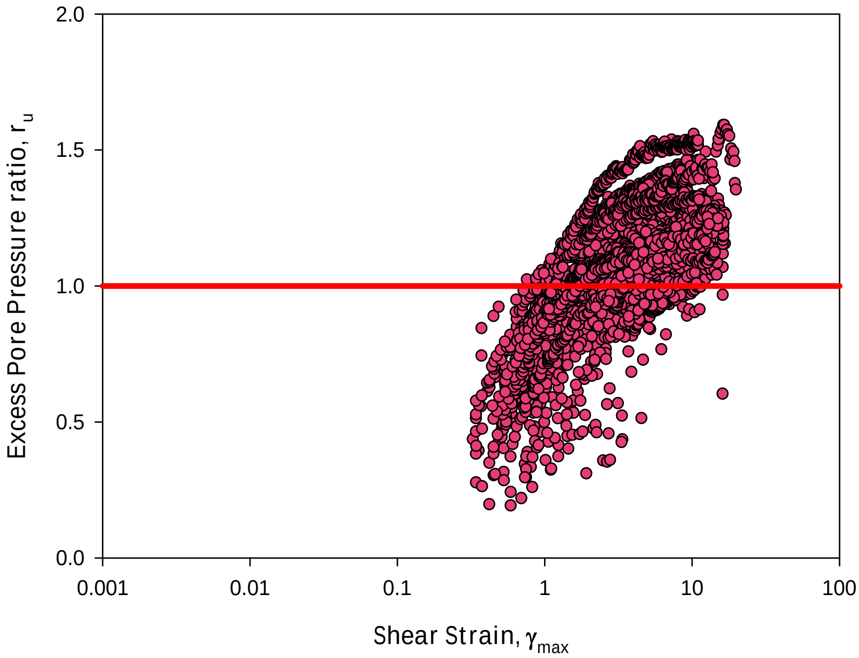

Figure 3.9 Graphical representation of CTX test results of Sancio (2003) on r_u vs. γ_{max} domain

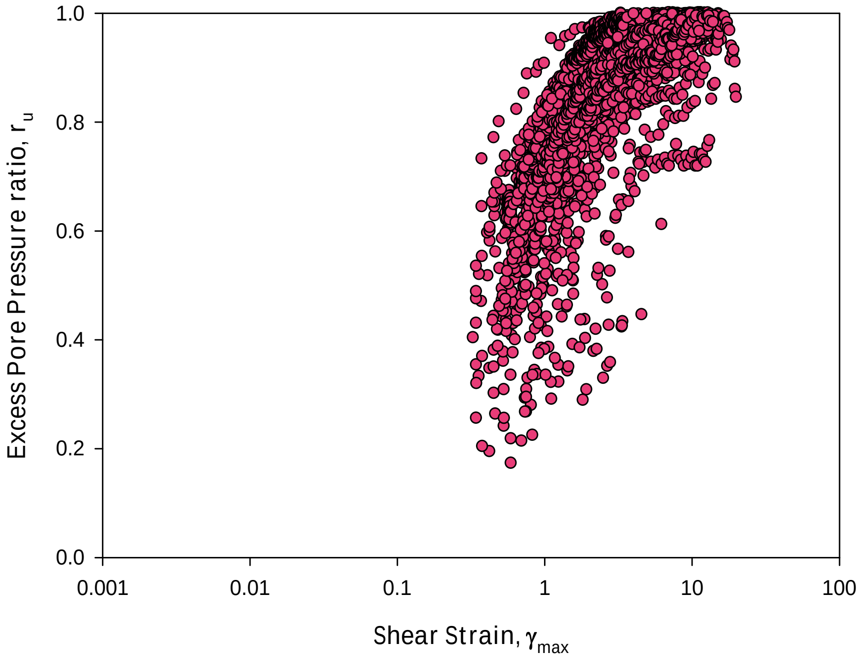

Figure 3.10 Graphical representation of "calibrated" CTX test results of Sancio (2003) on r_u vs. γ_{max} domain

After this adjustment, the data pairs are compared with data obtained from other data sources, in Figure 3.11 and it was observed that the adjusted data, surprisingly, located at the left side of saturated clean sand data which is not comforting considering the slower pore water pressure generation mechanism in cohesive soils. For this reason, this data was filtered again, and only 15 of the adjusted test data were selected to be used in development of correlations. However, calibrated database of Sancio (2003) is utilized for the evaluation of performance of previous susceptibility criteria, since it provides valuable information regarding clayey silt and silty clay mixtures.

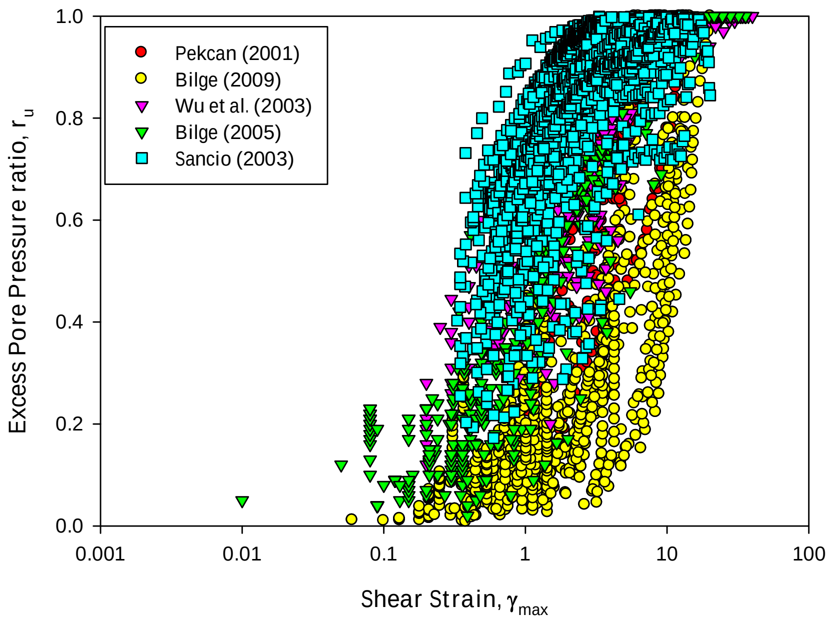

Figure 3.11 Comparison plot for calibrated Sancio (2003) database along with previously selected database.

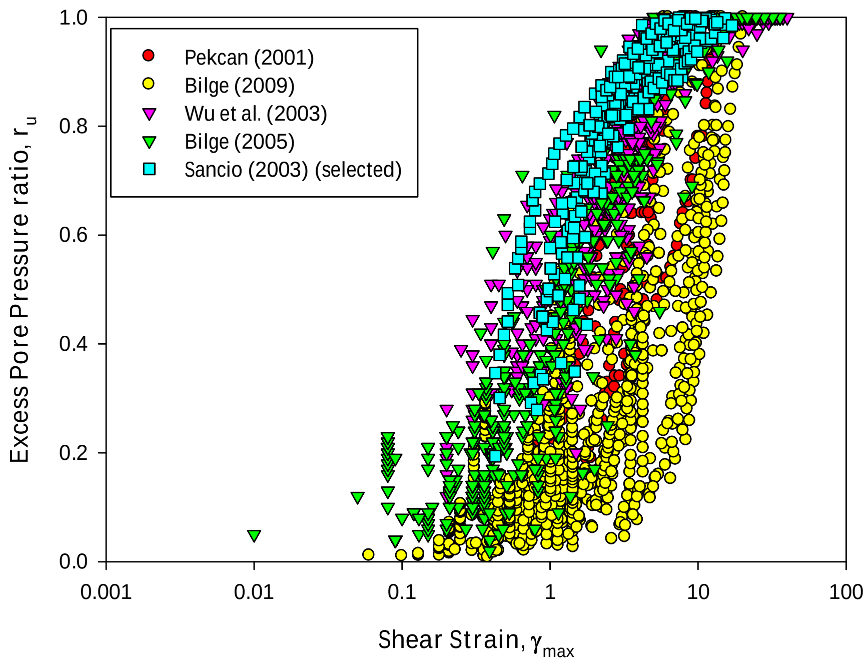

Figure 3.12 Graphical presentation of 15 selected calibrated CTX tests of Sancio (2003) along with previously selected database

3.3 Conclusions

Details of the data sources and data compilation efforts were introduced in this chapter. The compiled database will be used to check the performance of existing liquefaction susceptibility criteria, as well as to develop new criteria. Based on the previous discussions, data of Sancio (2003) remains the only problematic source; however, data of Sancio was used in almost all of the recent criteria (i.e. Seed et al. 2003, Bray and Sancio 2006, Boulanger and Idriss 2006), hence it is decided to include this data in the comparison study. Thus, a total number of 148 cyclic test results are available for evaluation of performance of available liquefaction susceptibility criteria. However, as the quality of the database will directly affect the overall performance of the proposed methodology, all of the data points were carefully studied and filtered out whenever any kind of inconsistency was determined. After filtering out the problematic data of Sancio (2003) and adding up data obtained from cyclic tests performed on clean sands, data from a total number of 158 tests have been compiled for development of correlations for the proposed liquefaction susceptibility criteria. This database represents a variety of soil classes such as SW, SP, CL, CH, CL-ML, ML and M according to USCS. PI values range from 0 to 58.9 along with LL range of 0 to 95, and w_c/LL ratio being between 0 and 1.5. 158 CTX tests provide a total number of 2829 data points for liquefaction susceptibility boundary development as illustrated in Figure 3.14

The compiled database used for comparison and model development studies are presented in Figure 3.13 and Figure 3.14, respectively. Similarly these data points are also located on plasticity chart and presented in Figure 3.15 and Figure 3.16 for databases used for comparison and model development studies, respectively.

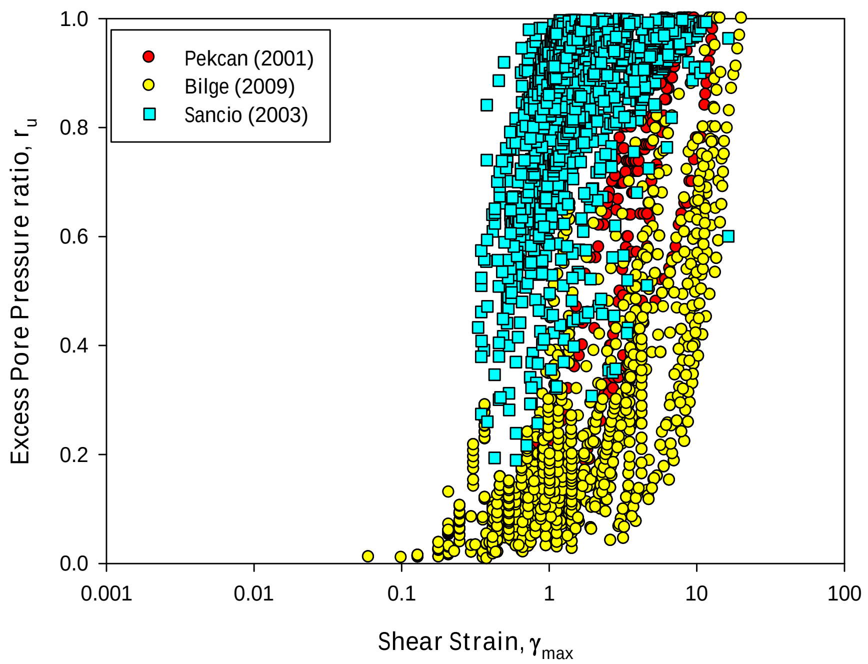

Figure 3.13 Compiled database for comparision of available liquefaction susceptibility criteria.

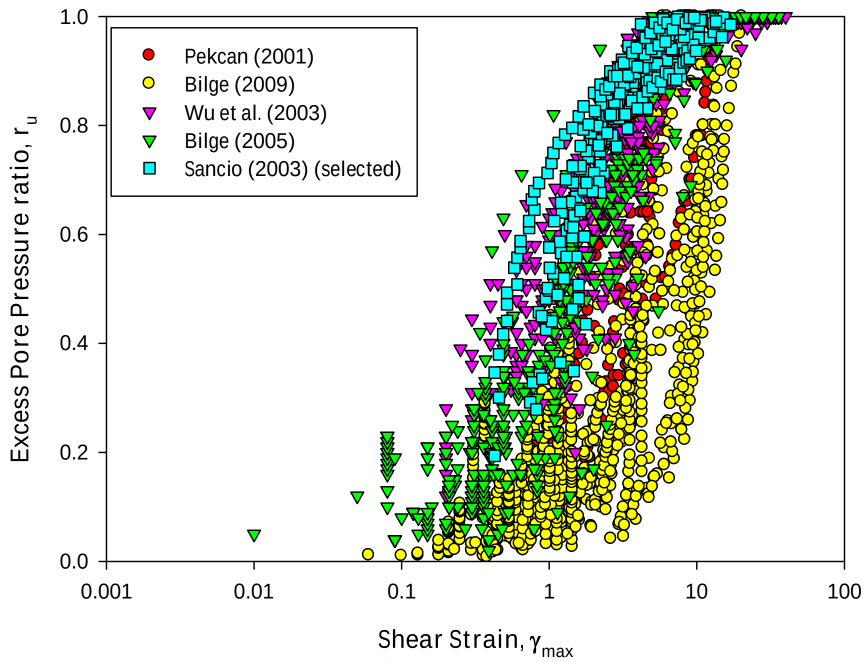

Figure 3.14 Presentation of compiled database for correlation development on r_u vs. γ_{max} domain

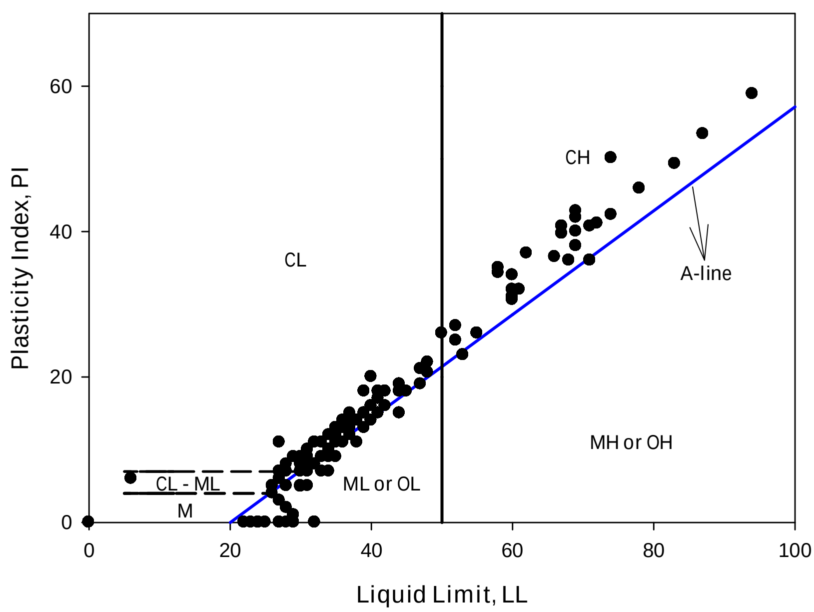

Figure 3.15 Plasticity chart showing 148 specimens constituting compiled database for comparision

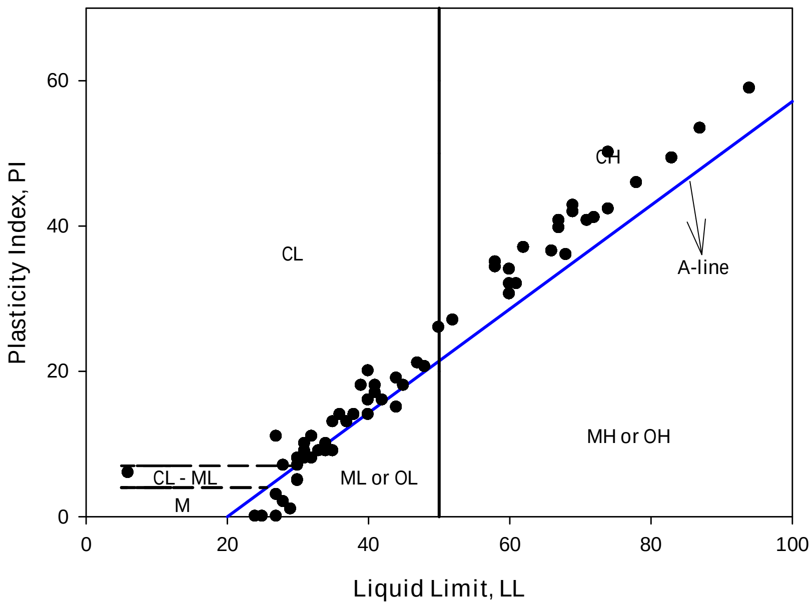

Figure 3.16 Plasticity chart showing 158 specimens constituting compiled database for correlations

CHAPTER 4

EVALUATION OF AVAILABLE LIQUEFACTION SUSCEPTIBILITY CRITERIA WITH COMPILED DATABASE

4.1 Introduction

Shear stresses due to propagation of seismic shear waves induce shear strains leading to compression of soil particles and generation of excess pore water pressures. Rate of shear strain accumulation and pore water pressure generation are functions of soil type, relative state of soil (whether it is contractive or dilative), index properties of soil, existing stress conditions (whether static shear stresses exist or not) and also earthquake-induced shear stresses. Hence theoretically, even clays of high plasticity can generate significant excess pore water pressures ($r_u \approx 1.00$) and accumulate significant levels of shear strains, if the duration of cyclic shearing is long enough. Therefore, any liquefaction susceptibility criteria for fine-grained soils should also take into account the level and amplitude of cyclic shear stresses beside the index properties of soils. However, as mentioned in previous sections, existing studies take into account neither level nor amplitude of cyclic shear stresses. As a general procedure, they define a liquefaction criterion, which is –unfortunately- not unique, and then they evaluate either field observations or test results based on the selected liquefaction criterion.

Considering its importance, existing criteria and corresponding liquefaction definitions need a re-visit.

This chapter focuses on the performance assessment of recently published liquefaction criteria of Seed et al. (2003), Bray and Sancio (2006), Boulanger and Idriss (2004 and 2006) by using different liquefaction definitions proposed based on different combinations of r_u and γ_{max} values. For this purpose, the database introduced in the previous chapter will be used.

4.2 Liquefaction Definitions from Previous Studies

Soil liquefaction phenomena have been recognized for many years and liquefaction has been defined in various ways as briefly overviewed in Cetin (2000). The term, liquefaction, has been used to define both flow failures and cyclic softening which represent fundamentally different soil responses. In simpler terms, as given in Cetin (2000), liquefaction can be defined as the significant reduction in shear strength and stiffness due to increase in pore water pressure. However, it is possible to encounter other definitions some of which will be reviewed in the following sections.

Primary step of developing a liquefaction susceptibility criterion involves defining liquefaction and almost all of the studies used different definitions, the subject of comparison becomes more difficult. Following discussion provides a close view on how these previous susceptibility criteria were developed.

Chinese criteria, as discussed earlier, used the case histories and liquefaction was identified by observable surface manifestations such as sand boils, ground cracking, lateral spreading and ground settlements, etc. Hence, there is no specific condition, which can be expressed in terms of excess pore water pressure ratio or shear strain, to identify occurrence of liquefaction in this study.

Seed et al. (2003) did not clearly state how they defined liquefaction; however they suggested that low plasticity sands and silty sands should be considered to be liquefied when certain level of shear strain (typically on the order of 3% to 6%) is accumulated. Moreover Seed et al. (2003) stated that fine grained materials may also produce large amplitudes of shear strains but it is not always accompanied by generation of significant pore pressures, which is referred as cyclic mobility. It is concluded that Seed et al. (2003) take into account both excess pore pressure ratio and shear strain accumulation while defining liquefaction but they do not state any specific boundary for these parameters.

Bray et al. (2004b) and Bray & Sancio (2006) proposed criteria for liquefaction susceptibility based on cyclic laboratory tests performed on samples taken from liquefied sites of Adapazari. In these companion studies, soils exceeding 3 % single amplitude axial strain (generally in extension) or 5% double amplitude axial strain were accepted to be liquefied. However, the cyclic shear stress levels and duration of cyclic loading which will produce these strain amplitudes were not defined. They did not propose an r_u-based criterion due to difficulties associated with reliable pore water pressure measurements under a loading frequency of 1 Hz.

Boulanger and Idriss (2004, 2006) distinguish the cyclic response of fine-grained soils using terms “sand-like” and “clay-like”. According to Boulanger and Idriss, only “sand-like” fine-grained soils can undergo cyclic liquefaction; whereas for "clay-like" fine-grained soils the governing mechanism is cyclic mobility. This distinction was made based on solely PI of the specimens and the proposed criterion was presented by a chart showing the transition from sand-like behaviour to clay-like behaviour in CSR vs. PI domain. For sand-like soils, initial liquefaction is achieved when excess pore pressure ratio (r_u) becomes equal to 1.0; whereas clay-like soils undergo cyclic mobility when excess pore pressure ratio, $r_u \geq 0.8$. However, the basis of "sand-like" and "clay-like" classification was not discussed.

4.3 Evaluation of previous liquefaction susceptibility criteria with compiled database

In this section, recently published liquefaction susceptibility criteria of Seed et al. (2003), Bray and Sancio (2006) and Idriss and Boulanger (2006) will be evaluated using compiled database based on different definitions of liquefaction, such as; (i) at γ = 3.5% , $r_u \geq$ 0.7, (ii) at γ = 3.5% , $r_u \geq$ 0.8, (iii) at γ = 3.5% , $r_u \geq$ 0.9, (iv) at γ = 3.5% , $r_u \geq$ 1.0, (v) at γ = 5.0% , $r_u \geq$ 0.7, (vi) at γ = 5.0% , $r_u \geq$ 0.8, (vii) at γ = 5.0% , $r_u \geq$ 0.9, (viii) at γ = 5.0% , $r_u \geq$ 1.0, (ix) at γ = 7.5% , $r_u \geq$ 0.7, (x) at γ = 7.5% , $r_u \geq$ 0.8, (xi) at γ = 7.5% , $r_u \geq$ 0.9, (xii) at γ = 7.5% , $r_u \geq$ 1.0, where γ and r_u represent double amplitude shear strain and excess pore pressure ratio, respectively. However, in this chapter results are presented only for liquefaction definition (vi), i.e. if at shear strain levels $\gamma_{max} \leq$ 5.0%, excess pore pressure ratio r_u exceeds the value of 0.8, then it is classified as liquefied, which is considered as a critical combination and the rest of this comparison study, for other liquefaction definitions, is presented in Appendix A. It should be noted that liquefaction definitions used in previous studies were based on either accumulated strain level or generated excess pore pressure ratio. On the other hand, this study considers both of these conditions together, because sometimes fine-grained soils accumulate significant shear strains, without producing sufficient amounts of excess pore pressure. In literature, this condition is generally referred as “cyclic mobility” rather than “cyclic liquefaction”.

4.3.1 Seed et al. (2003)

The criteria proposed by Seed et al. (2003) state that soils are susceptible to liquefaction if they satisfy all following three conditions, (i) $PI < 12$, (ii) $LL < 37$ and (iii) $w_c/LL > 0.80$; whereas, soils satisfying the following three conditions, (i) $12 < PI < 20$, (ii) $37 < LL < 47$, (iii) $w_c/LL > 0.85$ are considered to need further testing to be classified as potentially liquefiable. Soils not satisfying any of these conditions are considered to be non-liquefiable according to these criteria. The performance of Seed et al.’s criteria is evaluated by using the listed liquefaction definitions. Figure 4.1 illustrates liquefaction boundaries proposed by Seed et al. (2003) along with compiled database evaluated for the selected liquefaction definition, i.e. γ=5.0% and $r_u \geq 0.8$. The model performance is summarized also in Table 4.1 and model error is found to be 19.6 %. The error was calculated by simply dividing total number of points that cannot be successfully predicted by criteria (summation of third column of first and first column of second row) to total number of points. The points located at Zone B, i.e. when further testing is required, were considered to be successfully predicted the criteria.

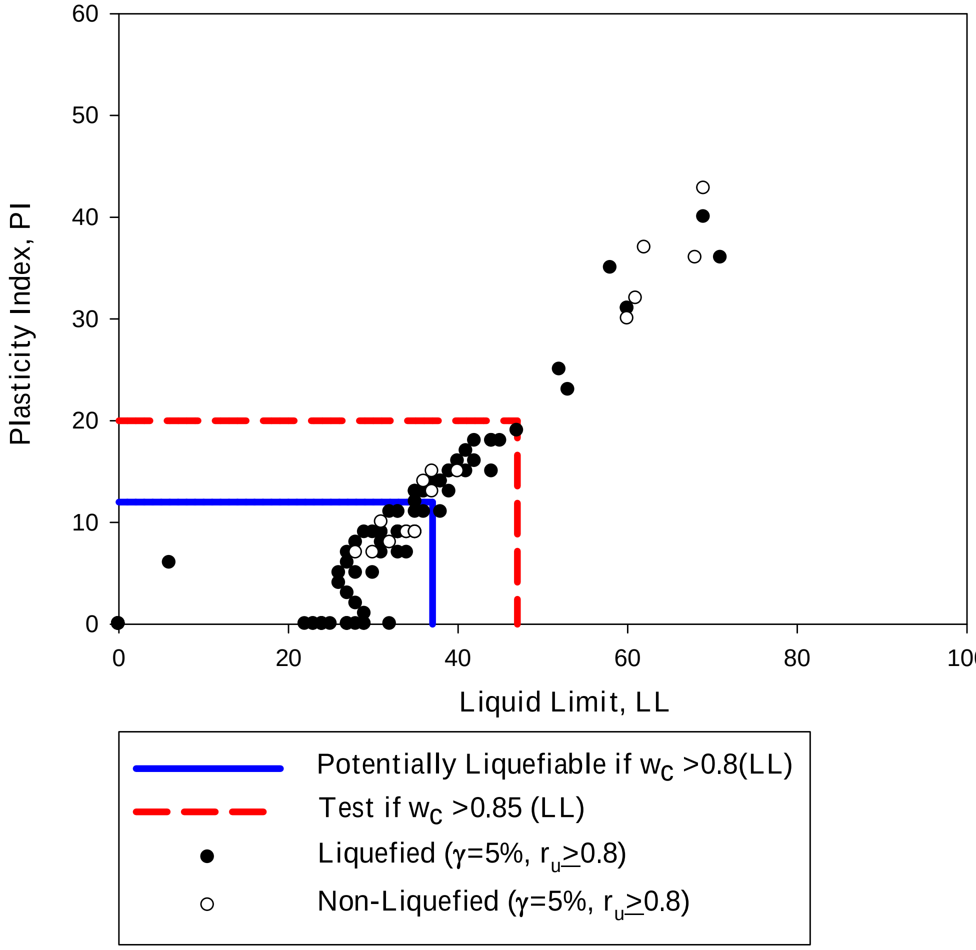

Figure 4.1 Evaluation for liquefaction susceptibility condition (γ = 5.0% , $r_u \geq$ 0.8 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table 4.1 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 5 %, ru≥0.8 .

Condition: γ=5 %, $r_u \geq$ 0.8			
Seed et al. (2003) / Observed	YES	TEST	NO
YES	44	21	8
NO	11	4	9

4.3.2 Bray and Sancio (2006)

Bray and Sancio (2006) proposed liquefaction susceptibility criteria using the cyclic laboratory tests performed on samples obtained from liquefied sites of Adapazari. According to this criteria, fine-grained soils with PI < 12 and w_c/LL >0.85 are considered to be susceptible to liquefaction; whereas fine-grained soils with 12 < PI < 18 and w_c/LL >0.80 need further testing to assess their potential of liquefaction. Fine-grained soils that do not satisfy either of these criteria are accepted to be non-liquefiable. Bray and Sancio defined liquefaction as the accumulation of either 3 % single amplitude axial strain or 5 % double amplitude axial strain. The performance of the proposed criteria was evaluated by the compiled database by γ= 5.0% and $r_u \geq 0.8$ liquefaction definition, and the results are presented in Figure 4.2. The model error of Bray and Sancio's criteria was calculated as 19.6 % which is equal to the error estimated for Seed et al. (2003) criteria.

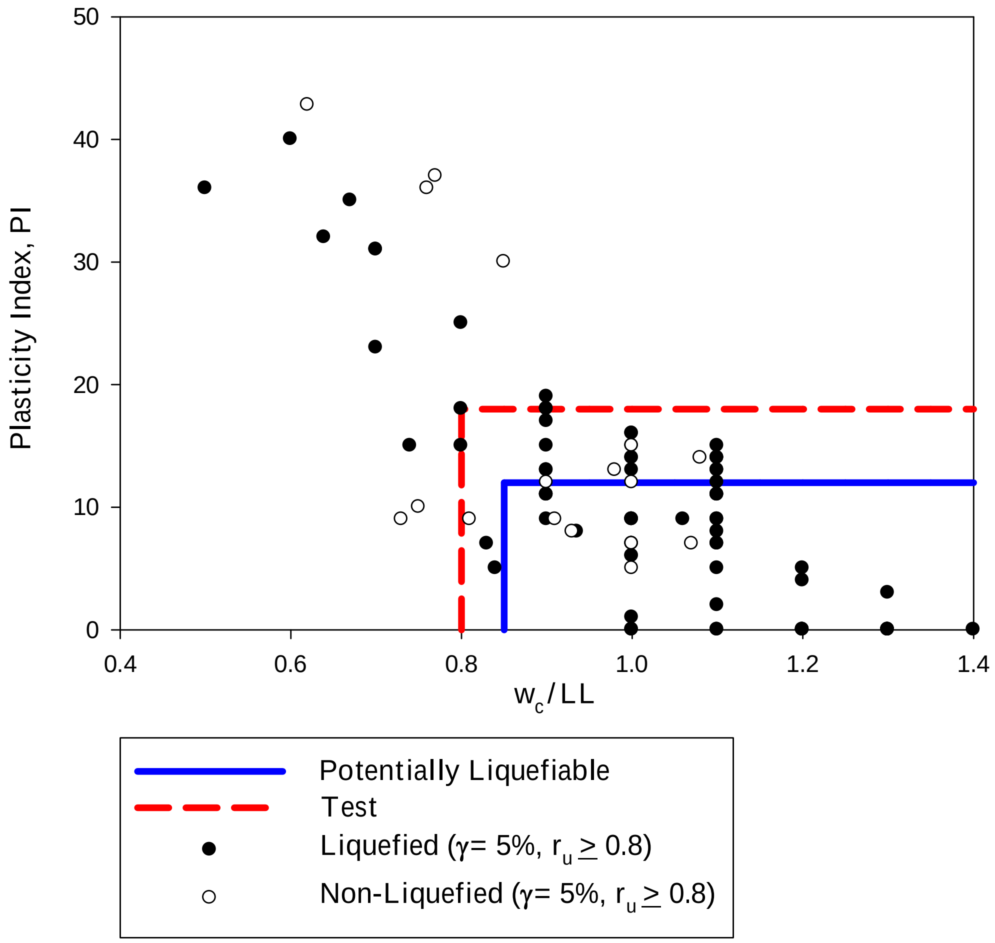

Figure 4.2 Evaluation for liquefaction susceptibility condition (γ = 5.0% , $r_u \geq$ 0.8 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table 4.2 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 5%, ru$\geq$ 0.8 .

Condition: γ=5 %, $r_u \geq$ 0.8			
Bray & Sancio (2006) / Observed	YES	TEST	NO
YES	43	20	10
NO	9	5	10

4.3.3 Boulanger and Idriss (2004, 2006)

The criterion of Boulanger and Idriss (2004, 2006) is based on PI and according to authors; fine-grained soils having PI $\leq$ 3 are named as "sand-like" and they can exhibit "cyclic liquefaction" type response; whereas, for fine-grained soils having PI $\geq$ 7 are named as "clay-like" and they are expected to exhibit "cyclic mobility" type response. In between these PI ranges, i.e. 3 to 7, a transition is expected from sand-like behaviour to clay-like behaviour. Authors presented these criteria in the CSR vs. PI domain without a scale. Thus, comparison of this criterion with current database was performed by considering only the PI values. Figure 4.3 presents the performance evaluation of these criteria for liquefaction definition of γ= 5%, ru $\geq$ 0.8. This figure shows that this criterion failed to separate liquefiable and non-liquefiable soils. Table 4.3 summarizes this figure numerically. The model error is calculated to be 45.1 % which is two times more than the errors calculated for the other criteria. This poor performance is mostly due to using a single parameter, i.e. PI, and it is an indication of necessity of another parameter as criterion to evaluate liquefaction susceptibility of fine-grained soils.

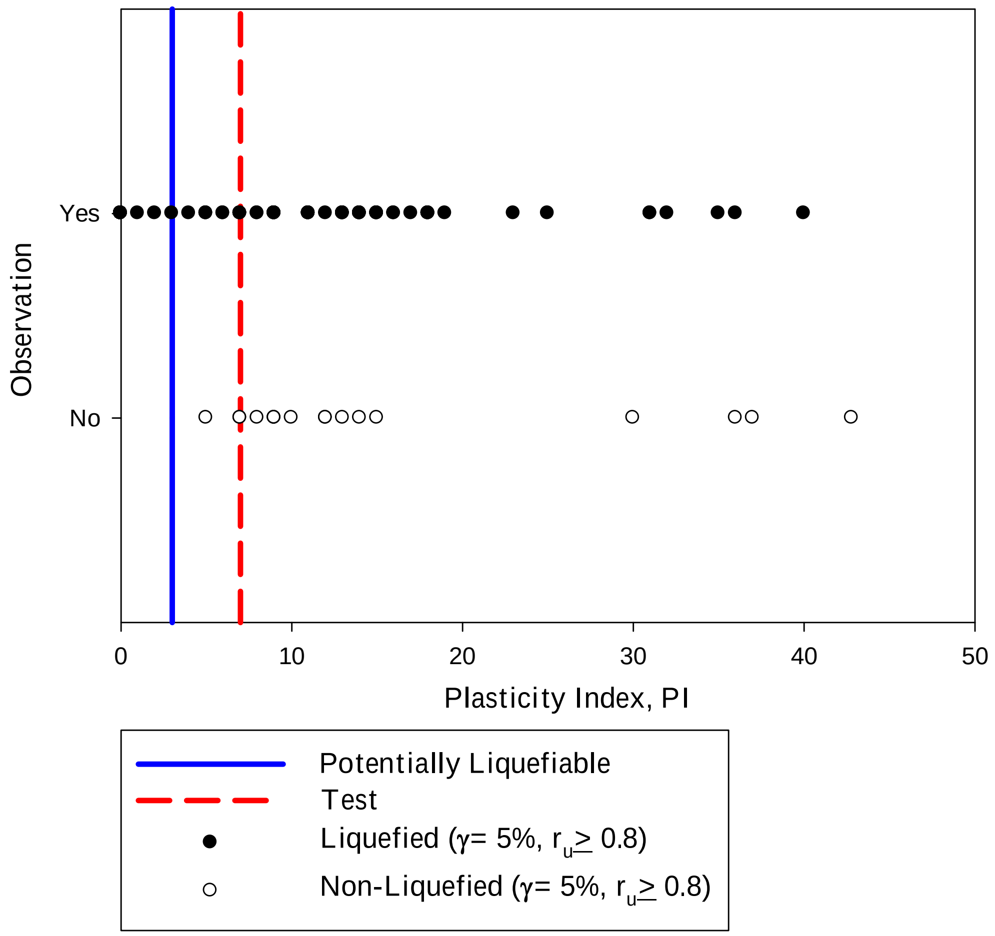

Figure 4.3 Compiled database evaluated for liquefaction (γ = 5.0%, $r_u \geq$ 0.8 condition) plotted on Boulanger and Idriss (2004, 2006) liquefaction susceptibility criteria

Table 4.3 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004, 2006) with compiled database for liquefaction condition γ= 5%, ru≥ 0.8 .

Condition: γ= 5%, r_u≥ 0.8			
Boulanger & Idriss (2004, 2006) / Observed	YES	TEST	NO
YES	22	10	46
NO	0	4	20

4.4 Conclusion

This chapter focused on the performance of previous liquefaction susceptibility criteria using the compiled database. After reviewing liquefaction definitions used in these studies, 12 alternative definitions were proposed and the available criteria were tested considering all of these definitions. The condition of γ= 5.0% and r_u≥ 0.8 is accepted as a critical combination and the calculated results based on this definition were presented in this chapter; whereas, rest of the comparison study is given in Appendix A. The results of this study revealed that single parameter-based susceptibility criteria, i.e. Boulanger and Idriss (2004, 2006), cannot successfully identify fine-grained soils susceptible to liquefaction and pointed out the importance of w_c/ LL ratio as a screening tool. Table 4.4 summarizes the error terms calculated for each criterion considering 12 different conditions defining occurrence of liquefaction. Error margins observed from this table suggested that available criteria still need improvement and the following chapters of this thesis will be devoted to the development of new criteria.

Table 4.4 Error terms associated with available liquefaction susceptibility criteria for different definitions of liquefaction evaluated with different database

Criteria	Reference	γ (%)	r_u	YY+NN+TN+TY	YN+NY	Error (%)	Criteria	Reference	γ (%)	r_u	YY+NN+TN+TY	YN+NY	Error (%)	Criteria	Reference	γ (%)	r_u	YY+NN+TN+TY	YN+NY	Error (%)
Seed et al. (2003)	Bilge (2009) and Pekcan (2001)	3.50	0.7	21	7	25.00	Bray & Sancio (2006)	Bilge (2009) and Pekcan (2001)	3.50	0.7	22	6	21.43	Boulanger & Idriss (2004, 2006)	Bilge (2009) and Pekcan (2001)	3.50	0.7	27	1	3.57
			0.8	18	10	35.71				0.8	21	7	25.00				0.8	27	1	3.57
			0.9	17	11	39.29				0.9	20	8	28.57				0.9	27	1	3.57
			1	17	11	39.29				1	20	8	28.57				1	27	1	3.57
	Sancio (2003)		0.7	55	16	22.54		Sancio (2003)		0.7	54	17	23.94		Sancio (2003)		0.7	35	41	53.95
			0.8	51	19	27.14				0.8	50	20	28.57				0.8	33	42	56.00
			0.9	40	30	42.86				0.9	41	29	41.43				0.9	45	30	40.00
			1	29	44	60.27				1	29	44	60.27				1	57	21	26.92
	All		0.7	76	23	23.23		All		0.7	76	23	23.23		All		0.7	62	42	40.38
			0.8	69	29	29.59				0.8	71	27	27.55				0.8	60	43	41.75
			0.9	57	41	41.84				0.9	61	37	37.76				0.9	72	31	30.10
			1	46	55	54.46				1	49	52	51.49				1	84	22	20.75
	Bilge (2009) and Pekcan (2001)	5.00	0.7	15	9	37.50		Bilge (2009) and Pekcan (2001)	5.00	0.7	15	9	37.50		Bilge (2009) and Pekcan (2001)	5.00	0.7	20	4	16.67
			0.8	16	8	33.33				0.8	17	7	29.17				0.8	22	2	8.33
			0.9	15	9	37.50				0.9	16	8	33.33				0.9	24	0	0.00
			1	13	11	45.83				1	16	8	33.33				1	23	1	4.17
	Sancio (2003)		0.7	65	8	10.96		Sancio (2003)		0.7	64	9	12.33		Sancio (2003)		0.7	32	46	58.97
			0.8	62	11	15.07				0.8	61	12	16.44				0.8	34	44	56.41
			0.9	47	25	34.72				0.9	45	27	37.50				0.9	36	41	53.25
			1	30	43	58.90				1	30	43	58.90				1	56	22	28.21
	All		0.7	80	17	17.53		All		0.7	79	18	18.56		All		0.7	52	50	49.02
			0.8	78	19	19.59				0.8	78	19	19.59				0.8	56	46	45.10
			0.9	62	34	35.42				0.9	61	35	36.46				0.9	60	41	40.59
			1	43	54	55.67				1	46	51	52.58				1	79	23	22.55
	Bilge (2009) and Pekcan (2001)	7.50	0.7	13	8	38.10		Bilge (2009) and Pekcan (2001)	7.50	0.7	13	8	38.10		Bilge (2009) and Pekcan (2001)	7.50	0.7	15	6	28.57
			0.8	13	8	38.10				0.8	13	8	38.10				0.8	15	6	28.57
			0.9	12	6	33.33				0.9	11	5	31.25				0.9	15	3	16.67
			1	9	8	47.06				1	10	7	41.18				1	16	1	5.88
	Sancio (2003)		0.7	66	7	9.59		Sancio (2003)		0.7	65	8	10.96		Sancio (2003)		0.7	31	47	60.26
			0.8	64	9	12.33				0.8	63	10	13.70				0.8	32	46	58.97
			0.9	60	12	16.67				0.9	59	13	18.06				0.9	31	46	59.74
			1	40	33	45.21				1	41	32	43.84				1	51	27	34.62
	All		0.7	79	15	15.96		All		0.7	76	16	17.39		All		0.7	46	53	53.54
			0.8	77	17	18.09				0.8	72	18	20.00				0.8	47	52	52.53
			0.9	72	18	20.00				0.9	46	18	28.13				0.9	46	49	51.58
			1	49	41	45.56				1	51	39	43.33				1	67	28	29.47

CHAPTER 5

PROPOSED APPROACH FOR THE ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY OF FINE GRAINED SOILS

5.1 Introduction

The common approach followed by the previous liquefaction susceptibility criteria associate liquefaction potential of fine-grained soils with Atterberg limits and natural moisture content. However, as discussed in previous chapters, these criteria are valid for only a limited number of conditions, i.e. their validity depend on the selected liquefaction definition. Moreover, none of the previous criteria takes into account the amplitude of cyclic loading which is considered as a controlling parameter. Similarly, the difference between cyclic liquefaction and cyclic mobility type responses are not considered except the criteria of Boulanger and Idriss (2004, 2006) which yields the least accurate predictions as revealed by the performance evaluation study (Chapter 4). Based on these discussions, it is obvious that present liquefaction susceptibility criteria need improvement and these improved criteria should; i) take into account the cyclic shearing level, ii) distinguish governing soil response, i.e. “cyclic liquefaction” and “cyclic mobility” type responses, and iii) be flexible enough to be used for any liquefaction definition. Hence, it is aimed to consider all these listed needs while developing the new liquefaction susceptibility criteria. This chapter is devoted to introduce the theoretical background of the proposed approach before giving details of it.

5.2 Background Information

The pioneer studies on the cyclic behaviour of fine-grained soils have focused on stiffness degradation under dynamic loading conditions (e.g. Thiers and Seed 1968, Hardin and Drnevich 1972, Castro and Christian 1976, Idriss et al. 1978). In time, with improvements in reliability of cyclic testing, various researches have also focused on this subject and performed valuable studies (e.g. Ishihara 1986, Sun et al. 1988, Vucetic and Dobry 1991, Ishibashi and Zhang 1993, Darendeli 2001). In fact, none of these studies mentioned occurrence of liquefaction; however, due to the deficiencies of existing susceptibility criteria, it is considered that these studies may be helpful while developing the proposed criteria.

Cyclic stress-strain characteristics of soils are usually defined through; (i) G_{max}, the value of shear modulus at small strains, (ii) the relation between secant modulus (G) and the cyclic shear strain amplitude (γ_c) which is generally expressed in G/G_{max} vs. γ_c domain, (iii) the relation between damping ratio (λ) and γ_c, and (iv) the degradation of G after N cycles of γ_c. Vucetic and Dobry (1991) summarized previous studies performed on this subject and proposed shear modulus degradation curves as a function of PI, which is considered to be the most important controlling parameter of dynamic behaviour of cohesive soils. The relations proposed in Figure 5.1 suggest that i) with increasing PI, soils are more resistant to cyclic loading and modulus degradation takes places in a slower rate, ii) soils of high plasticity exhibit a more linear response compared to non-plastic soils, iii) the strain level required to pass non-linear response range is in order of 10^{-5} for non-plastic soils; whereas, this level is around 10^{-4}.

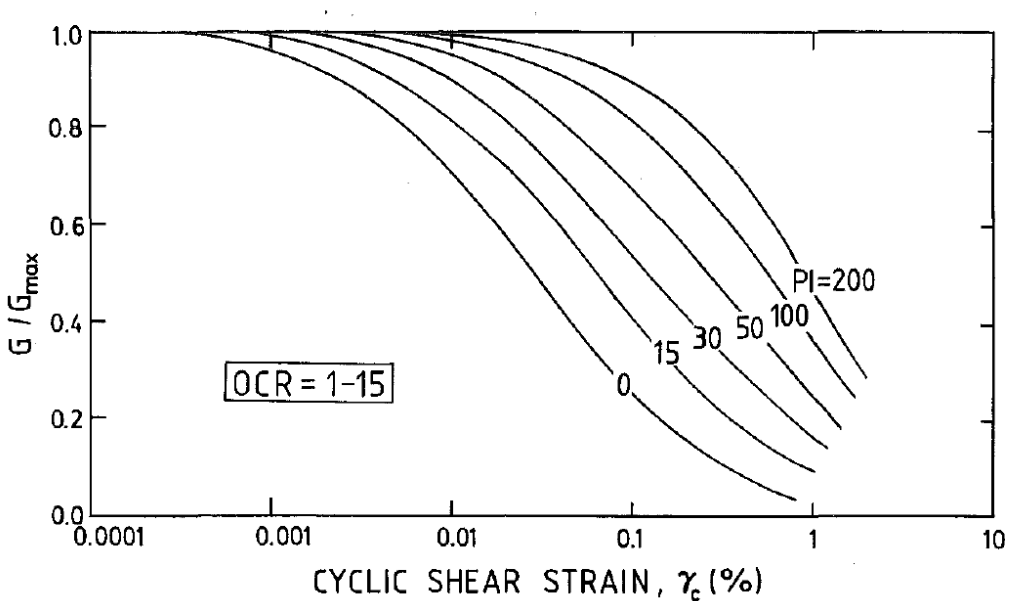

Figure 5.1 Relation between G/G_{max} and γ_c curves and soil plasticity for normally and overconsolidated soils. (Vucetic and Dobry, 1991)

As mentioned in Cetin and Bilge (2009) the elevated pore pressures due to rearrangement of soil particles resulted in reduction of effective confining stress-dependent soil stiffness. It triggers the vicious cycle of further strain and pore pressure generation. In the extreme, the excess pore water pressure may approach total stress, defining the on-set of cyclic soil liquefaction.

The strength degradation behaviour of soils under cyclic loading is generally associated with either (i) excess pore water pressure build-up with a reduction in effective stress or (ii) remoulding of soils caused by maximum strain level.

The effect of these components have been considered by the pioneer study of Hardin and Drnevich (1972) in which modulus degradation was defined as a function of shear strain, effective stress state (i.e. excess pore water pressure) and shear strength parameters as follows;

$$\frac{G}{G_{max}} = \frac{1}{1+\dfrac{\gamma}{\gamma_r}} \tag{5.1}$$

where γ_r is reference shear strain and is defined as:

$$\gamma_r = \frac{\tau_{max}}{G_{max}} \tag{5.2}$$

where τ_{max} is the shear stress at failure. The value of τ_{max} depends on the initial state of stress in the soil and the way in which the shear stress is applied and it is defined as:

$$\tau_{max} = \left\{ \left(\frac{1+K_0}{2} \cdot \sigma'_v \cdot sin\phi' + c' \cdot cos\phi' \right)^2 - \left(\frac{1-K_0}{2} \cdot \sigma'_v \right)^2 \right\}^{1/2} \tag{5.3}$$

where K_0 is the coefficient of lateral stress at rest and σ'_v is the vertical effective stress.

The results of this study have inspired many succeeding efforts and are important for this study, as they clearly implied the significance of both shear strain accumulation and excess pore water pressure generation on cyclic response of soils.

Being inspired from study of Hardin and Drnevich, Seed et al. (1986) suggest Equation (5.4) which defines shear modulus as a function of confining stress and a coefficient representing soil properties.

$$G = 2000K_2\sqrt{\sigma'_m} \tag{5.4}$$

G is the shear modulus in terms of kPa, K_2 is the shear modulus coefficient which is mainly a function of particle size, relative density and accumulated shear strain in soil and σ'_m is the effective mean principal stress in psf. The effective mean principal stress, shown in Equation (5.4), changes based on generation of excess pore pressure. Moreover, excess pore pressure is due to applied cyclic loading which also causes accumulation of shear strains. For certain level of shear modulus, G, K_2 value corresponding to each strain level is a function of accumulated shear strain and it represents the remoulding of soil. Thus, based on relation proposed by Seed et al. (1986), it can be inferred that remoulding of soil and generation of excess pore pressure are interrelated.

5.3 Proposed Approach

This study aims to provide a new unified system for the determination of liquefaction potential of fine grained soils which (i) is valid for all liquefaction definitions, (ii) considers the amplitude of cyclic loading and (iii) differentiates cyclic liquefaction and cyclic mobility type soil responses. Hence, it is an obvious need to find a framework satisfactorily covering all these issues. Based on the discussion presented in the previous section, the relation between r_u and γ_{max} is considered to be useful for this purpose. Moreover, the screening tools of current criteria i.e. PI and w_c/LL, will also be implemented into the proposed framework as they have a significant effect on cyclic response.

Based on the modulus degradation curves proposed by Vucetic and Dobry (1991) and experimental observations, the relation between r_u and γ_{max} is expected to vary as a function of PI as shown in Figure 5.2. The most significant effect of PI is that increasing PI values also increase the threshold strain levels separating different phases of material behaviour, e.g. linear and nonlinear elastic and elastoplastic responses.

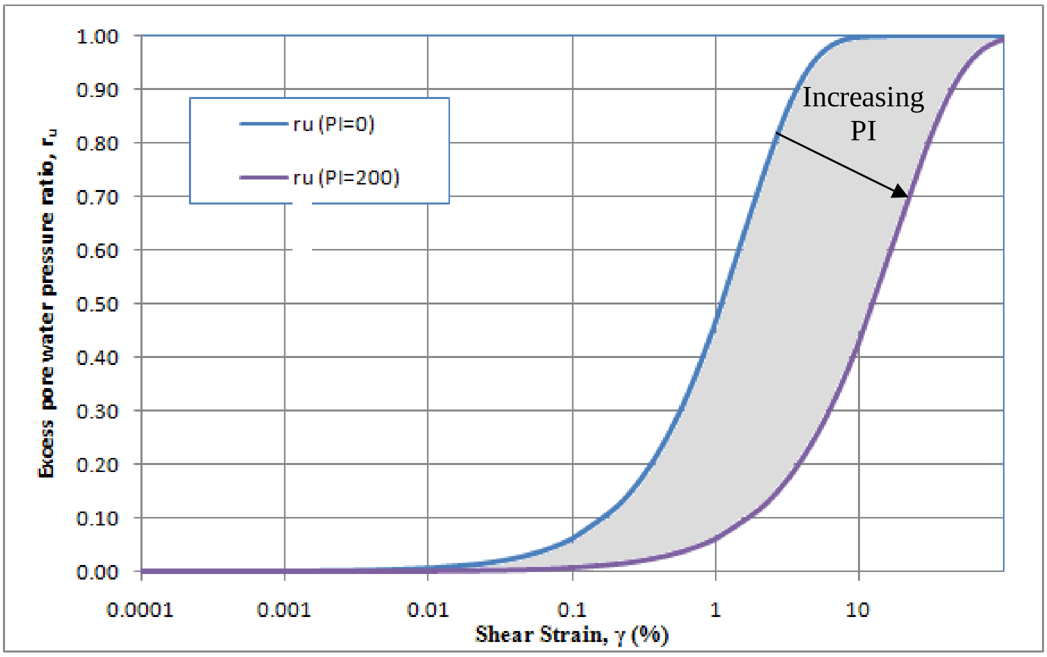

Figure 5.2 Sensitivity of excess pore pressure response to plasticity index.

Increasing excess pore pressure leads to decrease in mean effective stress term of Equation (5.4) which should be compensated by an increase in modulus coefficient K_2 which corresponds to an increase in shear strain. Previous researchers discussed the difference between excess pore pressure generation behaviour of cohesive and cohesionless soils based on modulus degradation such that rather than the excess pore pressure generation, cyclic loading of high plasticity soils are generally counterbalanced by shear strain accumulation up to a threshold value after which a sharp increase in excess pore pressure is observed. Whereas for cohesionless soils a smoother increase in excess pore pressure is observed in smaller shear strain levels. This difference in behaviour is mostly controlled by plasticity index of soils; therefore a shift towards the right is expected with increasing plasticity index on r_u vs. γ_{max} domain as shown in Figure 5.2.

Figure 5.3shows the compiled database on r_u vs. γ_{max} domain which is consistent with the approach proposed in this study. As plasticity index increases, the data pairs shift towards right side of the figure. Additionally, as discussed previously, for soils with higher plasticity, generation of excess pore pressures starts at relatively higher shear strain levels.

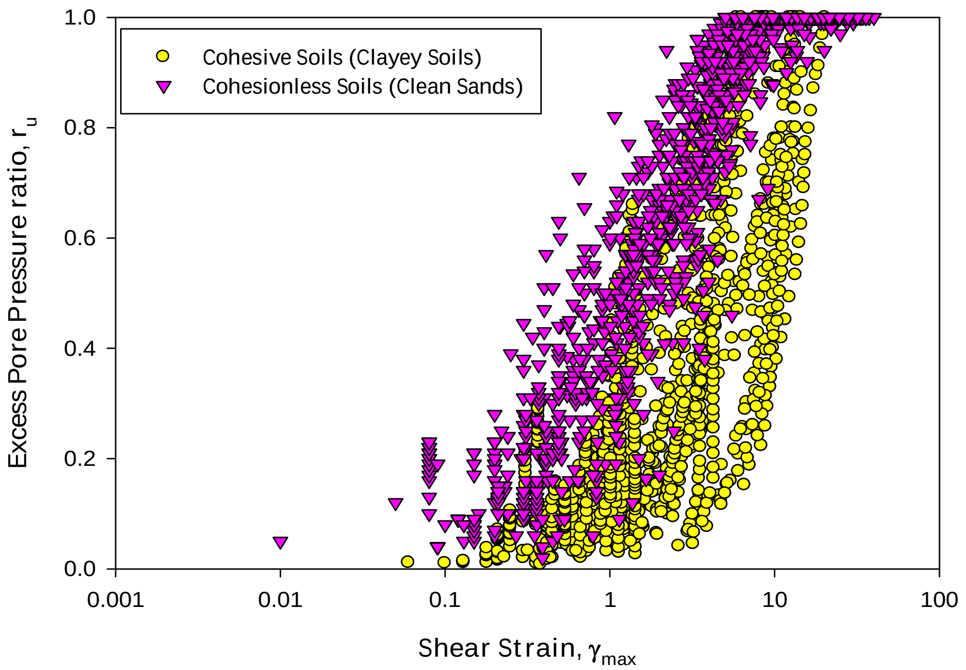

Figure 5.3 Compiled database presented on r_u vs. γ_{max} domain

5.4 Conclusions

Liquefaction susceptibility criteria proposed previously in literature suffers from the limitations that arise from (i) the conditions defining liquefaction, (ii) the ambiguity in loading amplitudes for which they were defined and (iii) deficiencies in differentiating cyclic liquefaction from cyclic mobility type soil responses. Additionally, it is shown that domains used in available studies cannot satisfactorily capture trends separating potentially liquefiable soils from non-liquefiable soils. Thus, an attempt is made to develop new liquefaction susceptibility criteria using a new domain. Results of studies on modulus degradation response of fine-grained soils were re-examined to find a basis for this attempt.

Using the modulus degradation curves of Vucetic and Dobry (1991) along with the model proposed by Seed et al. (1986), the non-linear relation between r_u and γ_{max} as a function of PI was identified (Figure 5.2). Considering its mechanical basis and the facts that it i) is valid for any liquefaction definition, ii) considers the level of cyclic stresses and iii) differentiates the cyclic liquefaction and cyclic mobility type soil responses, r_u vs. γ_{max} relationship was adopted as the working domain in this study.

CHAPTER 6

DEVELOPMENT OF PROBABILISTICALLY-BASED PORE WATER PRESSURE GENERATION MODELS

6.1 Introduction

This chapter presents the development of probabilistically-based excess pore water pressure generation models, which will be further used in the development of proposed liquefaction susceptibility criteria suitable for fine grained soils. Previous studies provide criteria based on index parameters and natural water content in order to evaluate liquefaction triggering potential consistent with definition adopted for the development of the criteria. Considering their limitations, which have been already discussed in this thesis (Chapters 2 and 3), an alternative framework is developed. It is aimed that the proposed framework will be applicable regardless of the selected liquefaction definition and will take into account the effects of amplitude and duration of loading. For this reason, excess pore water pressure ratio vs. cyclic shear strain domain has been selected considering its ability to explain the soil response as discussed in the previous chapter. Probabilistically-based correlations providing a unified system for the estimation of r_u corresponding to any shear strain level is developed as a part of this thesis. Proposed model is defined as a function of cyclic shear strain and soil index and state parameters, such as, (i) plasticity index (PI), (ii) liquid limit (LL) and (iii) water content to liquid limit ratio (w_c/LL).

The following sections of this chapter present detailed information on limit state models, development of likelihood functions and estimation of model coefficients by the maximum likelihood methodology. Then, new liquefaction susceptibility criteria will be proposed based on developed likelihood function.

6.2 Limit State Models

The compiled database consists of results from 158 cyclic triaxial test results. Excess pore pressure ratios, corresponding to the shear strains accumulated at each uniform cycle, were plotted on r_u vs. γ_{max} domain as shown in Figure 6.1. Considering the discussions in Chapter 5, i) a linear behaviour followed by a nonlinear increase in excess pore pressure ratio (r_u) was observed with increasing double amplitude maximum cyclic shear strain (γ_{max}), ii) a nonlinear trend, varying based on soil index and state parameters, exist between r_u and γ_{max}, iii) a downward shift is noticed with increasing plasticity. Therefore, an attempt is made to develop a probabilistic model representing this behaviour. In this regard, maximum likelihood principle was employed for development of correlations.

Selecting a limit state model, which captures the previously stated important features of the observed behaviour, is the first step for the development of a probabilistic model. The model for the limit state function has the general form g = g ($\mathbf{x}, \mathbf{\Theta}$) where x is a set of descriptive parameters and Θ is the set of unknown model coefficients. Limit state function is defined as the difference (error) between the natural logarithms of the estimated and observed excess pore pressure ratios as illustrated in Equation (6.1).

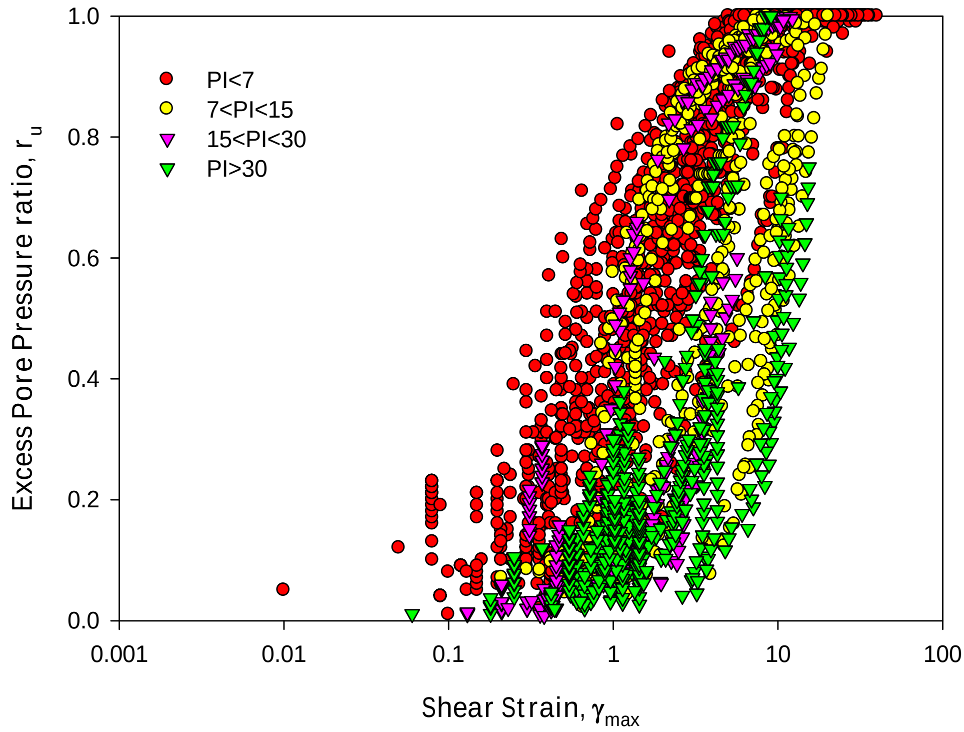

Figure 6.1 Compiled database for development of limit state models on r_u vs. γ_{max} domain

$$g_{r_u}\left(r_{u,obs}, r_{u,est}\right) = ln(r_{u,obs}) - ln(r_{u,est}) \mp \varepsilon_{\ln(r_u)} \qquad (6.1)$$

Equation 6.1 involves a random model correction term (ε) to account for the possible deficiencies due to i) missing descriptive variables; and ii) the adopted mathematical expression, which may not have the ideal functional form. It is reasonable and also convenient to assume that ε follows a normal distribution with a mean of zero for the aim of producing an unbiased model (i.e., one that on the average makes correct predictions). The standard deviation of ε, denoted as σ_ε, however is unknown and must be estimated.

Inspired by the trends observed from test results and previous studies, estimated excess pore pressure ratio ($r_{u,\ estimated}$) was defined by three different functional forms which have different combinations of descriptive variables. The descriptive variables were estimated key parameters affecting the excess pore water pressure generation response of fine-grained soils, which are decided to be double amplitude maximum cyclic shear strain (γ_{max}), plasticity index (PI), liquid limit (LL) and water content to liquid limit ratio (w_c/LL). Three different set of descriptive variables, used for the development of likelihood functions, are; (i) γ_{max} and PI, (ii) γ_{max}, PI and w_c/LL and (iii) γ_{max}, PI, LL and w_c/LL. These functions were composed of both descriptive variables and unknown model coefficients (θ). The set of unknown coefficients of the model, therefore, is $\Theta = (\theta, \sigma_\varepsilon)$.

6.3 Likelihood Function

The excess pore pressure ratios corresponding to each double amplitude shear strain level, of the compiled database, was considered to be independent and the likelihood function was developed for each ‘n’ case as a product of the probabilities of the observations, as presented in Equation (6.2).

$$L_{ru}(\theta,\sigma_\varepsilon)=\prod_{i=1}^{n} P\left[g_{ru}(r_{u,estimated,i},\theta,\sigma_\varepsilon)=0\right] \quad (6.2)$$

Likelihood function given in Equation (6.2) can be further modified by assuming that $(r_u)_i$, $(PI)_i$, $(LL)_i$, $(w_c/LL)_i$ and $(\gamma_{max,N})_i$ values of the ith test are exact (i.e.: no measurement error), and statistically independent, noting that $g(...)=\hat{g}(...)+\varepsilon_i$ has the normal distribution with mean $\hat{g}$ and standard deviation σ_ε. Modified likelihood function, presented in Equation (6.3), is a function of unknown coefficients.

$$L_{ru}(\theta,\sigma_\varepsilon)=\prod_{i=1}^{n} \varphi\left[\frac{g_{ru}(r_{u,observed,i},r_{u,estimated,i},\theta)}{\sigma_\varepsilon}\right] \quad (6.3)$$

In above equation $\varphi[.]$ designates the standard normal probability density function considering the possible sampling disparity problem as pointed out briefly in previous sections. According to the maximum likelihood principle, the set of unknown parameters, $\Theta=(\theta, \sigma_\varepsilon)$, should be estimated in order to maximize the likelihood equation, given as Equation (6.3).

The number of points representing the excess pore pressure ratio, corresponding to a double amplitude shear strain, of cohesionless and cohesive soils was not equal. Thus, a bias was introduced to system. To correct against this sampling disparity problem, the likelihood function for j and k number of tests performed on coarse- and fine-grained soil specimens, respectively, can be written for the selected limit state function as shown by Equation (6.4).

$$L_{r_u}(\boldsymbol{\theta},\sigma_\varepsilon) = \prod_{i=1}^{j}\{\varphi[\cdot]\}^{w_{coarse-grained}} \times \prod_{i=j+1}^{j+k}\{\varphi[\cdot]\}^{w_{fine-grained}} \tag{6.4}$$

Number of points representing behaviour of cohesionless soils ($n_{coarse-grained}$) are 1328, whereas number of points representing behaviour of fine-grained soils ($n_{fine-grained}$) are 1141. The weighting factors were applied are consistent with the available literature (Manski and Lerman (1977), Hsieh et al. (1985), Cetin et al. (2002)). In the estimation of these weighting factors, most attention was given not to increase the database size or available data information (i.e. likelihood information) artificially through weighting factors. Thus, it was preferred simultaneously to down-weight the over-represented and up-weight the under-represented.

The ratio of the weighting factors that will be applied to coarse and fine-grained soils ($w_{coarse-grained}$ and $w_{fine-grained}$, respectively) should be inversely proportional with the ratio of the number of points of each soil type, Equation (6.5). Additionally, the sum of these weighting factors should be equal to 2 in order to prevent development of any artificial data, Equation (6.6).

$$\frac{w_{coarse-grained}}{w_{fine-grained}} = \frac{n_{fine-grained}}{n_{coarse-grained}} \tag{6.5}$$

$$w_{coarse-grained} + w_{fine-grained} = 2 \tag{6.6}$$

Weighting factors for cohesionless and fine grained soils were estimated as 0.9 and 1.1, respectively, by solving Equations (6.5) and (6.6) together. A detailed discussion on how to assess databases with sampling disparity problems and weighting factors is available at Manski and Lerman (1977) and Cetin et al. (2002).

6.4 Development of Probabilistically-based Pore Water Pressure Generation Models

This study aims to provide a unified system which is applicable to both cohesionless and cohesive soils. In this regard, the correlations developed within the confines of this study attempt to represent any kind of soil behaviour on r_u vs. γ_{max} domain. Maximum likelihood principle, which is discussed in the previous section, was employed for the development of these correlations.

In the previous chapters, parameters affecting the behaviour of soils under cyclic loading were discussed in detail. Almost all previous researches agree that plasticity index (PI) has the major role on the behaviour of soil under cyclic loading. However, there is not such an agreement concerning other parameters such as liquid limit (LL) and water content to liquid limit ratio (w_c/LL). Thus, mathematical models were generated by considering three different sets of descriptive variables: (i) γ_{max} and PI, (ii) γ_{max}, PI and w_c/LL and (iii) γ_{max}, PI, LL and w_c/LL.

The mathematical formulation from Cetin and Bilge (2009) is adopted as a valid limit state function alternative. In their recent work, authors have focused on excess pore water pressure generation response of saturated clean sands and proposed the following model as a function of relative density (D_R), initial vertical effective stress (σ'_{v0}), double amplitude maximum shear strain ($\gamma_{\max,N}$) and model coefficients (θ_i) as shown in Equation (6.7).

$$\ln(r_u) = ln\left[1 - exp\left(-\frac{\theta_1 \cdot \gamma_{max,N}}{\theta_2 + \theta_3 \cdot ln(\sigma'_{vo}) - \left({D_r}/{100}\right)}\right)\right] \pm \varepsilon_{\ln(r_u)} \qquad (6.7)$$

Inspiring from this functional form various functions were tried in order to capture the ideal behaviour of soils on r_u vs. γ domain for each set of descriptive variables. Consistent with the maximum likelihood methodology, model coefficients were estimated by maximizing the likelihood functions given in Table 6.1. The results of these analyses are also summarized in Table 6.1. As greater likelihood values and lower model errors are indication of a superior model, Model 3.5 (Equation 6.8) was decided to model excess pore water pressure generation response with highest accuracy and the corresponding limit state function was employed in the development of the proposed framework.

$$\hat{g}(r_u, \gamma) = \ln(r_u) - \ln[1 - \exp(\alpha)] \pm \varepsilon_{\ln(r_u)} \qquad (6.8)$$

$$\alpha = \left(\frac{\gamma_{max}}{\theta_1 - \theta_2 \cdot [ln(\theta_3 \cdot PI + 1)]^{\theta_4} - \theta_5 \cdot [ln(\theta_6 \cdot LL + 1)]^{\theta_7} + \theta_8 \cdot [ln(\theta_9 \cdot LL + 1)]^{\theta_{10}}}\right)$$

where r_u is the excess pore pressure ratio, γ_{max} is the double amplitude maximum shear strain and PI, LL and w_c/LL are descriptive parameters representing plasticity index, liquid limit and water content to liquid limit ratio, respectively.

Table 6.1 Results of maximum likelihood analysis performed on different functional forms.

Model No	Formulation	σ_ε	Σ likelihood
1.1	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_3*PI+1]\}\wedge\theta_4)]\}$	0.509	-2102.018
1.2	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[PI+1]\}\wedge\theta_3)]\}$	0.509	-2102.137
1.3	$\ln(r_u)=\ln\{1-\exp[\gamma\wedge\theta_1/(\theta_2-\theta_3\{\ln[PI+1]\}\wedge\theta_4)]\}$	0.503	-2063.602
1.4	$\ln(r_u)=\ln\{1-\exp[\gamma\wedge\theta_1/(\theta_2-\theta_3\{\ln[\theta_4*PI+1]\}\wedge\theta_5)]\}$	**0.503**	**-2063.507**
2.1	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_3*PI+1]\}\wedge\theta_4+(w_c/LL))]\}$	0.509	-2101.465
2.2	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[PI+1]\}\wedge\theta_3+(w_c/LL))]\}$	0.509	-2101.822
2.3	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_4*PI+1]\}\wedge\theta_3+\theta_4(w_c/LL))]\}$	0.509	-2099.178
2.4	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_4*PI+1]\}\wedge\theta_3+(w_c/LL)\wedge\theta_4)]\}$	0.508	-2095.883
2.5	$\ln(r_u)=\ln\{1-\exp[\{\gamma*(1+\theta_1*\{w_c/LL\})\}/(\theta_2-\theta_3\{\ln[PI+1]\}\wedge\theta_4)]\}$	**0.505**	**-2076.829**
3.1	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[PI+1]\}\wedge\theta_3-\theta_4\{\ln[PI+1]\}\wedge\theta_5+(w_c/LL))]\}$	0.504	-2073.728
3.2	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[PI+1]\}\wedge\theta_3-\theta_4\{\ln[PI+1]\}\wedge\theta_5+\theta_6(w_c/LL))]\}$	0.493	-1988.405
3.3	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_3PI+1]\}\wedge\theta_4-\theta_5\{\ln[\theta_6*PI+1]\}\wedge\theta_7+\theta_8(w_c/LL))]\}$	0.487	-1973.903
3.4	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_3PI+1]\}\wedge\theta_4-\theta_5\{\ln[\theta_6*PI+1]\}\wedge\theta_7+\theta_8\{\ln[w_c/LL+1]\}\wedge\theta_9)]\}$	0.485	-1962.923
3.5	$\ln(r_u)=\ln\{1-\exp[\gamma/(\theta_1-\theta_2\{\ln[\theta_3PI+1]\}\wedge\theta_4-\theta_5\{\ln[\theta_6*PI+1]\}\wedge\theta_7+\theta_8\{\ln[\theta_9*w_c/LL+1]\}\wedge\theta_{10})]\}$	**0.485**	**-1962.807**

Errors associated with this likelihood function were accounted by $\varepsilon_{\ln(r_u)}$ term which has a normal distribution with zero mean and standard deviation of σ_ε. θ_1 through θ_{10} along with σ_ε constitutes set of unknown model parameters whose values were estimated in order to maximize the value of likelihood function. Table 6.2 presents the estimated values of these parameters based on maximum likelihood principle.

Table 6.2 Values of unknown model parameters of Model 3.5 estimated by maximum likelihood principal

	Coarse Grained	Fine Grained
θ_1	-1.576	-1.576
θ_2	0.067	0.067
θ_3	0	0.055
θ_4	14.020	14.020
θ_5	7.007	7.007
θ_6	0	0.006
θ_7	0.134	0.134
θ_8	3.304	3.304
θ_9	0	1.702
θ_{10}	4.143	4.143
σ_ε	0.485	
Σ likelihood	-1962.81	

Figure 6.2 presents the mean boundary curves developed for coarse-grained and fine-grained (for the mean values of the compiled database, PI=22, LL=45, wc/LL=0.82) soils along with $\pm$ one standard deviation (σ_ε) curves and compiled database. This figure revealed that proposed model and the suggested error bands capture the observed soil response successfully.

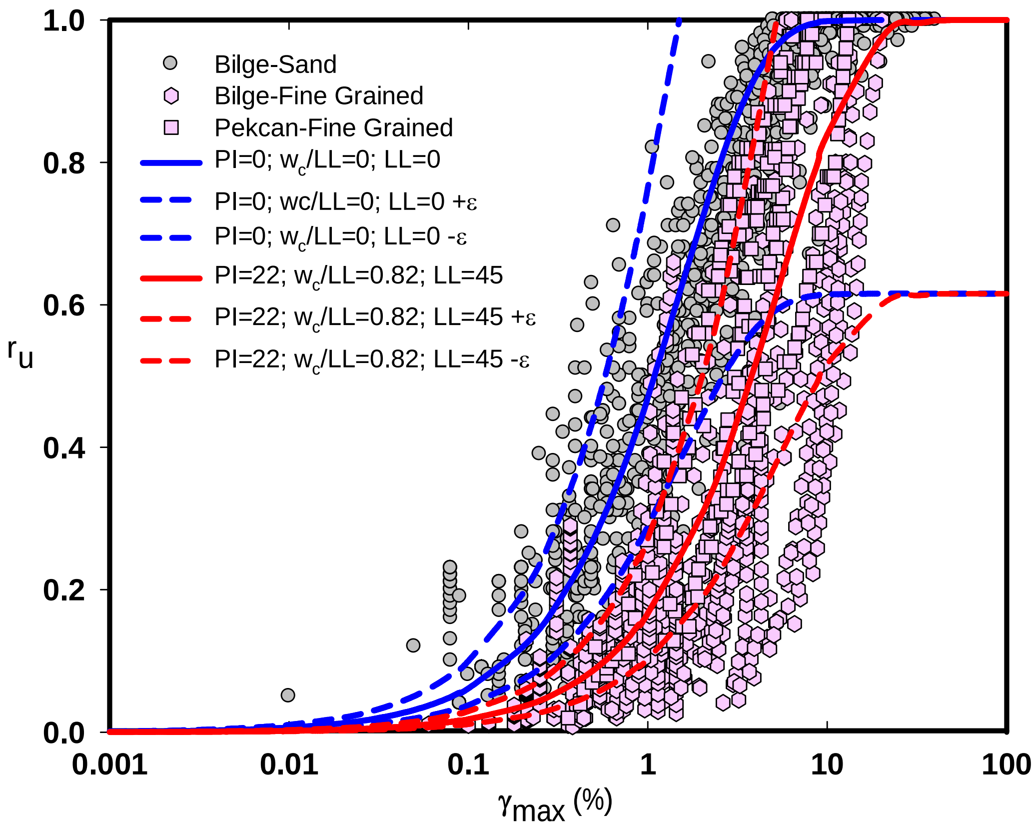

Figure 6.2 Mean boundary curves with one standard deviation bounds for clean sands and fine-grained soils.

The boundary curves obtained for saturated clean sands, by Eqn. 6.2, are then compared with the model proposed by Cetin and Bilge (2009) in Figure 6.3 along with the compiled database. Although the models give comparable results for the selected initial conditions, there is a certain degree of difference as a result of different model errors. However, further comparison is not fair since Cetin and Bilge model focuses only on the response of clean saturated sands and uses physically meaningful parameters (D_R and σ'_{v0}) and consequently gives more accurate results. This plot further validates use of the correlation proposed within the confines of this study for all types of soils.

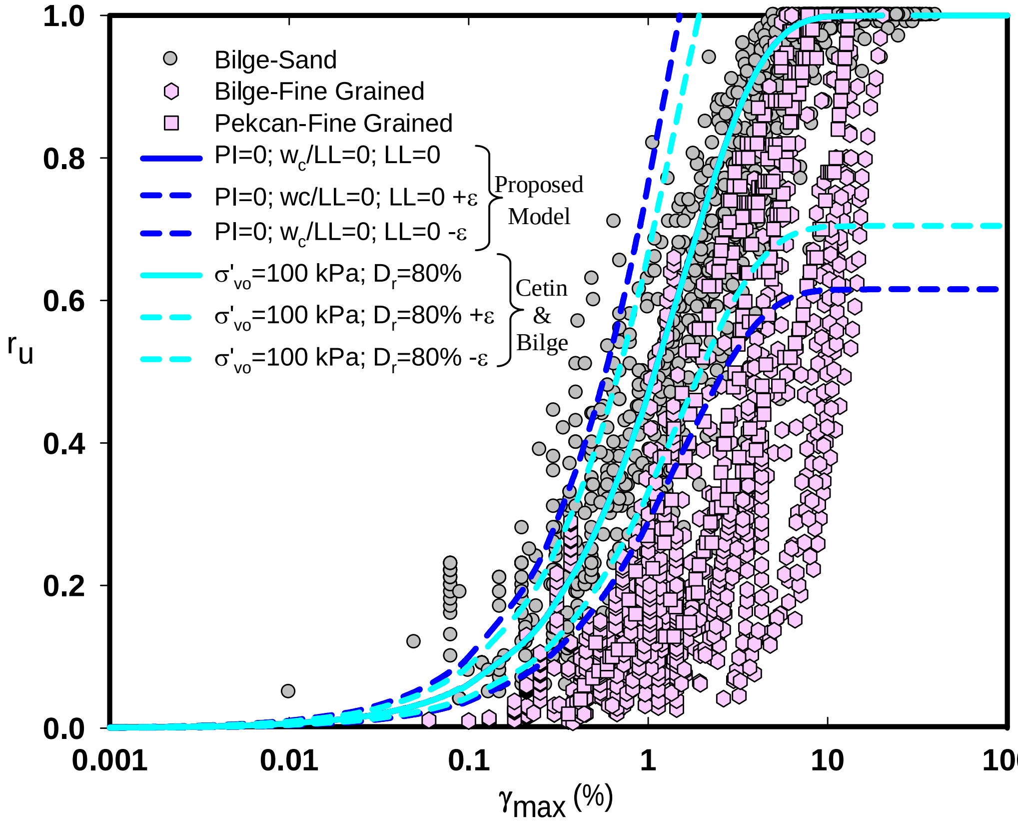

Figure 6.3 Comparision of the performance of proposed likelihood function on clean sands with the corralation proposed for sands by Bilge (2009)

Figure 6.4 presents the upper (non-plastic, PI=0, wc/LL=0, LL=0) and lower (PI=60, wc/LL=1.2, LL=95) boundary curves obtained by Model 3.5 for the existing boundaries in the compiled database. This figure shows that these curves capture almost all of the data, which strengthen the further use of the proposed model.

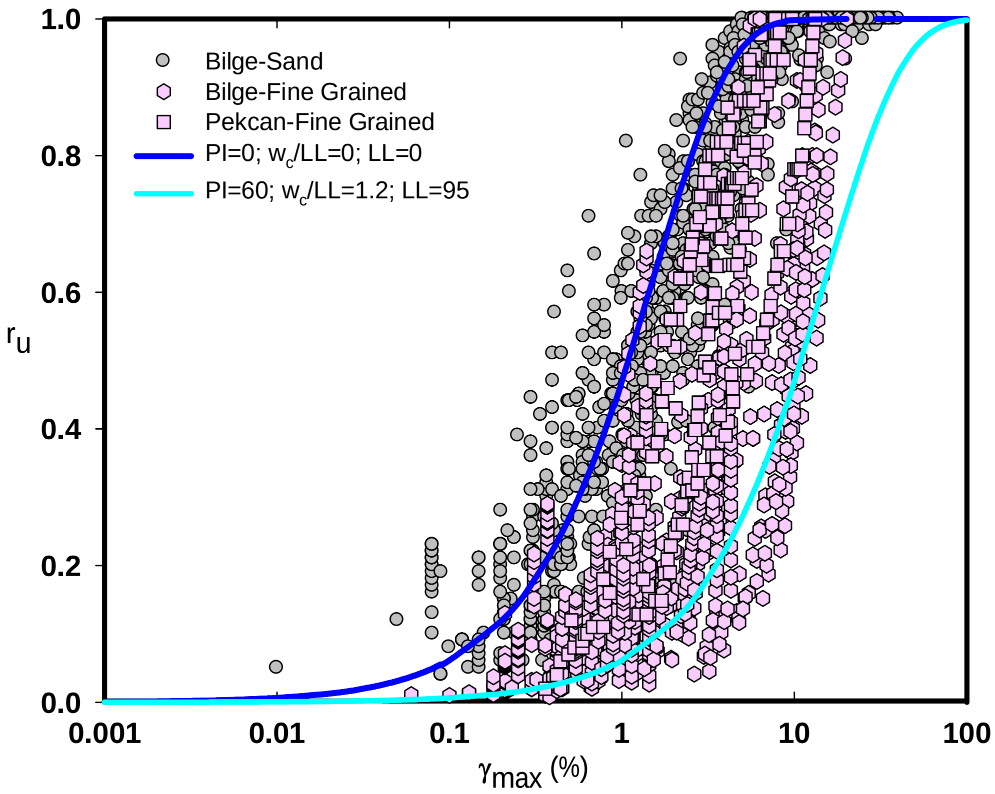

Figure 6.4 Boundary curves representing for top and bottom values of the database.

A sensitivity study is also performed to identify the response of the proposed model for various combinations of PI, LL and wc/LL. Figures 6.5 through 6.7 present the results of this study for variations in PI, LL and w_c/LL ratio, respectively.

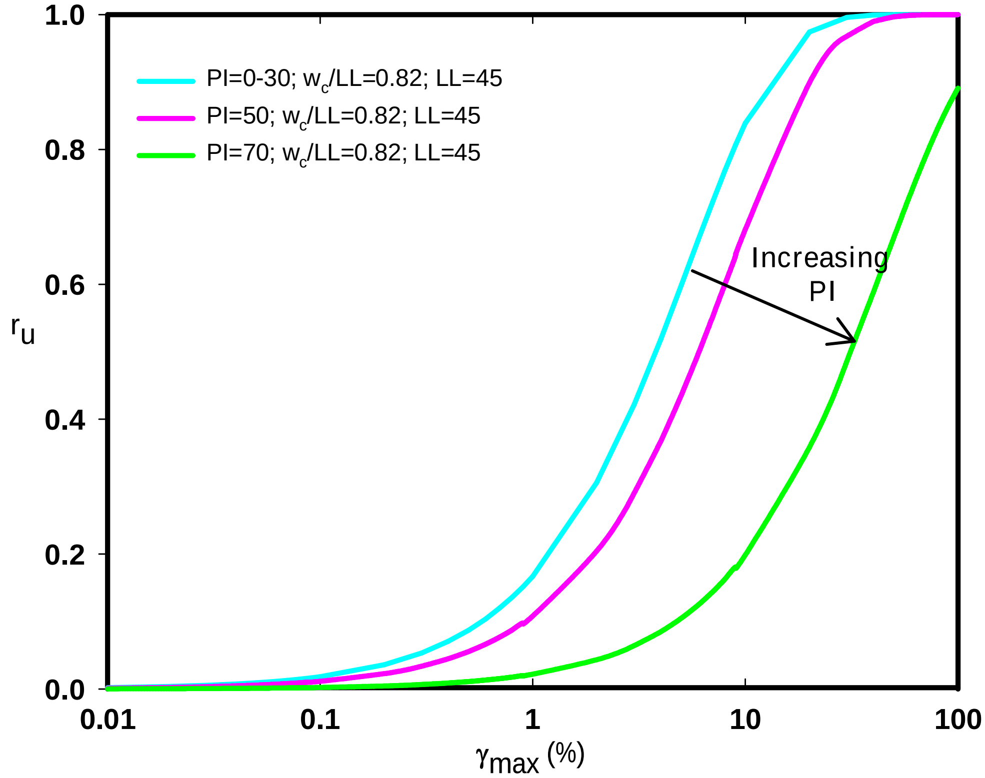

Figure 6.5 Boundary curves for varying plasticity index values

Figure 6.5 shows that, downward shift of boundary curves for low plasticity clays is less significant. For high PI values, a higher increase in PI value results in a sharp decrease in generated excess pore pressure. This indicates that small changes in PI alone, does not affect the pore pressure generation capacity of fine grained soils for PI < 30. Thus, Figure 6.5 points out the importance of other descriptive parameters, LL and w_c/LL ratio, defining the response of fine-grained soils under cyclic loading.

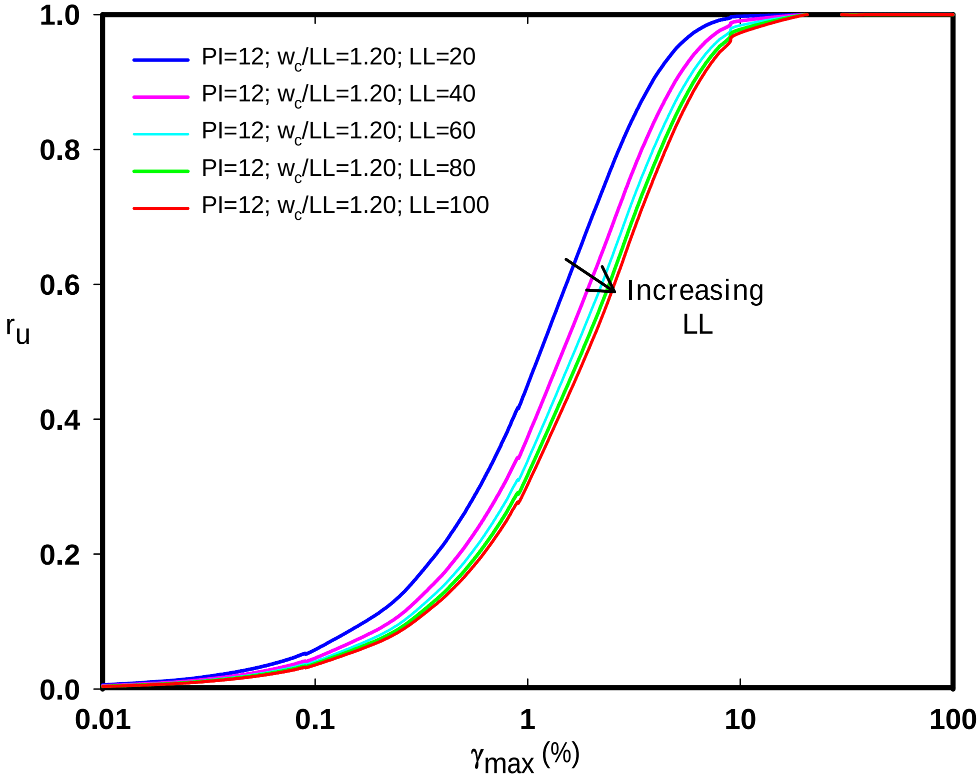

Figure 6.6 Boundary curves for varying liquid limit values.

Figure 6.6 illustrates the effect of variations in liquid limit values on excess pore pressure generation. This figure reveals that increasing LL results in an increase in cyclic resistance, i.e. same level of shear strain induces smaller excess pore water pressure in a soil with greater LL. While keeping other parameters constant, an increase in LL results in a shift toward right in plasticity chart, i.e. increasing plasticity. Hence observed response is consistent with the expected soil response.

The effect of variations in w_c/LL ratio is presented in Figure 6.7. An increase in w_c/LL ratio, while keeping other parameters constant, results in a decrease in cyclic resistance, which corresponds to an increase in the excess pore pressure generation capacity of fine-grained soils. This increase is more pronounced for w_c/LL ratio above 1.00 as shown in Figure 6.7.

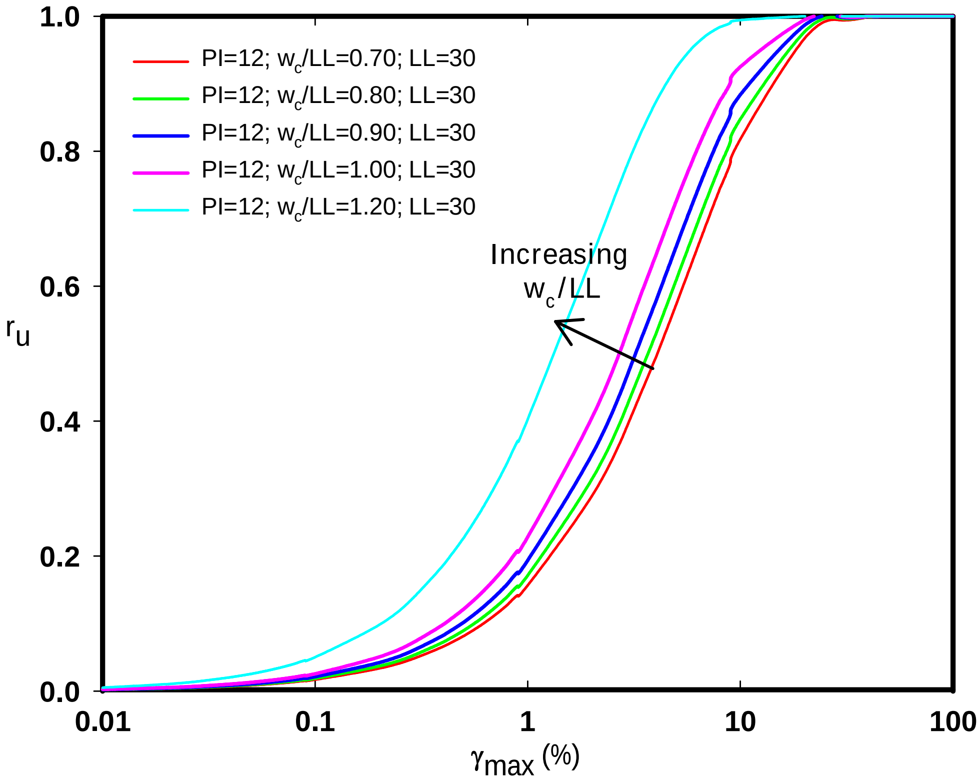

Figure 6.7 Boundary curves for varying water content to liquid limit values.

6.5 Development of Liquefaction Susceptibility Criteria

Likelihood function, presented in preceeding sections, enables estimation of excess pore pressure ratio (r_u) corresponding to a double amplitude maximum shear strain (γ_{max}) based on soil index and state parameters (PI, LL and w_c/LL ratio). New liquefaction susceptibility criteria can be assessed based on this likelihood function. Assessment of new liquefaction susceptibility criteria requires a definition for liquefaction to classify soils as either liquefiable or non-liquefiable. Thus, considering available literature and the observed cyclic response trends of fine-grained soils (Figure 6.2), liquefaction was defined as;

For γ_{max}=7.5 %;,

if $0.85 < r_u \leq 1.00$ $\longrightarrow$ Liquefiable

$0.70 < r_u \leq 0.85$ $\longrightarrow$ Test

Otherwise $\longrightarrow$ Non-Liquefiable

Based on this liquefaction definition, the error of the proposed methodology was assessed to be 10% by using the database compiled for the evaluation of available liquefaction susceptibility criteria, presented in Chapter 3. This error is significantly lower than the errors of the available liquefaction susceptibility criteria which were presented in Chapter 4. Table 6.3 summarizes the results of evaluation of proposed methodology for assessment of liquefaction susceptibility with compiled database for liquefaction condition stated above.

Table 6.3 Comparision of proposed methodology with compiled database for the new liquefaction condition

Observed\This Study	Yes	Test	No
Yes	42	24	5
Test	0	3	2
No	7	22	26

Figure 6.8 illustrates the new liquefaction susceptibility criteria for w_c/LL=1.00 condition. This new criteria is applicable to soils whose fines content is more than 35%. The liquid limit boundaries were estimated by solving the proposed likelihood function for the liquefaction boundaries specified above for w_c/LL=1.00 condition. However, plasticity index boundaries were defined according to the "database boundary line" shown in this figure. Equation (6.9) presents the functional form of database boundary line.

$$PI = 0.83LL - 11.46 \tag{6.9}$$

Database boundary line indicates the upper limit of compiled database on PI vs. LL. Thus, the limit state model presented in this study is not valid above this line. Even though higher PI values were obtained from likelihood function for these conditions, they are truncated considering this line. Hence, the PI boundaries vary based on the LL boundaries. Table 6.4 presents LL boundaries (LL_{Liq} and LL_{Test}) for different w_c/LL conditions.

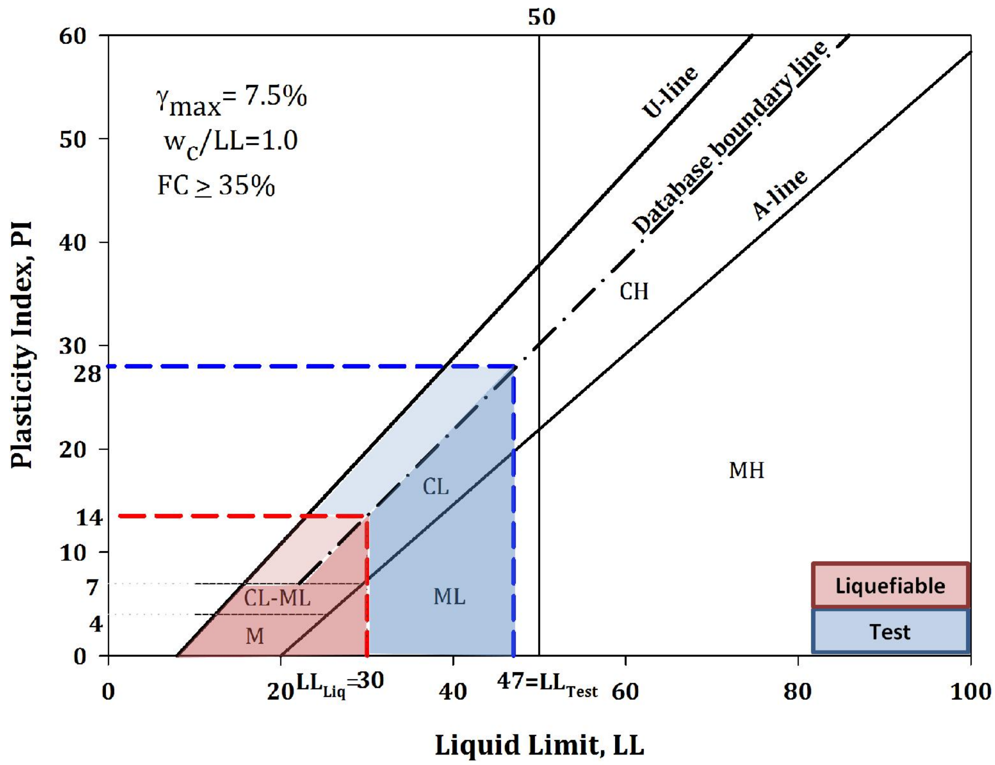

Figure 6.8 New liquefaction susceptibility criteria for fine grained soils for w_c/LL=1.00 condition.

Table 6.4 Liquid limit boundaries for varying w_c/LL conditions.

w_c/LL	LL_{Liq}	LL_{Test}
0.70	1	40
0.80	3	47
0.85	6	47
0.90	10	47
1.00	30	47
1.10	55	55
1.20	65	70

New liquefaction susceptibility criteria, proposed for w_c/LL=1.00 condition, suggest that soils lying below U-line with i)LL < 30 and ii) PI< 14 are susceptible to liquefaction, whereas those with i) 30 ≤ LL < 47 and ii) 14 ≤ PI < 28 need further testing. Soils satisfying none of these conditions are stated to be non-liquefiable for this condition. The areas filled with lighter colour lies above the database boundary line, thus, no data was available for these regions. Hence, extrapolation technique was used to assess liquefaction susceptibility of soils lying in these areas.

6.6 Comparison of New Liquefaction Susceptibility Criteria with Available Criteria

This section presents the comparison of developed liquefaction susceptibility criteria with liquefaction susceptibility criteria available in literature (Seed et al., 2003; Bray and Sancio, 2006 and Boulanger and Idriss, 2004,2006)

6.6.1 Seed et al. (2003)

The liquefaction susceptibility criteria proposed by Seed et al. (2003) was discussed in detail in Chapter 2 and 4. Figure 6.9 shows the criteria of Seed et al. (2003) plotted on the liquefaction susceptibility criteria proposed in this study. This figure indicates that, compared to Seed et al.(2003), for liquefiable zone, new criteria reduces the upper liquid limit boundary while slightly increasing upper plasticity index boundary. Upper LL boundary for test region of both criteria coincides while PI boundary of the new criteria is higher. It should be noted that Seed et al. (2003) provides boundaries of liquefiable and test regions for w_c/LL

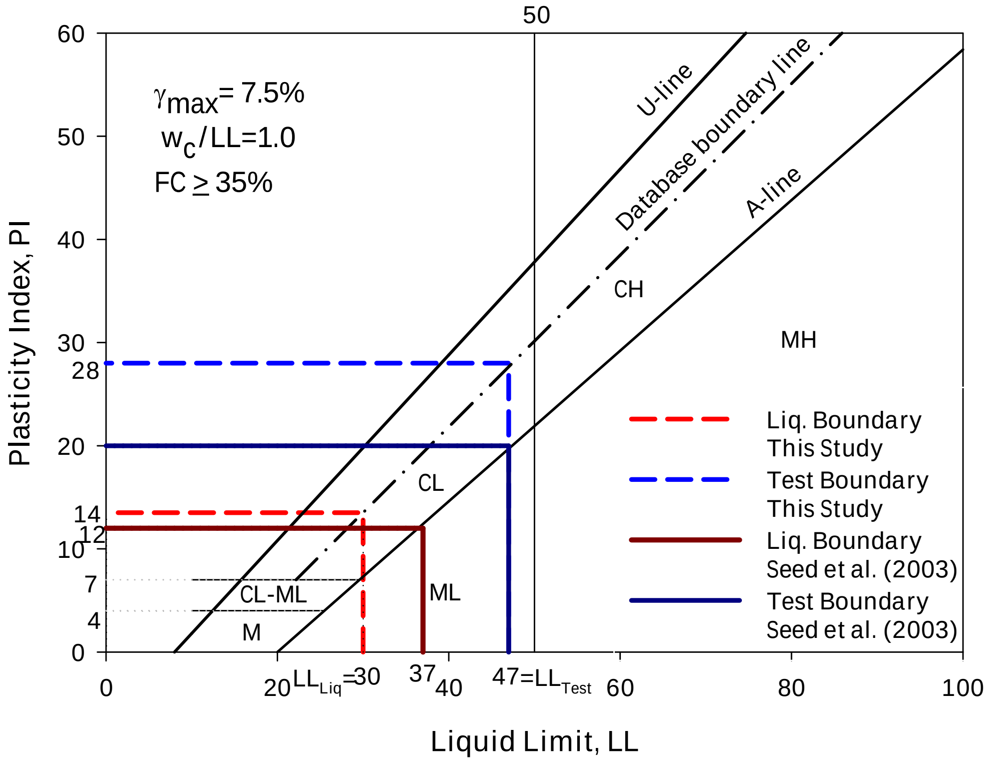

Figure 6.9 Comparision of new liquefaction susceptibility criteria with Seed et al. (2003).

ratios higher than 0.80 and 0.85, respectively. However the boundary conditions of new criteria were presented for w_c/LL = 1.00 condition. When the boundary conditions provided in Table 6.4 is observed, it will be seen that new liquefaction susceptibility criteria significantly changes for different levels of w_c/LL ratio.

6.6.2 Bray and Sancio (2006)

The liquefaction susceptibility criteria proposed by Bray and Sancio (2006) was discussed in detail in Chapter 2 and 4. Bray and Sancio (2006) proposed liquefaction susceptibility criteria on w_c/LL vs. PI domain. To be able to compare these two criteria, the liquefaction susceptibility criteria proposed in this study is redefined on this domain. However, this domain does not consider the variation in liquid limit while LL is one of the parameters of the limit state function used for developing the liquefaction susceptibility boundary curves of this study. Considering that the compiled database lies in the vicinity of A-line, the LL values corresponding to a PI value on A-line was used while developing the boundary curves on w_c/LL vs. PI domain. The boundary curves assessed for liquefiable and test zones on this domain were plotted with the ones proposed by Bray and Sancio (2006) as shown in Figure 6.10.

Figure 6.10 indicates that the w_c/LL boundary suggested in this study is lower than the suggested boundary of Bray and Sancio (2006)for test region, whereas it is higher than their suggestion for liquefiable region. Moreover, consistent with the expected behaviour, the PI limit of the proposed liquefaction susceptibility criteria increases with increasing w_c/LL ratio until w_c/LL ratio becomes equal to 1.00, whereas Bray and Sancio (2006) propose an upper PI limit which is not dependent on variation of w_c/LL ratio.

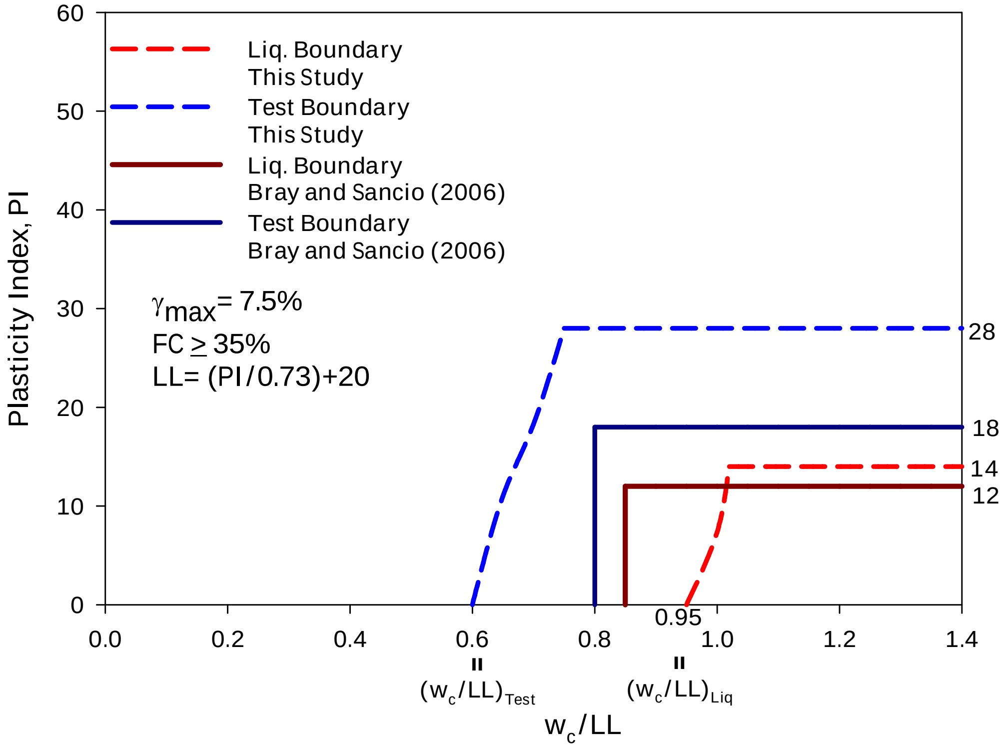

Figure 6.10 Comparision of new liquefaction susceptibility criteria with Bray and Sancio (2006)

6.6.3 Boulanger and Idriss (2004, 2006)

The liquefaction susceptibility criteria proposed by Boulanger and Idriss (2004, 2006) was discussed in detail in Chapter 2 and 4. Boulanger and Idriss classified soils either sand-like or clay-like and proposed a relation describing the transition from sand-like to clay-like behaviour on CSR vs. PI domain. However, the CSR axis is not scaled. Thus, only PI boundaries provide information regarding liquefaction potential of soils.

Figure 6.11 shows the criteria of Boulanger and Idriss (2004, 2006) along with the new liquefaction criteria proposed in this study.

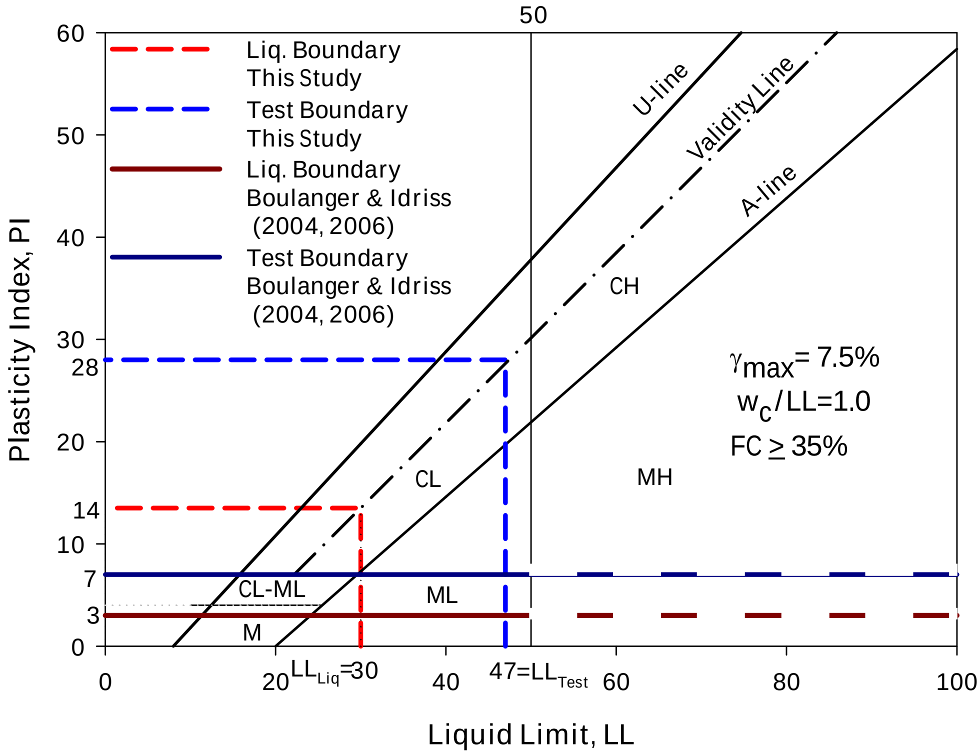

Figure 6.11 Comparision of new liquefaction susceptibility criteria with Boulanger and Idriss (2004.2006)

6.7 Conclusions

This chapter presents details of the probabilistic-based excess pore water pressure generation model. Various functional forms with three different sets of descriptive variables were tested using maximum likelihood methodology. The limit state model with maximum likelihood value and least standard deviation was selected for further analysis. Equation (6.10) presents proposed probabistically based framework for the assessment of excess pore water pressure ratio and maximum shear strain couples if PI, LL and w_c/LL ratio of the soil is known.

$$\hat{g}(r_{u,n};\gamma_{max}) = \ln(r_{u,n}) - \ln\left\{1 - exp\left[\frac{\gamma_{max}}{-1.576 - 0.067\langle\ln(0.055\times PI + 1)\rangle^{14.020} - 7.007\langle\ln(0.006\times LL + 1)\rangle^{0.134} + 3.304\times\langle\ln(1.702\times {}^{w_c}/_{LL} + 1)\rangle^{4.143}}\right]\right\} \pm 0.485 \tag{6.10}$$

The boundary curves developed based on this limit state model is compared with database along with error margins and it is concluded that the experimental trends for both cohesive and cohesionless soils can be successfully modeled. Moreover, the proposed model’s sensitivities to PI, LL and w_c/LL ratio were also investigated.

New methodology for the assessment of liquefaction potential of fine-grained soils considers the level of cyclic loading via level of shear strain. There exist various studies focusing on the relation between shear strain and the magnitude and duration of cyclic loading (Tokimatsu and Seed, 1984; Ishihara and Yoshimine, 1992; Wu et al., 2003; Cetin et al., 2009) for saturated cohesionless soils. Using the results of this study, it is possible to assess the cyclically-induced shear strains for saturated cohesionless soils, and the proposed model can be used to determine the corresponding level of excess pore water pressure. Afterwards, liquefaction susceptibility of a specific soil site can be estimated by examining the excess pore pressure ratio level associated with its index and state parameters. Another advantage of the proposed methodology is that it does not require a definition for liquefaction. Moreover, it can clearly differentiate “cyclic mobility” from “cyclic liquefaction” since it provides excess pore pressure ratio corresponding to cyclically-induced shear strain level.

New liquefaction definition was proposed considering the cyclic behaviour of fine grained soils, consistent with the literature as;

For γ_{max}=7.5 %;,

if $0.85 < r_u \leq 1.00$ $\longrightarrow$ Liquefiable

$0.70 < r_u \leq 0.85$ $\longrightarrow$ Test

Otherwise $\longrightarrow$ Non-Liquefiable

Error associated with the proposed methodology for this liquefaction definition was found to be 10% which is significantly lower than the errors associated with existing criteria for this definition (Seed et al. (2003)= 18.1%, Bray and Sancio (2006)= 20% and Boulanger and Idriss= 52.5%).

Considering this liquefaction definition, new liquefaction susceptibility criteria was developed based on proposed likelihood function for different w_c/LL ratios. Additionally, comparison of these new liquefaction susceptibility criteria with available criteria was presented in the confines of this chapter.

CHAPTER 7

ESTIMATION OF LIQUEFACTION SUSCEPTIBILITY MARGINS ON CPT DOMAIN

7.1 Introduction

Excess pore water pressure generation response of various soil types has been discussed in Chapter 6 by developing a probabilistic-based framework enabling prediction of r_u values as a function of shear strain and basic index properties. This robust framework enables assessment of liquefaction potential based on the induced pore water pressures without putting a constraint such as definition of liquefaction. In order to increase its potential use, it is decided to transfer boundary curves of proposed model into cone penetration test (CPT) domain, i.e. $q_{t,1,net}$ vs. F_R.

The CPT is an in-situ testing technique widely used in subsurface soil characterization studies for the purpose of geological and geotechnical data compilation. CPT is considered as one of the most reliable, repeatable and robust in-situ testing technique and this is the reason of selection of adopting CPT domain into the proposed liquefaction assessment framework. Despite its numerous advantages, CPT has one major drawback which is lack of sampling. This is an important problem for the proposed framework since it requires a prior knowledge of soil index properties, Atterberg limits and in-situ moisture content. To account for this need, new liquefaction susceptibility margins were developed on CPT domain based on the results of a recent study of Cetin and Ozan (2009).

This chapter will proceed with general information regarding cone penetration testing and introduce the findings of Cetin and Ozan (2009). Next, mapping procedure of the proposed boundary curves on to the CPT domain will be discussed and chart solutions which enable estimation of liquefaction potential of fine-grained soils by presenting the level of excess pore pressure ratio corresponding to CPT measurements, normalized net cone tip resistance ($q_{t,1,net}$) and friction ratio (F_R) will be provided. Finally, liquefaction susceptibility margins developed on CPT domain will be presented.

7.2 Background Information on CPT

CPT is an in-situ testing technique used widely in geotechnical practice owing to its reliability, robustness and repeatability. Consistent with D 6066-98 standards (ASTM, 2000), during a conventional CPT, a 10 cm^2-area steel cone is hydraulically pushed into the soil at a rate of 20 mm/sec and resistance at the tip and the sleeve of the cones is recorded during penetration. Soil resistance is expressed by the sum of (i) cone tip resistance, q_c, which represents the total force acting on the cone divided by the projected area of the cone and (ii) sleeve friction resistance, f_s, which represents the total friction force acting on the friction sleeve divided by its surface area. Side friction is commonly expressed by friction ratio, F_R.

Measured cone tip resistance is affected by some factors including; overburden pressures, end effects and thin layers. Numerous studies in literature proposed various methodologies regarding correction of cone tip resistance for these factors (e.g. Olsen and Mitchell, 1995; Robertson, 1999; Lunne et al., 1997; Moss et al., 2006 and Cetin and Ozan, 2009). Among these, the corrections proposed by Cetin and Ozan (2009) will be presented in this study since the proposed boundary curves were decided to be mapped on to the $q_{t,1,net}$ vs. F_R domain based on Cetin and Ozan (2009) to cope with the sampling problem.

Cetin and Ozan (2009) suggested the following correction scheme for estimation of corrected cone tip resistance (q_t) to correct for unequal end effects.

$$q_t = q_c + u \cdot (1 - a) \tag{7.1}$$

where a is the area ratio of the cone and u is the pore pressure measured behind the cone shoulder.

Net corrected tip resistance, $q_t - \sigma_v$, should be further normalized with vertical effective stress (σ'_v) of 1 atm, to eliminate the variability in vertical stress conditions. Cetin and Ozan (2009) proposed Equation (7.2) for the assessment of normalized net cone tip resistance, $q_{t,1,net}$.

$$q_{t,1,net} = \frac{q_t - \sigma_v}{\left(\frac{\sigma'_v}{P_a}\right)^c};\ 0.25 \leq c \leq 1.00\ \ and\ \ \left(\frac{\sigma'_v}{P_a}\right)^c \leq 2.00 \tag{7.2}$$

where P_a is the atmospheric pressure in terms of σ'_v and c is the power law stress normalization exponent recommended by Cetin and Isık (2007) (Equations 7.3-7.4)

$$c = \frac{R - 272.38}{275.19 - 272.38} \pm 0.085; \quad 272.38 < R < 275.19 \tag{7.3}$$

$$R = \sqrt{[\log(F_R) + 243.91]^2 + \left[\log\left(\frac{q_{t,1,net}}{P_a}\right) - 126.24\right]^2} \tag{7.4}$$

where F_R is the friction ratio which is defined as shown in Equation (7.5)

$$F_R = \frac{f_s}{q_t - \sigma_v} * 100 \tag{7.5}$$

The estimation of $q_{t,1,net}$ and c requires an iterative procedure. After a couple of iterations, compatible normalized tip resistance and friction ratio can be obtained for further use in developed chart solutions.

Cetin and Ozan (2009) employed Bayesian Updating model while developing limit state models for CPT-based estimation of soil index parameters, namely, fines content (FC), plasticity index, (PI) and liquid limit (LL). Figures 7.1 through 7.3 present the plots showing boundary curves assessed for fines content, plasticity index and liquid limit, respectively, by Cetin and Ozan (2009).

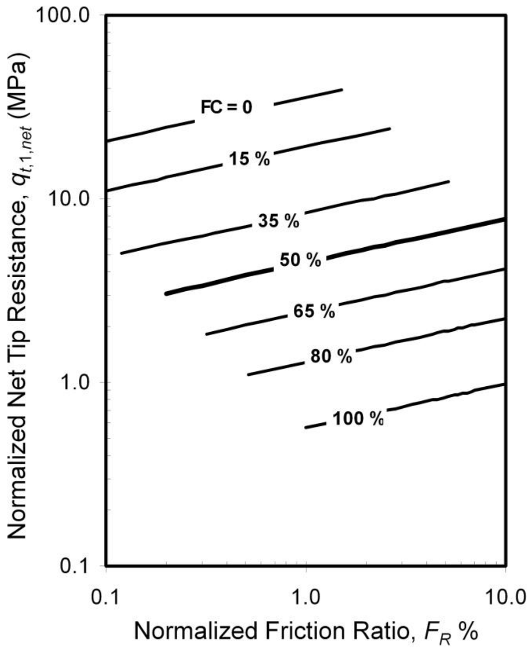

Figure 7.1 Fines content (FC) boundaries (after Cetin and Ozan, 2009)

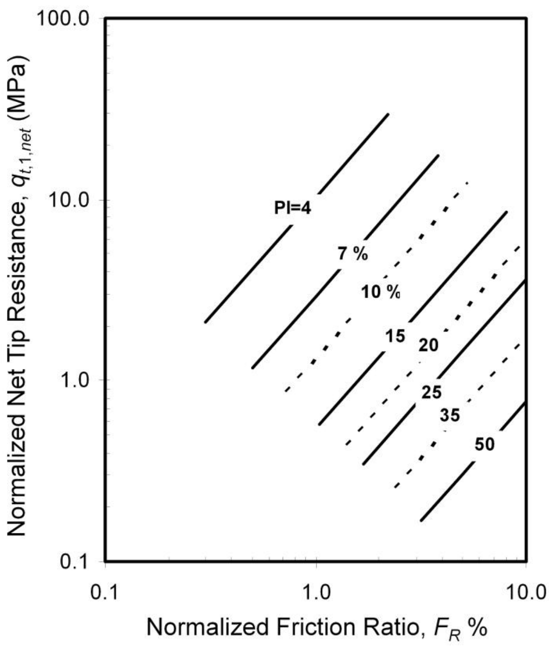

Figure 7.2 Plasticity index (PI) boundaries (after Cetin and Ozan, 2009)

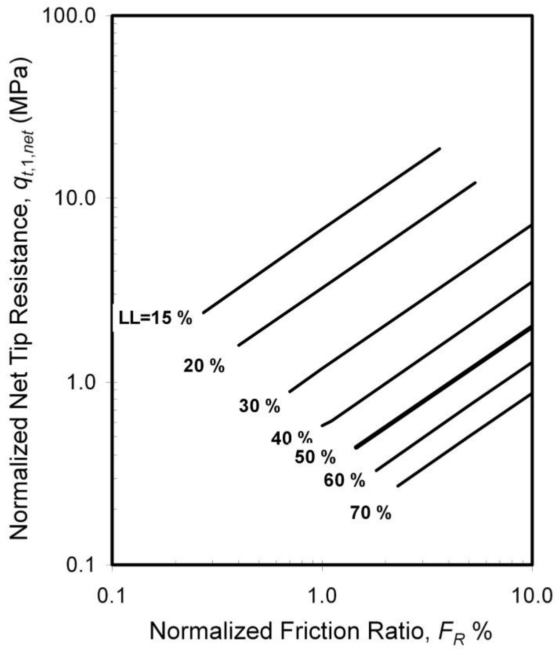

Figure 7.3 Liquid limit (LL) boundaries (after Cetin and Ozan, 2009)

7.3 Estimation of r_u on CPT Domain

Likelihood framework presented in Chapter 6 allows estimation of excess pore pressure ratio corresponding to a specific double amplitude shear strain as a function of Atterberg limits and natural water content. Even though this procedure enables user to estimate the liquefaction susceptibility of any kind of soil based on any liquefaction definition, it also requires field sampling for the determination of required soil index parameters. To overcome this limitation of CPT, the methodology proposed by Cetin and Ozan (2009) is utilized which provides probabilistic-based boundary curves for these index properties.

Chart solutions for the assessment of r_u were developed for three different shear strain amplitudes, 3, 5 and 7.5%, which are usually adopted as strain-based liquefaction criteria and define critical strain levels for soil performance. As Cetin and Ozan do not provide a relation between w_c/LL ratio and CPT measurements, for each shear strain level, chart solutions were developed for three different w_c/LL ratios, 0.80, 1.00 and 1.20. Within the confines of this chapter, chart solutions developed only for 7.5 % shear strain level will be presented; however, the rest of these chart solutions are available in Appendix B.

Figures Figure 7.4 through 7.6 present chart solutions for the assessment of r_u generation capacity of soils having w_c/LL ratios; 0.80, 1.00 and 1.20, respectively for 7.5% shear strain level. The minimum r_u levels induced by selected shear strain level (in this case 7.5 %) are indicated on these figures. For soils with FC < 35%, excess pore pressure ratios, r_u, were estimated based on Cetin and Bilge (2009) for relative density (D_R) values of 50, 80 and 100. For soils having FC ≥ 35%, excess pore ratios were estimated by the proposed model.

Figures 7.4 through 7.6 revealed that, excess pore pressure ratio increases with increasing water content. For soils of low-plasticity, i.e. PI ≤ 20, the controlling parameter is concluded to be LL and increasing LL values results in a decrease in r_u level. On the other hand, for soil of high plasticity, PI controls the induced r_u values. These figures also imply that increasing friction ratio results in decreasing excess pore pressure ratio; whereas, increasing net tip resistance may cause an increase in excess pore pressure ratio. However it should be noted that soils with net tip resistance more than 40 MPa are too stiff to be liquefiable, therefore, FC=0 can be considered as the upper limit for the proposed susceptibility criteria to identify liquefiable soils.

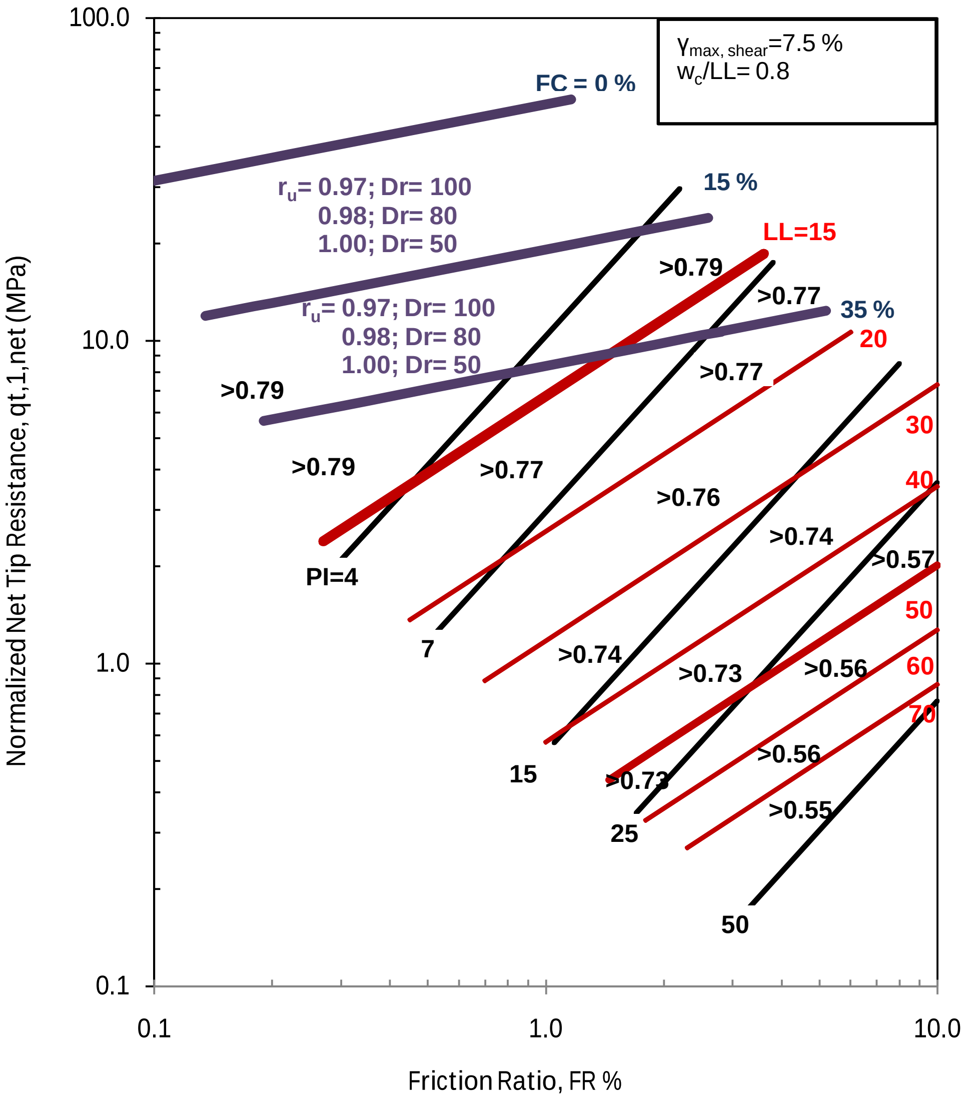

Figure 7.4 Chart solution of r_u for γ_{max}=7.5% and w_c/LL = 0.80 on CPT domain

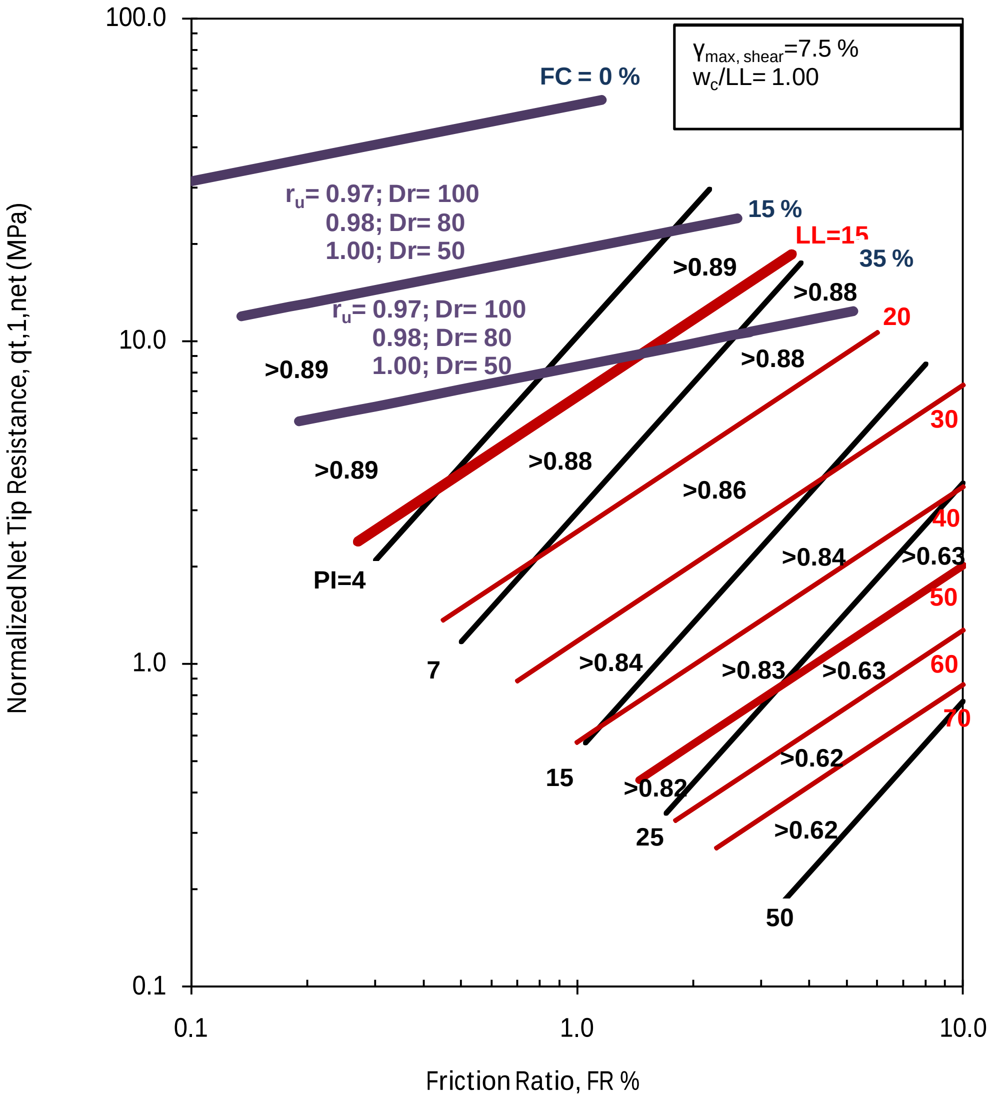

Figure 7.5 Chart solution of r_u for γ_{max}=7.5% and w_c/LL = 1.00 on CPT domain

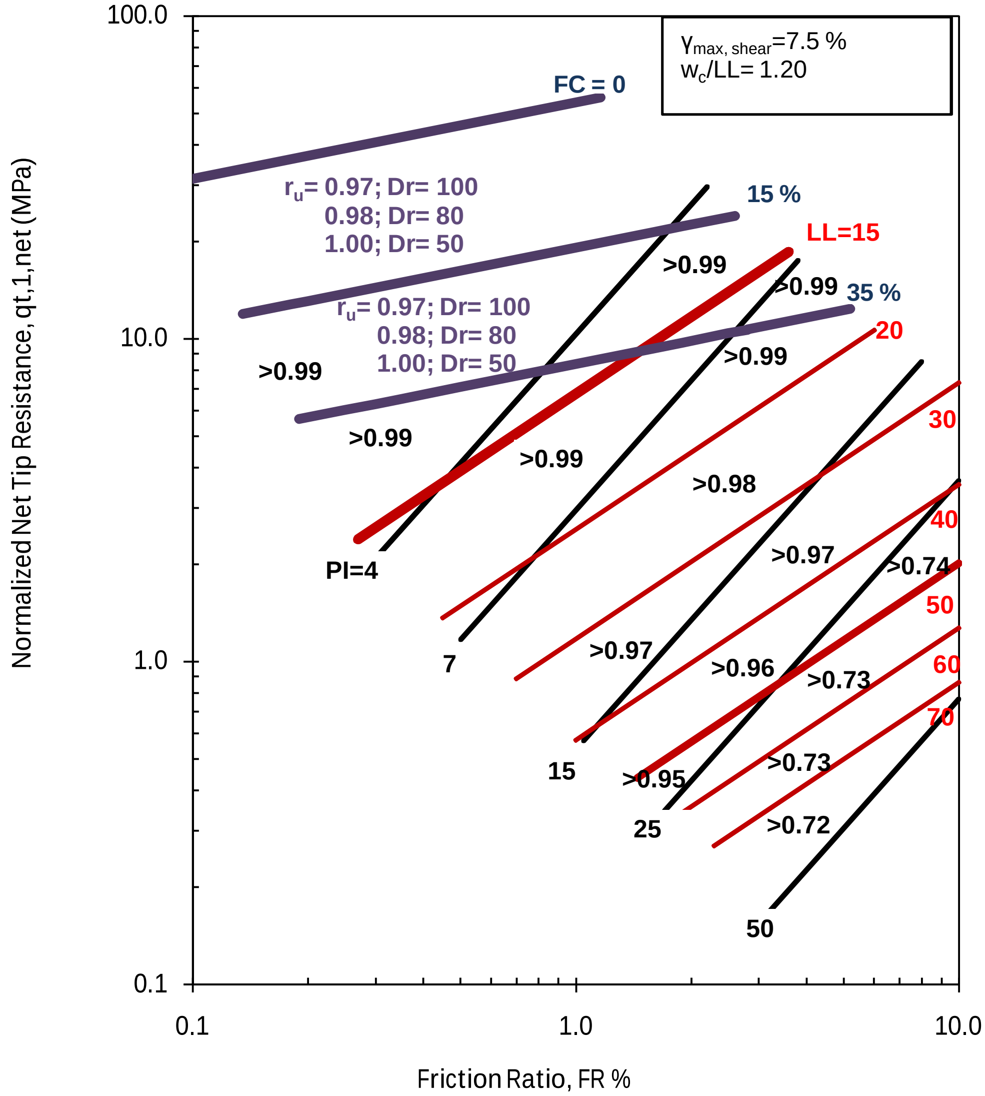

Figure 7.6 Chart solution of r_u for γ_{max}=7.5% and w_c/LL = 1.20 on CPT domain

7.4 Development of Liquefaction Susceptibility Criteria on CPT Domain

Chart solutions, presented in preceeding sections, enable estimation of excess pore pressure ratio (r_u) corresponding to a double amplitude maximum shear strain (γ_{max}) based on soil index and state parameters (PI, LL and w_c/LL ratio). Considering these chart solutions together with the liquefaction definition presented below, liquefaction susceptibility margins were developed on CPT domain.

For γ_{max}=7.5 %;

if $0.85 < r_u \leq 1.00 \longrightarrow$ Liquefiable

$0.70 < r_u \leq 0.85 \longrightarrow$ Test

Otherwise $\longrightarrow$ Non-Liquefiable

Figures 7.7 through 7.9 show the liquefaction susceptibility margins developed for w_c/LL ratio of 1.00, 0.90 and 0.80 respectively. These margins were adopted from the new liquefaction susceptibility criteria presented in Chapter 6.

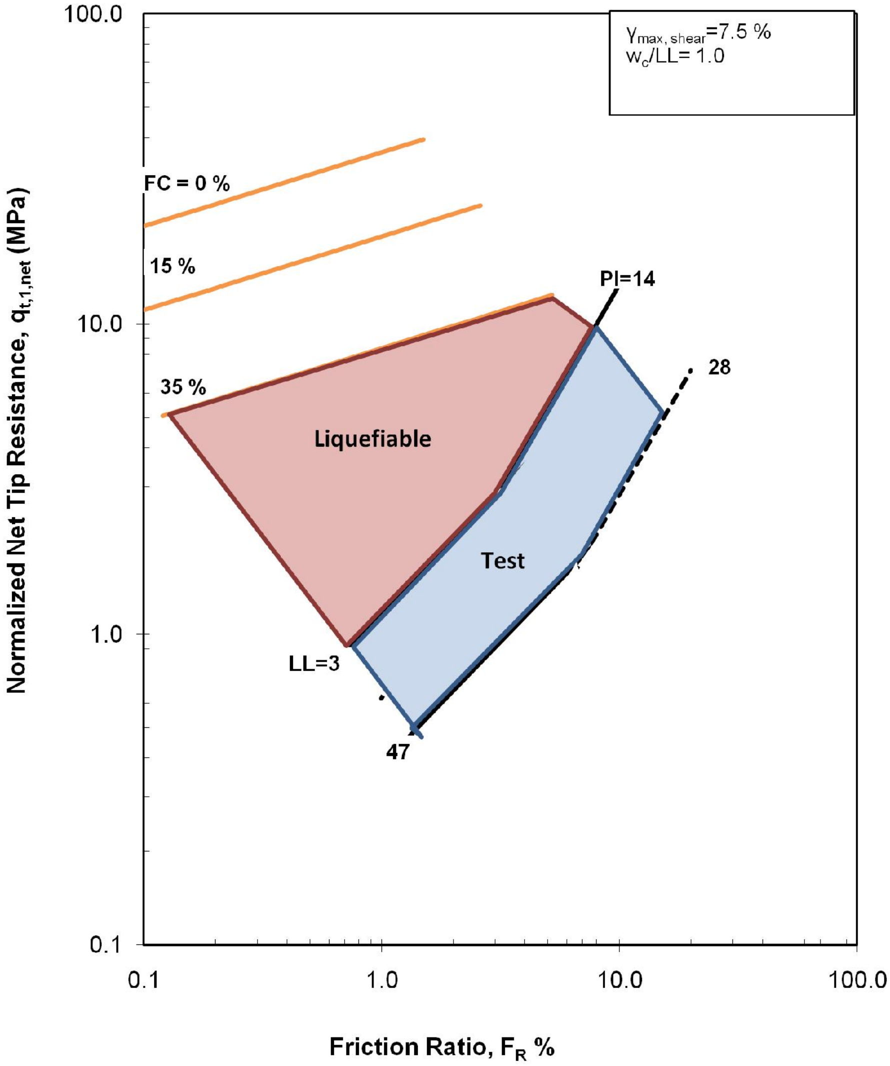

Figure 7.7 Liquefaction susceptibility margins for w_c/LL=1.00 on CPT domain.

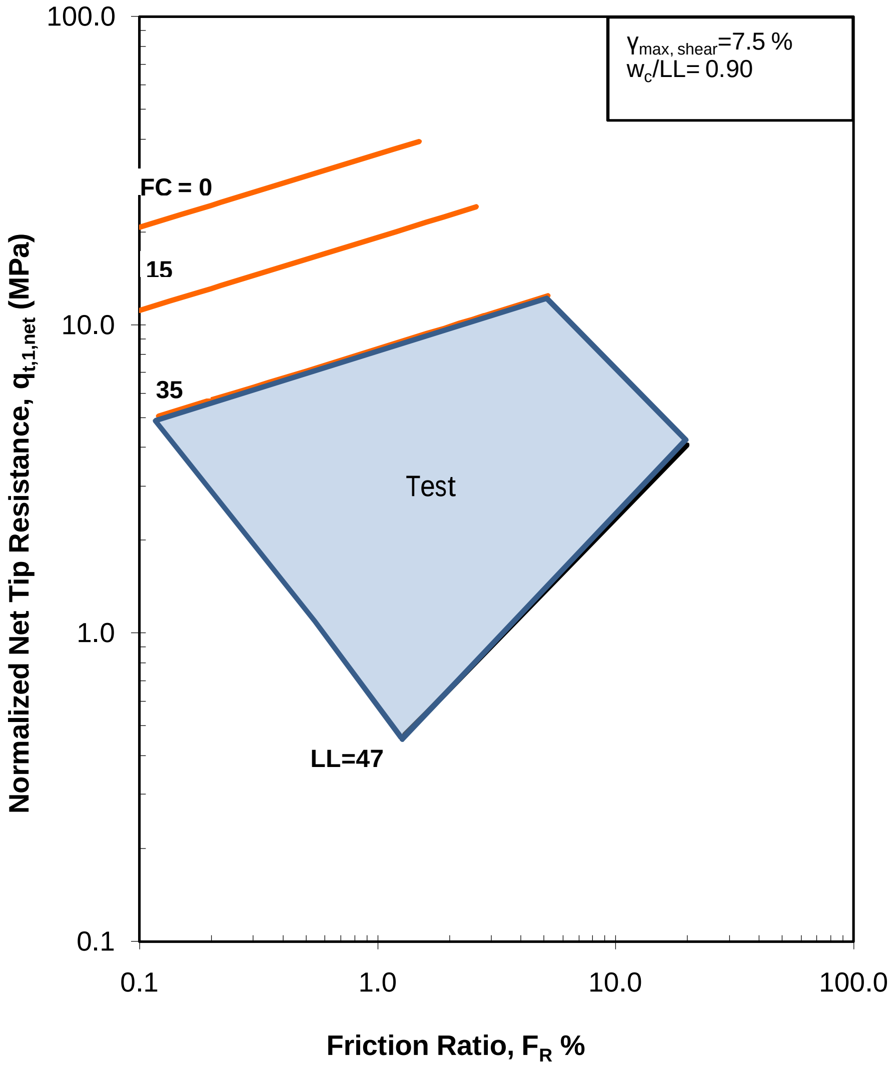

Figure 7.8 Liquefaction susceptibility margins for w_c/LL=0.90 on CPT domain

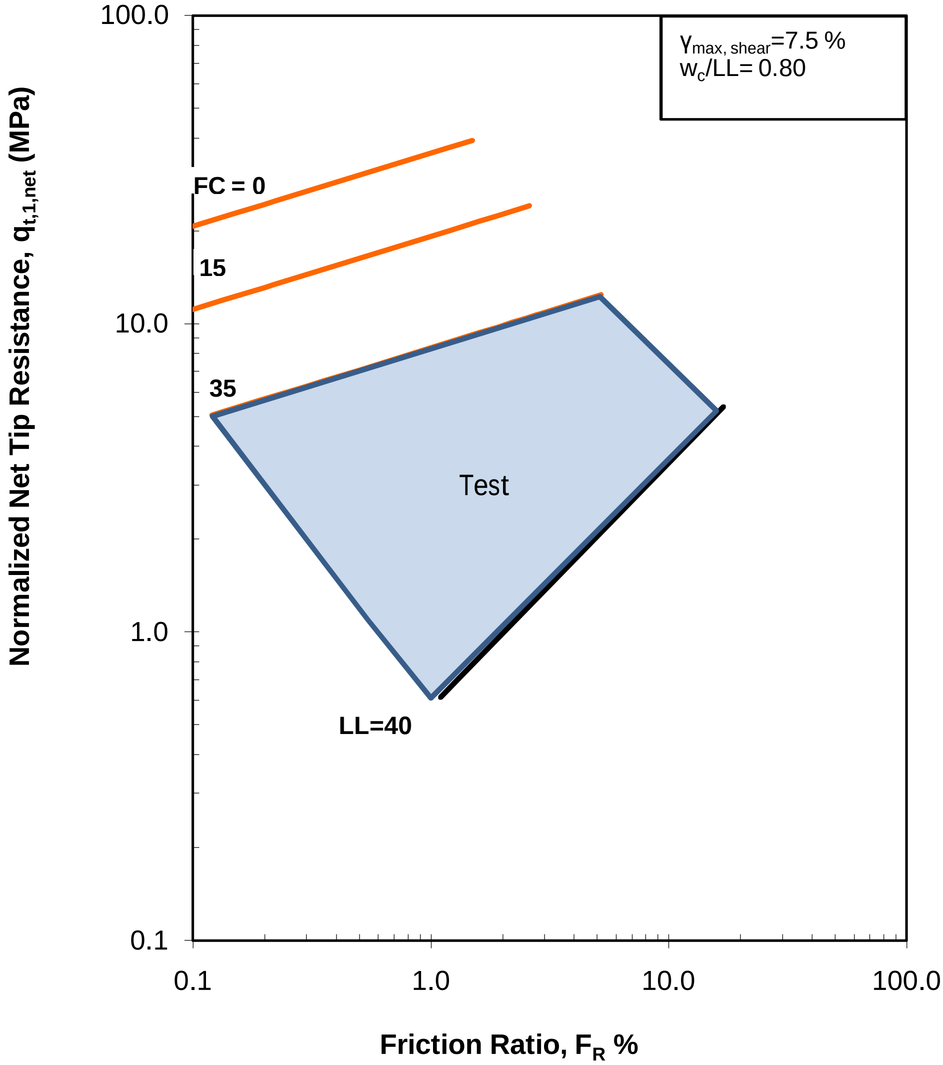

Figure 7.9 Liquefaction susceptibility margins for w_c/LL=0.80 on CPT domain.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Summary

The goal of this study is developing a robust methodology for the assessment of the susceptibility of fine-grained soils to liquefaction. Parallel to this goal, it is also intended to re-visit some of the contradictions in previous susceptibility criteria such as (i) definition of liquefaction, (ii) the effects of loading amplitude and (iii) distinguishing between cyclic mobility and cyclic liquefaction type soil responses.

The first notable effort to identify potentially liquefiable fine-grained soils was Chinese Criteria (Wang, 1979). These criteria have been used with some modifications (e.g. Seed et al.1983, Finn 1993) until the case histories compiled after some recent earthquakes (e.g. 1994 Northridge, 1999 Kocaeli, 1999 Chi-Chi) verified that neither their original form nor their modifications can successfully identify soils liquefied during and after these earthquakes. This fact has accelerated the studies focusing on development of new criteria. Based on field observations and results of laboratory cyclic tests on “undisturbed” samples from liquefied sites, Seed et al. (2003), Bray and Sancio (2006) and Boulanger and Idriss (2006) have proposed new criteria for the assessment of liquefaction potential of fine-grained soils recently. Although they are major improvements over previous efforts and help to better understand the soil response, they also suffer from some limitations such as (i) the lack of a clear definition of liquefaction, (ii) characterizationamplitude of loading, and (iii) separation of cyclic mobility and cyclic liquefaction. Considering these limitations, this study aims to develop a framework to assess liquefaction susceptibility of fine-grained soil.

Two different databases were compiled for (i) the evaluation of previous liquefaction susceptibility criteria and (ii) correlation development. For the evaluation of previous liquefaction susceptibility criteria a database was compiled from cyclic triaxial (CTX) tests performed mainly on fine-grained soils. Database provided by Pekcan (2001), Sancio (2003) and Bilge (2009) were evaluated. Database obtained from Sancio (2003) was reassessed in order to prevent any inconsistency, since the excess pore pressure ratios were reported to be more than 1.00, while database of Pekcan (2001) and Cetin and Bilge (2010) were directly taken into consideration. A database is compiled from 148 CTX result for the evaluation of previous liquefaction susceptibility criteria. Previous liquefaction susceptibility criteria (Seed et al, 2003; Bray and Sancio, 2006; Boulanger and Idriss, 2004, 2006) were evaluated with this database by considering various liquefaction definitions. The results revealed that these criteria are inadequate to capture the differentiating trends between potentially liquefiable and non-liquefiable soils with their current parameter selections and domains. It strengthens the need for an improved criteria.

High quality database is the main requirement of an empirical or a semi empirical model. Therefore, for development of correlations most of the questionable data of Sancio (2003) was eliminated, only 15 CTX test data out of 93 were considered. Additionally, CTX test results performed on sands by Wu et al. (2003) and Bilge (2005) were taken into consideration to be able to develop a unique correlation for sands and clays. These databases were also examined carefully to eliminate questionable data. Compiled database for development of correlation consists of data from 158 CTX tests providing 2829 r_u vs. γ data points for development of correlations. This database represents a variety of soils types which can be classified as SW, SP, CL, CH, CL-ML, ML and M according USCS. Therefore it covers a wide range of Atterberg limits and natural water content ($0< PI< 60$, $0 <LL< 95$, $0< w_c/LL< 1.50$) which are selected as the descriptive variables of the likelihood function.

Cyclic behaviour of soils had been generally explained by the degradation of stiffness under dynamic loading. Based on available modulus degradation curves in the literature, a relation was established between excess pore pressure ratios (r_u) and double amplitude maximum shear strain (γ_{max}). Similar to degradation behaviour, i) a non-linear increase in excess pore pressure ratio with increasing shear strain after a threshold value and ii) a downward shift in boundary curves with increasing plasticity was established causing increasing cyclic resistance. Thus r_u vs. γ domain was selected for the development of likelihood functions.

Compiled database of 2829 data points from 158 CTX tests were plotted on r_u vs. γ domain, they have shown the established behaviour based on modulus degradation with clear trends. Thus, a likelihood function of soils was attempted to be established relating excess pore pressure ratio to double amplitude shear strain while considering other descriptive variables such as PI, LL, w_c/LL. In this regard, likelihood functions were developed using maximum likelihood principal considering various functional forms with three different sets of descriptive variables. Likelihood function with maximum likelihood and minimum standard deviation was achieved for descriptive variable set consisting of PI, LL and w_c/LL.

The likelihood function derived provides a closed form solution for the estimation of excess pore pressure ratio corresponding to a double amplitude shear strain level. Rather than only considering soil index parameters, this correlation also accounts for the amount of accumulated shear strain which is related to amplitude and duration of cyclic loading. Moreover, this correlation does not depend on any liquefaction definition. It is able to differentiate “cyclic liquefaction” from “cyclic mobility” type soil responses owing to its ability to provide cyclic shear strain induced excess pore water pressure. Thus, the proposed framework has answers to previously encountered limitations.

New liquefaction susceptibility criteria were developed considering a liquefaction definition, which is based on the observations of cyclic behavior of fine-grained soils. Boundaries defined by these solution were used to back-calculate the soil index parameters satisfying these conditions via proposed likelihood function. While developing the liquefaction susceptibility boundaries, the validity limit of the proposed likelihood function was also taken into account. In this regard, new liquefaction susceptibility criteria was developed for different w_c/LL conditions.

The boundary curves assessed by this new methodology were then mapped into CPT domain in consistence with the study of Cetin and Ozan (2009). Accordingly, new chart solutions were developed in CPT domain, enabling estimation of excess pore pressure ratio level based on CPT measurements in normalized net cone tip resistance ($q_{t,1,net}$) vs. friction ratio (F_R) domain. Additionally, the liquefaction susceptibility criteria developed previously was mapped on CPT domain which enables estimation of liquefaction potential of fine-grained soils on this domain.

8.2 Conclusions

The new methodology presented in this study is considered to represent a robust and defensible basis for assessment of liquefaction susceptibility of fine-grained soils. It has number of significant advantages over available liquefaction susceptibility criteria. These include:

1. In prior studies, an agreement concerning the definition of liquefaction does not exist. Each study set its own liquefaction definition for the development of their criteria. Thus, these criteria were not applicable for different definitions of liquefaction. However, the framework, proposed herein, did not depend on any liquefaction definition, but it provides the level of excess pore pressure ratio corresponding to a shear strain level.

2. Previous liquefaction susceptibility criteria do not consider the level of cyclic loading. They do not provide any information regarding the rate and duration of cyclic loading for which these criteria are valid. Modulus degradation curves in accordance with the curves developed in this study showed that even high plasticity clays can generate excess pore pressure ratios close to 1.00 if cyclic loading is applied long enough. Thus, their applicability for any loading amplitude is questionable. This study considers the level of cyclic loading through level of shear strain.

Various researchers in the literature presented the relation of accumulated shear strain level with magnitude and duration of cyclic loading (Tokimatsu and Seed, 1984; Ishihara and Yoshimine, 1992; Wu et al., 2003; Cetin et al., 2009). Shear strain corresponding to scenario earthquake of the site can be predicted by one of these correlations and excess pore pressure ratio corresponding to this shear strain level can be estimated through use of the methodology proposed in this study.

3. Previous researchers consider only “cyclic liquefaction” and they have failed to differentiate “cyclic mobility” type soil response.. However, these different soil responses can be clearly differentiated with the proposed methodology, since it provides excess pore pressure ratio for any shear strain level.

4. The error associated with classifying liquefaction susceptibility behaviour of soils by the proposed framework was found to be significantly less than the errors of existing criteria. This framework was also used to generate the boundaries of proposed liquefaction susceptibility criteria, based on the conditions of liquefaction, which was defined based on observed cyclic behaviour of fine-grained soils on r_u vs. γ_{max} domain.

5. Idriss and Boulanger criterion is concluded to be unconservatively biased, since there exist significant number of cases with PI higher than 7 and are still liquefiable at reasonably low CSR values.

6. Bray and Sancio criterion being independent of LL makes it impossible to differentiate high plastic vs low plastic responses.

7. Seed et al. criterion is not a continuous function of w_c/LL, thus it can not model any significant effect of small changes in this ratio.

8. This study, by providing a closed form, continous solution founded on probabilistically-based, well defined and widely accepted r_u vs. γ_{max} response is believed to provide a superior alternative than existing solutions.

9. This study also provides chart solutions on CPT domain which enables estimation of excess pore pressure ratio from normalized net tip resistance and friction ratio. These charts were provided for double amplitude shear strains of 3, 5 and 7.5%. Using the results of Cetin and Ozan (2009), this study eliminates the vital need of soil sampling, which is the most significant drawback of CPT, to some extent. Additionally developed liquefaction susceptibility criteria was similarly mapped on CPT domain, providing liquefaction susceptibility assessment of fine grained soils based on CPT test results.

REFERENCES

Andrews, D. C., & Martin, G. R. (2000). Criteria for liquefaction of silty soils. *12th World Conf. on Earthquake Engineering* (p. Paper No. 0312). Upper Hutt, New Zealand: NZ Soc. for EQ Engrg.

Bilge, T. B. (2005). Volumetric and deviatoric strain behavior of cohesionlesssoils under cyclic loading. *Msc thesis* . Ankara, Turkey: Middle East Technical University.

Boulanger, R. W., & Idriss, I. M. (2004). *Evaluating the liquefaction or cyclic failure of silts and clays.* Univ. of Calif., Davis, Calif., Center for Geotech. Modeling.

Boulanger, R. W., & Idriss, R. W. (2006). Liquefaction Susceptibility Criteria for Silts and Clays. *J. of Geotech. and Geoenviron. Eng. , 132:11*, 1413-1424.

Boulanger, R. W., Meyers, M. W., Meija, L. H., & Idriss, I. M. (1998). Behavior of a fine-grained soil during the Loma Preita earthquake. *Canadian Geotechnical J , V35*, 146-158.

Bray, J. D., & Sancio, R. B. (2006). Assessment of the liquefaction susceptibility of fine-grained soils. *J. Geotech. Geoenviron. Eng. , 132*, 1165-1177.

Bray, J. D., Sancio, R. B., Durgunoglu, H. T., Onalp, A., Seed, R. B., Stewart, J. P., et al. (2001, August 24-25). Ground Failure In Adapazari, Turkey. *Proceedings of Earthquake Geotechnical Engineering Satellite*

Conference of the XVth International Conference on Soil Mechanics & Geotechnical Engineering . Istanbul, Turkey.

Bray, J. D., Sancio, R. B., Reimer, M. F., & Durgunoglu, H. T. Liquefaction Susceptibility of fine-grained soils. *proc., 11th Int. Conf. On Soil Dynamics and Earthquake Engineering and 3rd Int. Conf. On Earthquake Geotechnical Engineering. 1*, pp. 655-662. Berkeley, California: Stallion Press.

Castro, G., & Christian, J. T. (1976). Shear strength of soils and cyclic loading. *J Geotech Engng , 102* (9), 887-894.

Cetin, K. O. (2000, May). Reliability-based assessment of seismic soil liquefaction initiation hazard. *Ph.D. thesis* . University of California, Berkeley.

Cetin, K. O., & Isik, N. S. (2007). Probabilistic assessment of stress normalization for CPT data. *J. of Geotech and Geoenvir. , 133* (7), 887-897.

Cetin, K. O., & Ozan, C. (2009). CPT-based probabilistic soil characterization and classification. *ASCE J. of Geotech. and Geoenvir. Eng. , 135*, 46-61.

Cetin, K. O., Bilge, H. T., Wu, J., Kammerer, A. M., & Seed, R. B. (2009). Probabilistic models for cyclic straining of saturated clean sands. *J. of Goetech. and Geoenviron. Eng. , 135* (3), 371-386.

Cetin, K. O., Der Kiureghian, A., & Seed, R. B. (2002). Probabilistic models for the initiation of seismic soil liquefaction. *Journal of Structural Safety , 24* (1), 67-82.

Darendeli, M. (2001). Development of a new family of normalized modulus reduction and material damping curves. *Ph.D. thesis* . Texas: Dept. of Civil Engineering, Univ. of Texas, Austin.

Finn, L. W., Ledbetter, R. H., & Guoxi, W. U. (1994). Liquefaction in silty soils: design and analysis. *Ground failures under siesmic conditions, Geotechnical Special Publication. 44*, pp. 51-79. Reston: ASCE.

Gratchev, I. B., Sassa, K., & Fukuoka, H. (2006). How reliable is the plasticity for estimating liquefaction potential of clayey sands? *J. Geotech. Geoenviron. Eng. , 132* (1), 124-127.

Hardin, B. O., & Drnevich, V. P. (1972). Shear modulus and damping in soils: design equations and curves. *J. Soil Mech. and Found. Div. , 98* (7), 667-692.

Hardin, B. O., & Drnevich, V. P. (1972). Shear modulus and damping in soils: design equations and curves. *J. Soil Mech. and Found. Div. , 98* (7), 667-692.

Hayati, H., & Andrus, R. D. (2008). Liquefaction susceptibility of fine-grained soils in Charleston, South Carolina Based on CPT. *GeoCongress 2008: Geosustainability and Geohazard Mitigation* (pp. 327-334). ASCE.

Holzer, T. L., Bennett, M. J., Ponti, D. J., & Tinsley, J. C. (1999). Liquefaction and soil failure during 1994 Northridge earthquake. *J. Geotech. Geoenviron. , 125*, 438-452.

Hsieh, D., Manski, C. F., & McFadden, D. (185). Estimation of Response Probabilities from augmented retrospective observations. *Journal of the American Statistical Association , 80* (109).

Hsu, C.-C., & Vucetic, M. (2006). Threshold shear strain for cyclic pore-water pressure in cohesive soils. *J Geotech Geoenviron Eng , 132* (10), 1325-1335.

Idriss, I. M., Singh, R. D., & Dobry, R. (1978). Nonlinear Behavior of Soft Clays during Cyclic Loading. *Journal of the Geotechnical Engineering , 104* (12), 1427-1447.

Ishibashi, I., & Zhang, X. (1993). Unified dynamic shear moduli and damping ratios of sand and clay. *SOILS FOUND. , 33* (1), 182-191.

Ishihara, K. (1986). Evaluation of soil properties for use in earthquake response analysis. *Geot. Mod. Earthq. Eng. Pr.* , 241–275.

Ishihara, K., & Yoshimine, M. (1992). Evaluation of settlements in sand deposits following liquefaction during earthquakes. *Soils Found. , 32* (1), 861-878.

Koester, J. P. (1992). The influence of test procedure on correlation of Atterberg limits with liquefaction in fine-grained soils. *Geotech. Test. J. , 15* (4), 352-361.

Krizek, R. J., McLean, F. G., & Giger, M. W. (1974). Effect of particle characteristics on wave velocity. *J. Geotech Engrg Div , 100* (1), 89-94.

Kuribuyashi, E., Iwaski, T., & Tatsuoka, F. (1974). Effects of stress conditions on dynamic properties of sands. *Bulletein Intl. of Seismology and Earthquake Engineering , 12*.

Li, D. K., Juang, C. H., Andrus, R. D., & Camp, W. M. (2007). Index properties-based criteria for liquefaction susceptibility of clayey soils: A critical Assessment. *J. Geotech Geoenviron. Eng. , 133* (1), 110-115.

Liu, A. H., Stewart, J. P., Abrahamson, N. A., & Moriwaki, Y. (2001). Equivalent number of uniform stress cycles for soil liquefaction analysis. *J. Geotech. Geoenviron. Eng , 127* (12), 1017-1026.

Lunne, T., Robertson, P. K., & Powell, J. J. (1997). Cone Penetration Testing in Geotechnical Practice. Blackie Academic & Professional.

Manski, C. F., & Lerman, S. R. (1977). The Estimation c/f Choice Probabilities from Choice-Based samples. *Econometrica , 45* (8), 88.

Matasovic, N., & Vucetic, M. (1995). Generalized cyclic degradation-pore pressure generation model for clays. *J Geotech Engng , 121* (1), 33-42.

Moss, R. E., Seed, R. B., & Olsen, R. S. (2006). Normalizing the CPT for overburden stress. *J. Geotech. Geoenviron. Eng , 132* (3), 378 – 387.

Olsen, R. S., & Mitchelle, J. K. (1995). CPT stress normalization and prediction of soil classification. *Proc., Int. Symp. on Cone Penetration Testing, CPT 95*, *2*, pp. 257-262. Linkoping, Sweeden.

Pekcan, O. (2001). Cyclic behaviour of Adapazarı clayey silts. *Msc thesis* . Ankara, Turkey: Middle East Technical University.

Perlea, V. G. (2000). Liquefaction of cohesive soils. *Soil dynamis and liquefaction 2000, The 2000 Speciality Conf.* (pp. 58-75). Denver: ASCE geotechnical special publications.

Robertson, P. K. (1999). Estimation of minimum undrained shear strength for flow liquefaction using the CPT. *2nd International Conference on Earthquake Geotechnical Engineering.* Lisbon.

Robetson, P. K., & Wride, C. E. (1998). Evaluating cyclic liquefaction potential using cone penetration test. *Can. Geotech. J. , 35* (3), 442-459.

Sancio, R. B. (2003). Ground Failure and Building Performance in Adapazari, Tukey. *Ph.D. Dissertation* . University of California, Berkeley.

Sancio, R. B., Stewart, J. P., Youd, T. L., Durgunoglu, H. T., Onalp, A., Seed, R. B., et al. (2002). Correlation between ground failure and soil condition in Adapazari, Turkey. *Soil Dynamics and Earthquake Engineering , 22*.

Seed, H. B., & Idriss, I. M. (1982). Ground motions and soil liquefaction during earthquakes. Berkeley, California: EERI Monograph.

Seed, H. B., & Idriss, I. M. (1970). *Soil moduli and damping factors for dynamic response analysis.* Univ. of California, Berkeley, Earthquake Engineering Research Center.

Seed, H. B., Wong, R. T., Idriss, I. M., & Tokimatsu, K. (1986). Moduli and damping factos for dynamic analyses of cohesionless soils. *Journal of Goetech. Eng. , 112* (11), 1016-1032.

Seed, R. B., et al. (2003). Recent advances in soil liquefaction engineering: A unified consistent framework. Berkely, California: Earthquake Engineering Research Institute.

Sun, J. I., Golesorkhi, R., & Seed, H. B. (1988). *Dynamic moduli and damping ratios for cohesive soils.* University of California,Berkeley. Earthquake Engineering Research Center.

Thiers, G. R., & Seed, H. B. (1968). Cyclic stress–strain characteristics of clay. *Journal of the Soil Mechanics and Foundations Division , 94*, 555–569.

Tokimatsu, K., & Seed, H. B. (1984). *Simplified procedures for the evaluation of settlements in clean sands.* Earthquake Engineering Research Center, College of Engineering Univ. of California, Berkeley, Calif.

Vucetic, M., & Dobry, R. (1991). Effect of Soil Plasticity on Cyclic Response. *Journal of Geotechnical Engineering Division , 117* (1), 89-107.

Wang, W. (1984). Earthquake Damages to Earth Dams and Levees in Relation to Soil Liquefaction. *Proc., Int. Conf on Case Histories in Geotechnical Engg.* (pp. 512-522). Rolla: University of Missouri.

Wang, W. (1981, September 11-16). Foundation Problems in Aseismatic Design of Hydraulic Structures. *In Proceedings of Joint US- PRC Microzonation Workshop* . Harbin, PRC.

Wang, W. (1979). Some Findings in Soil Liquefaction. *Report Water Conservancy and Hydro-electric Power Scientific Research Institute* (pp. 1-17). Beijing, China: Report Water Conservancy and Hydro-electric Power Scientific Research Institute.

Wu, J., Seed, R. B., & Pestana, J. M. *Liquefaction triggering and post liquefaction deformation of Monterey 0/30 sand under unidirectional* *cyclic simple shear loading.* Geotechnical Engineering Research Report, Univ. of Califoria, Berkeley, Calif.

Zavoral, D. C., & Campanella, R. G. (1994). Frequency effects on damping/modulus of cohesive soils. *ASTM Special Technical Publication No. 1213* , 191-201. (V. P. R. J. Ebelhar, Ed.) Philadelphia.

Zhang, G., Robertson, P. K., & Brachman, R. W. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground. *Can. Geotech. J. , 35* (5), 1168-1180.

.

APPENDIX A

RESULTS OF EVALUATIONS OF AVAILABLE LIQUEFACTION SUSCEPTIBILITY CRITERIA WITH COMPILED DATABASE

Available liquefaction susceptibility criteria were evaluated by considering different conditions for defining liquefaction within the confines of this study. 12 different definitions were used for evaluation; (i) γ = 3.5% , $r_u \geq 0.7$, (ii) γ = 3.5% , $r_u \geq 0.8$, (iii) γ = 3.5% , $r_u \geq 0.9$, (iv) γ = 3.5% , $r_u \geq 1.0$, (v) γ = 5.0% , $r_u \geq 0.7$, (vi) γ = 5.0% , $r_u \geq 0.8$, (vii) γ = 5.0% , $r_u \geq 0.9$, (viii) γ = 5.0% , $r_u \geq 1.0$, (ix) γ = 7.5% , $r_u \geq 0.7$, (x) γ = 7.5% , $r_u \geq 0.8$, (xi) γ = 7.5% , $r_u \geq 0.9$, (xii) γ = 7.5% , $r_u \geq 1.0$. Among these evaluations, only the results of definition corresponding to (vi) γ = 5.0% , $r_u \geq 0.8$ are discussed in Chapter 4 and results of evaluations for each liquefaction criteria considering other 11 definition will be discussed in Appendix A.

i) $\gamma = 3.5\%$, $r_u \geq 0.7$

a) Seed et al. (2003)

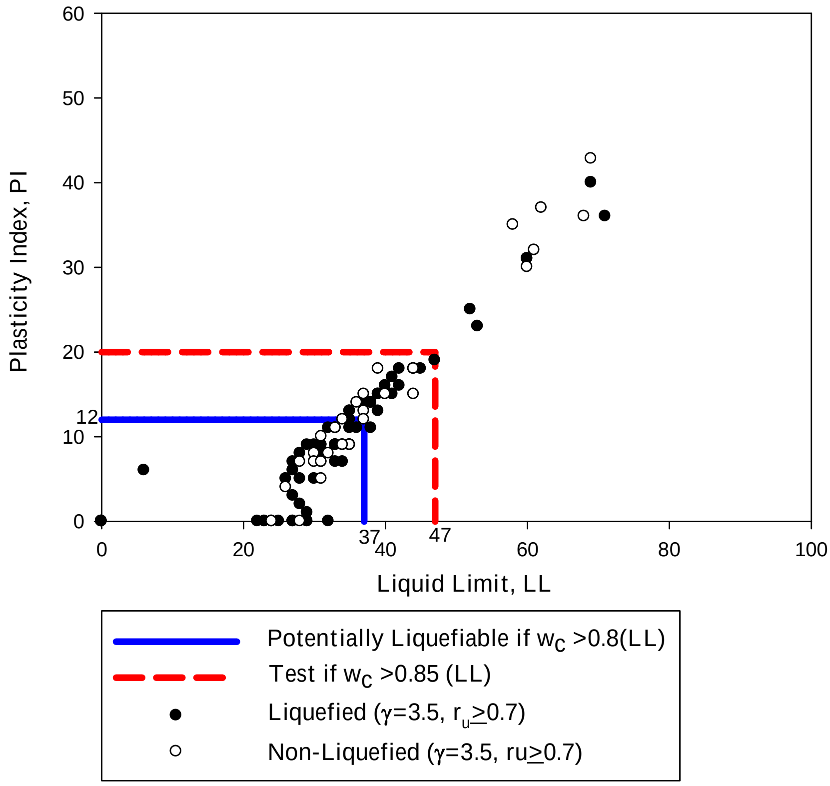

Figure A.1 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq 0.7$ condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.1 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.7$

γ=3.5 %, $r_u \geq 0.7$			
Observed/ Seed et al.	YES	TEST	NO
YES	39	18	7
NO	16	5	14

b) Bray and Sancio (2006)

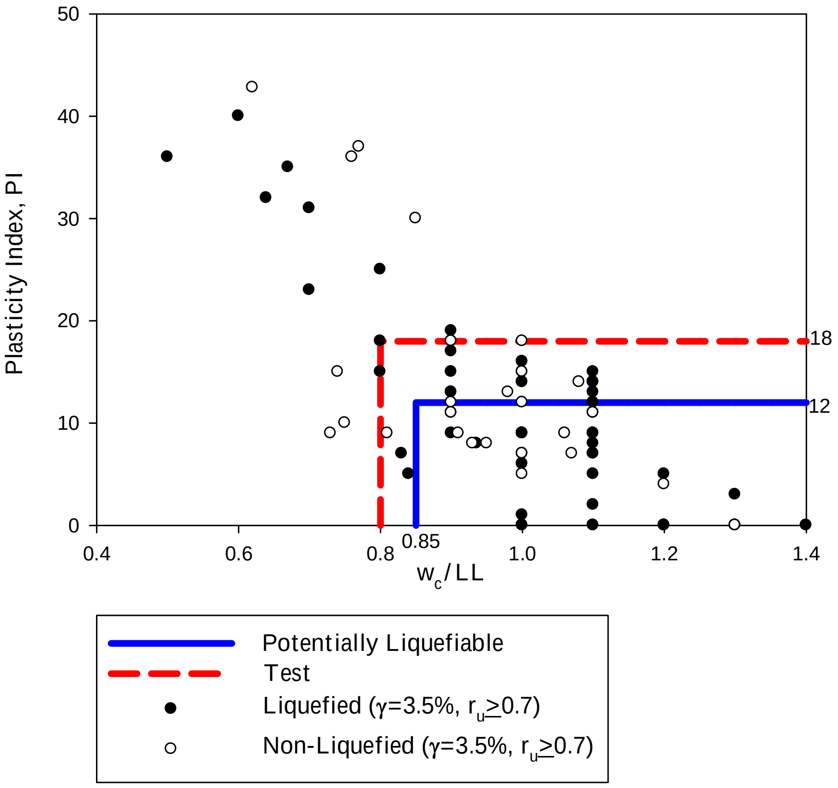

Figure A.2 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.7 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.2 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.7$

γ=3.5 %, r_u≥0.7			
Observed/ Bray & Sancio	YES	TEST	NO
YES	38	17	9
NO	14	6	15

c) Boulanger and Idriss (2004, 2006)

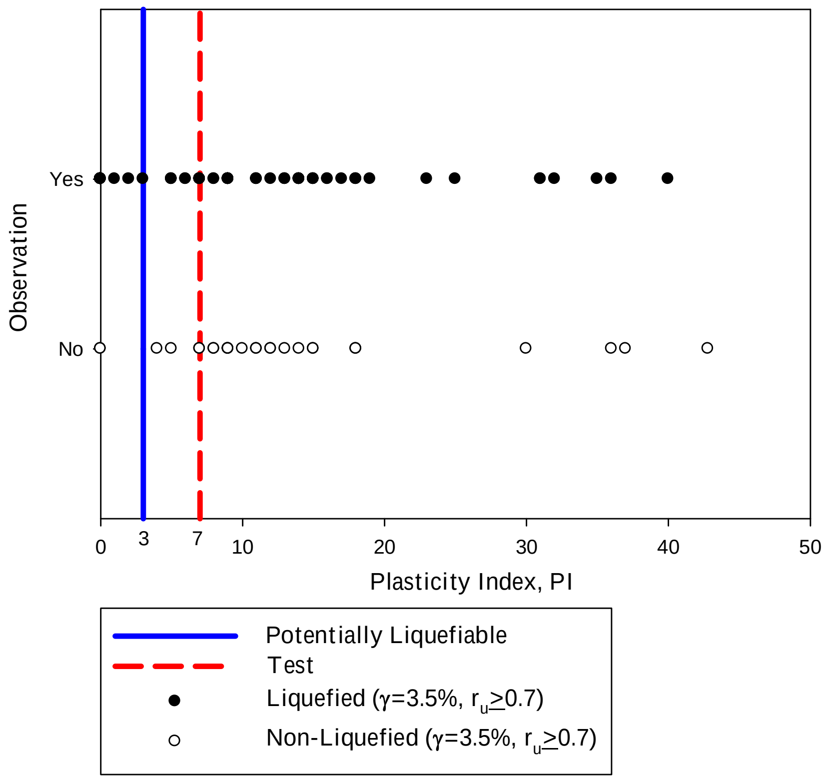

Figure A.3 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.7 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.3 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.7$.

γ=3.5 %, $r_u \geq 0.7$			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	20	9	40
NO	2	5	28

ii) γ **= 3.5% , $r_u \geq$ 0.8**

a) Seed et al. (2003)

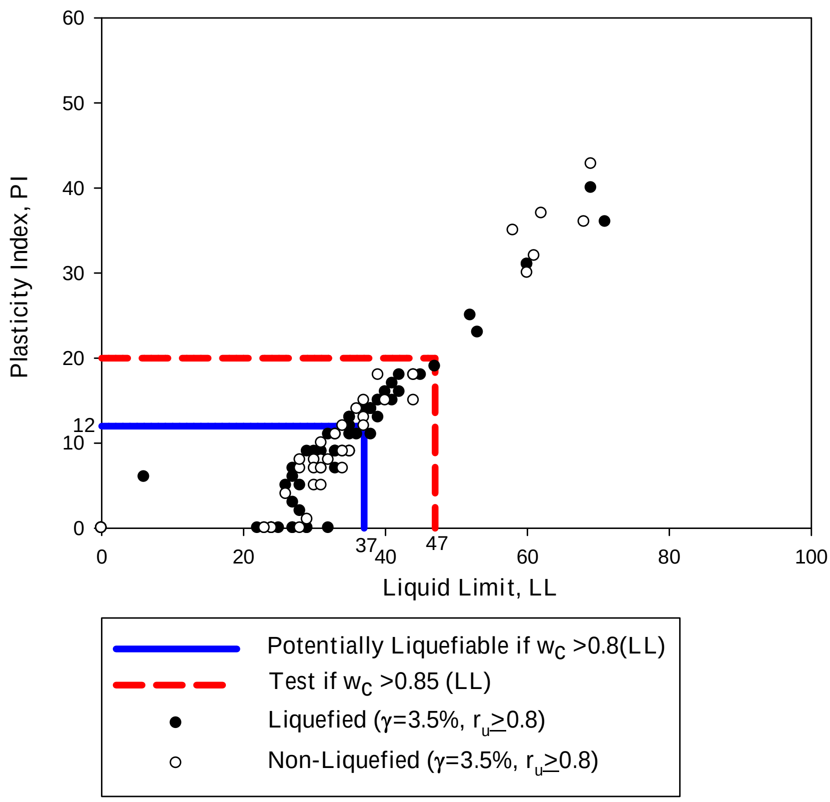

Figure A.4 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.8 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.4 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq$ 0.8.

γ=3.5%, $r_u \geq$0.8			
Observed/ Seed et al.	YES	TEST	NO
YES	32	18	7
NO	22	5	14

b) **Bray and Sancio (2006)**

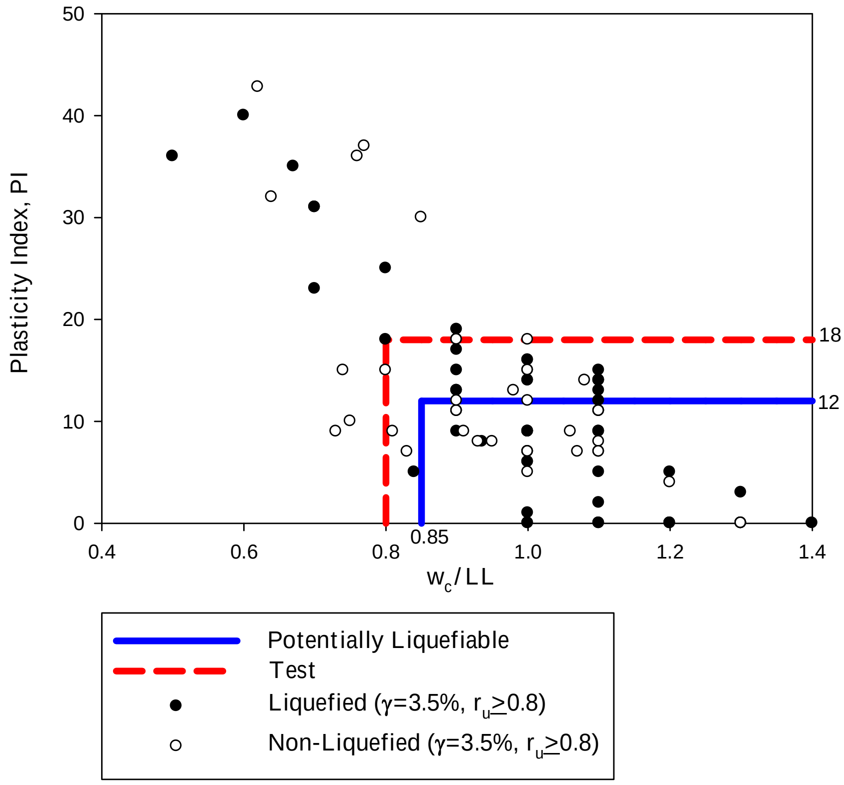

Figure A.5 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.8 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.5 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.80$

γ=3.5%, r_u≥0.8			
Observed/ Bray & Sancio	YES	TEST	NO
YES	32	17	8
NO	19	6	16

c) Boulanger and Idriss (2004, 2006)

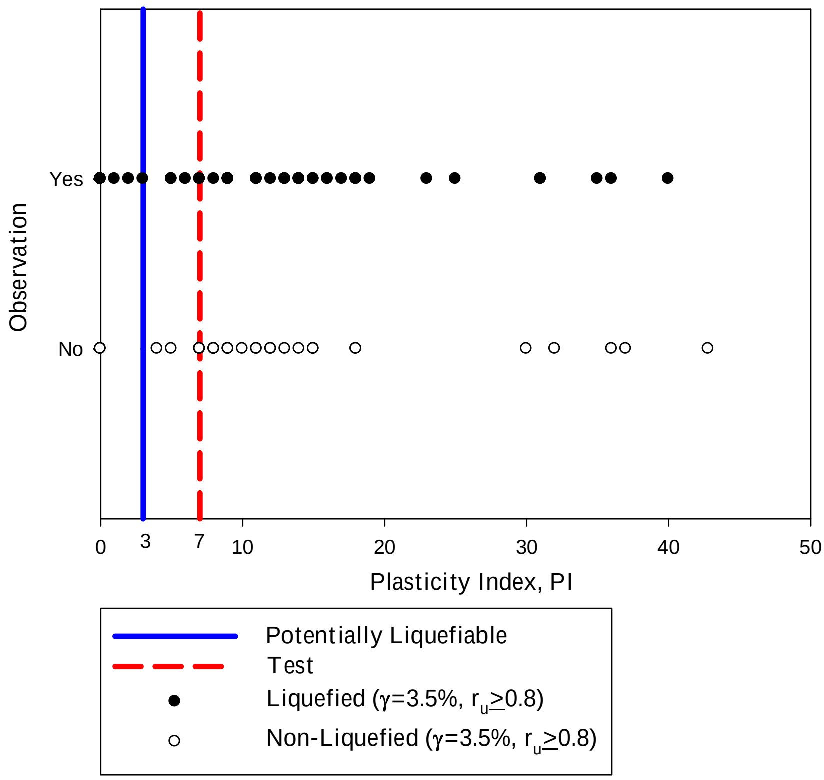

Figure A.6 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.8 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.6 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.8$

γ=3.5 %, $r_u \geq 0.8$;			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	16	7	38
NO	5	7	30

iii) γ = 3.5% , $r_u \geq$ 0.9

a) Seed et al. (2003)

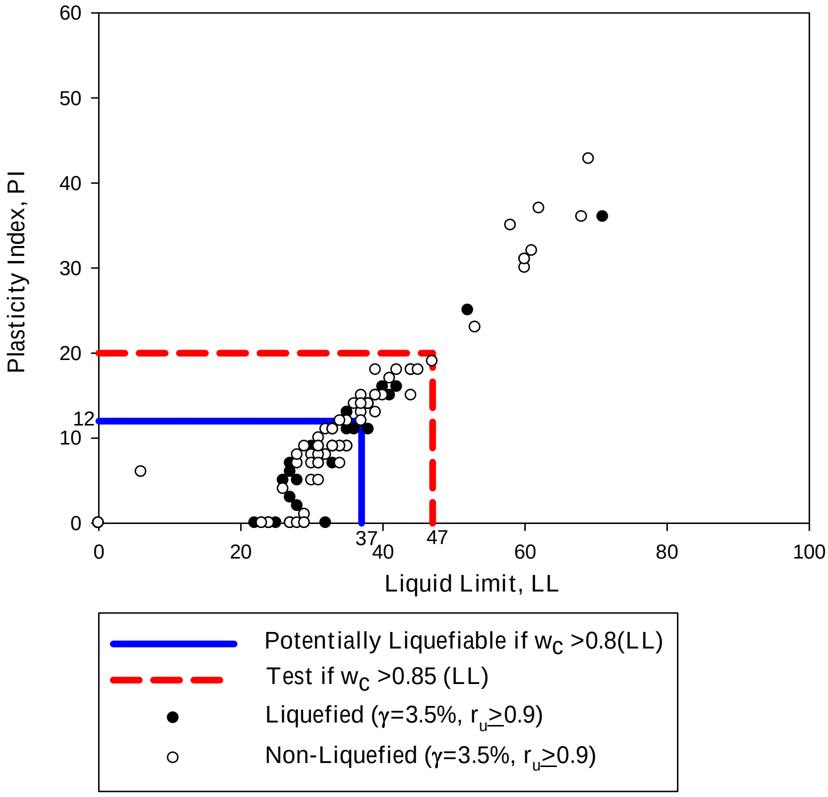

Figure A.7 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.9 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.7 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq$ 0.9.

γ=3.5 %, $r_u \geq$0.9			
Observed/ Seed et al.	YES	TEST	NO
YES	17	9	3
NO	38	14	17

b) Bray and Sancio (2006)

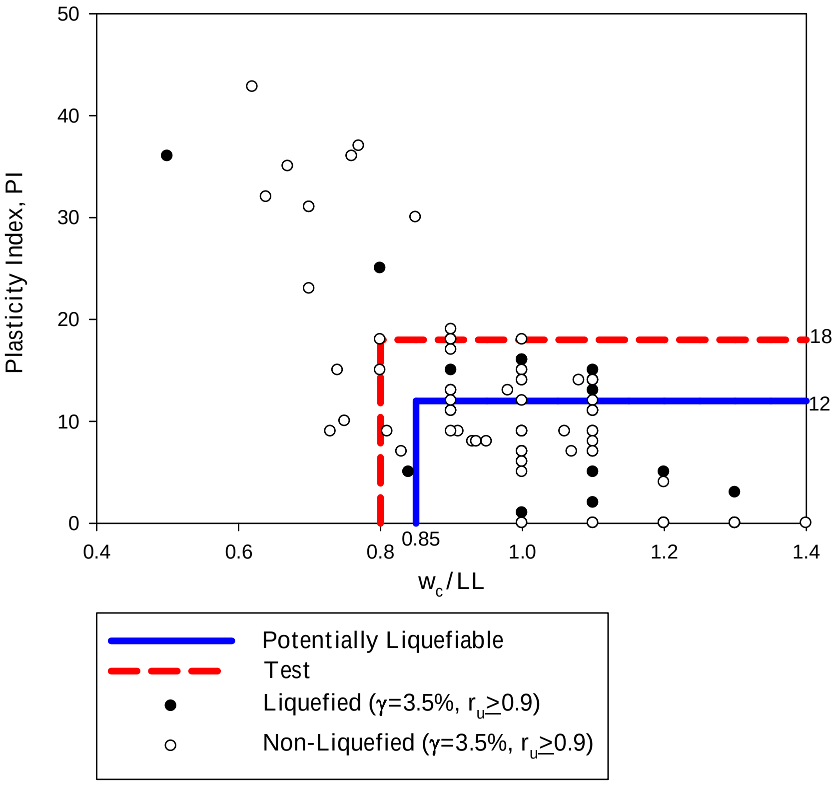

Figure A.8 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.9 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.8 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.9$

γ=3.5 %, $r_u \geq$0.9			
Observed/ Bray & Sancio	YES	TEST	NO
YES	18	8	3
NO	34	15	20

c) Boulanger and Idriss (2004, 2006)

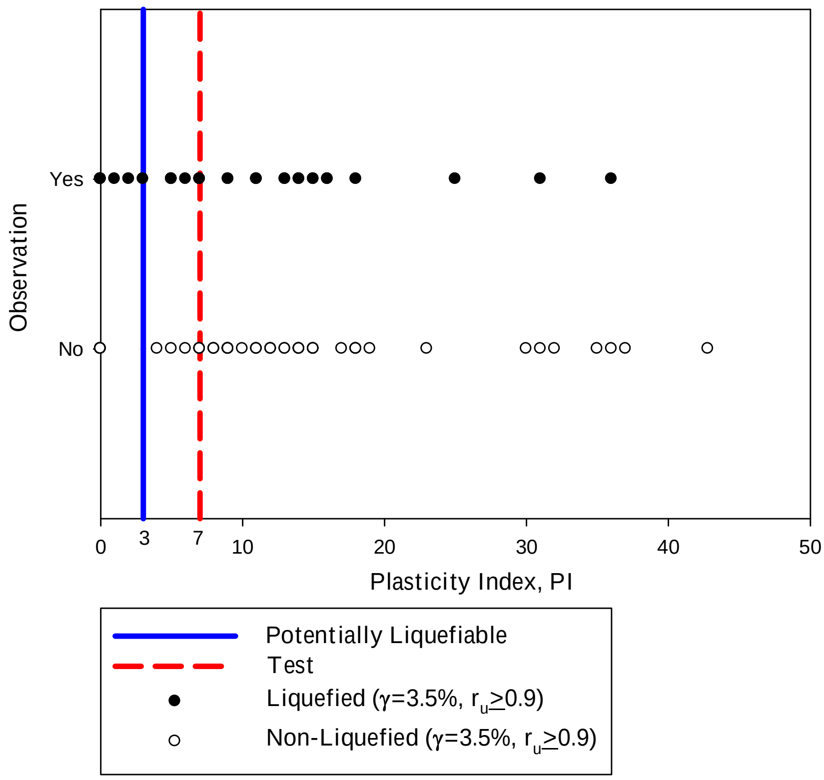

Figure A.9 Evaluation for liquefaction susceptibility condition (γ = 3.5%, $r_u \geq$ 0.9 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.9 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 0.9$

γ=3.5 %, $r_u \geq$0.9			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	8	6	17
NO	14	8	50

iv) γ = 3.5% , $r_u \geq$ 1.0

a) Seed et al. (2003)

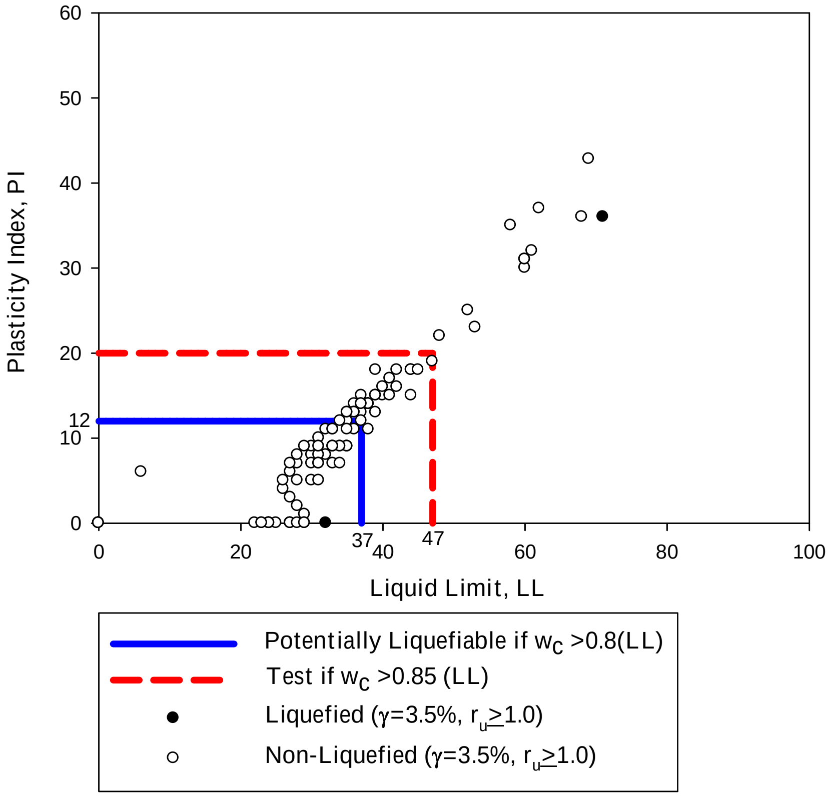

Figure A.10 Evaluation for liquefaction susceptibility condition (γ=3.5%, $r_u \geq$ 1.0 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.10 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq$ 1.0.

γ=3.5 %, $r_u \geq$1.0			
Observed/ Seed et al.	YES	TEST	NO
YES	1	0	1
NO	54	25	20

b) Bray and Sancio (2006)

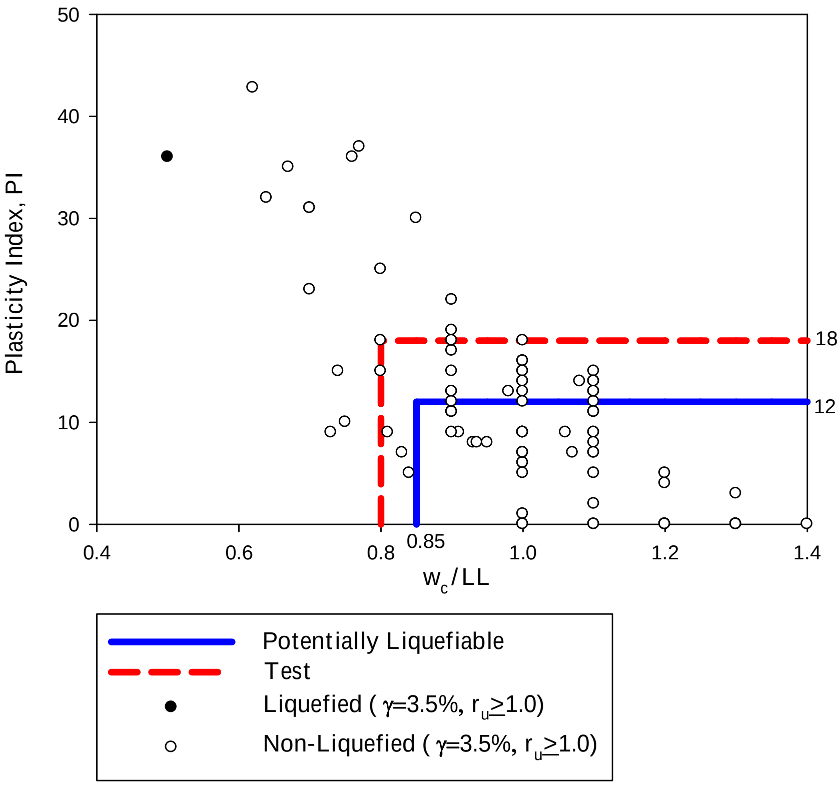

Figure A.11 Evaluation for liquefaction susceptibility condition (γ=3.5%, $r_u \geq$ 1.0 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.11 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 1.0$.

γ=3.5 %, $r_u \geq$1.0			
Observed/ Bray & Sancio	YES	TEST	NO
YES	1	0	1
NO	51	25	23

c) Boulanger and Idriss (2004, 2006)

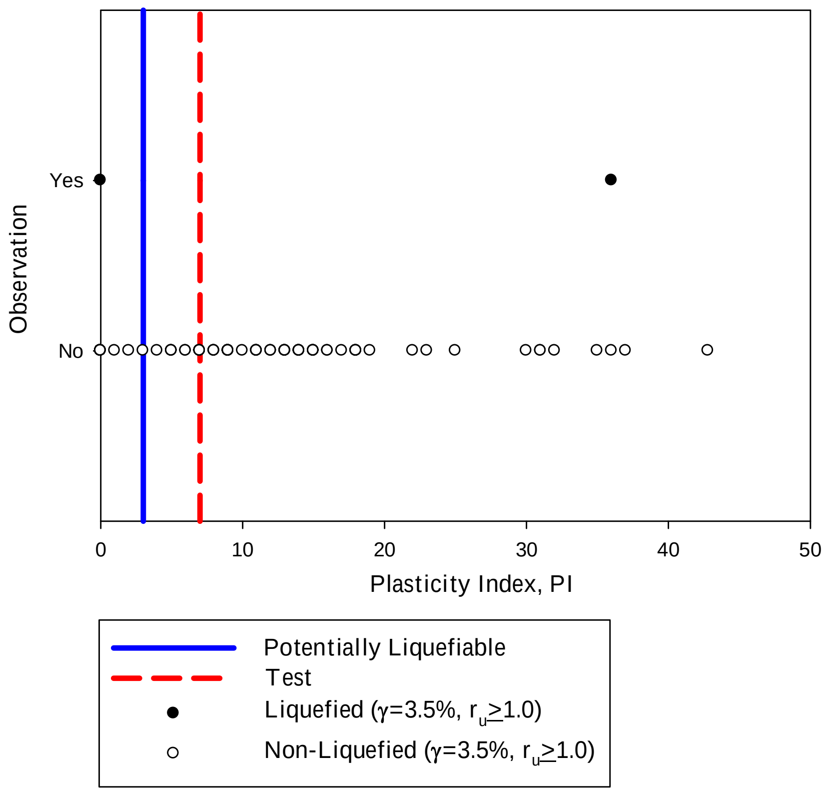

Figure A.12 Evaluation for liquefaction susceptibility condition (γ=3.5%, $r_u \geq$ 1.0 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.12 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 3.5 %, $r_u \geq 1.0$

γ=3.5 %, $r_u \geq 1.0$			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	1	0	1
NO	21	14	69

v) γ = 5.0% , $r_u \geq$ 0.7

a) Seed et al. (2003)

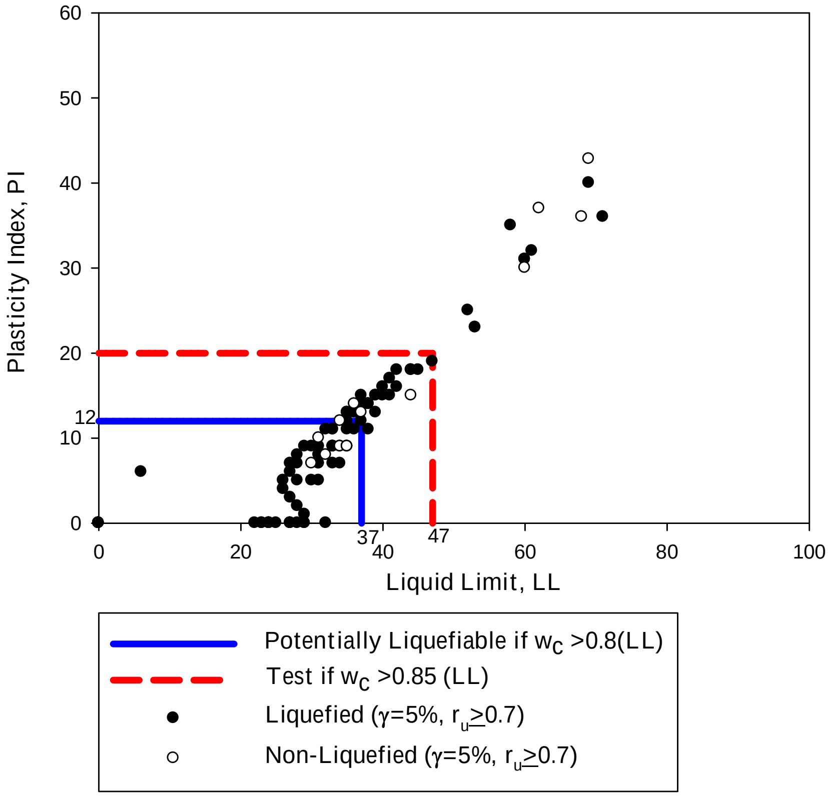

Figure A.13 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.7 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.13 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq 0.7$

γ=5 %, $r_u \geq$0.7			
Observed/ Seed et al.	YES	TEST	NO
YES	48	22	10
NO	7	3	7

b) Bray and Sancio (2006)

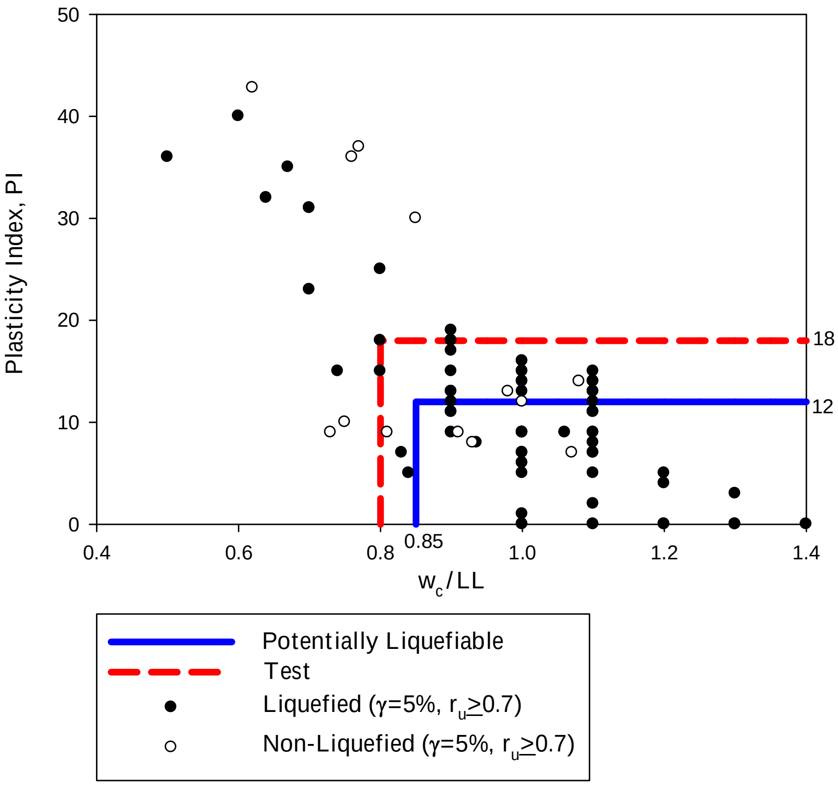

Figure A.14 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.7 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.14 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq 0.7$

γ=5 %, $r_u \geq 0.7$			
Observed/ Bray & Sancio ay	YES	TEST	NO
YES	46	22	12
NO	6	3	8

c) Boulanger and Idriss (2004, 2006)

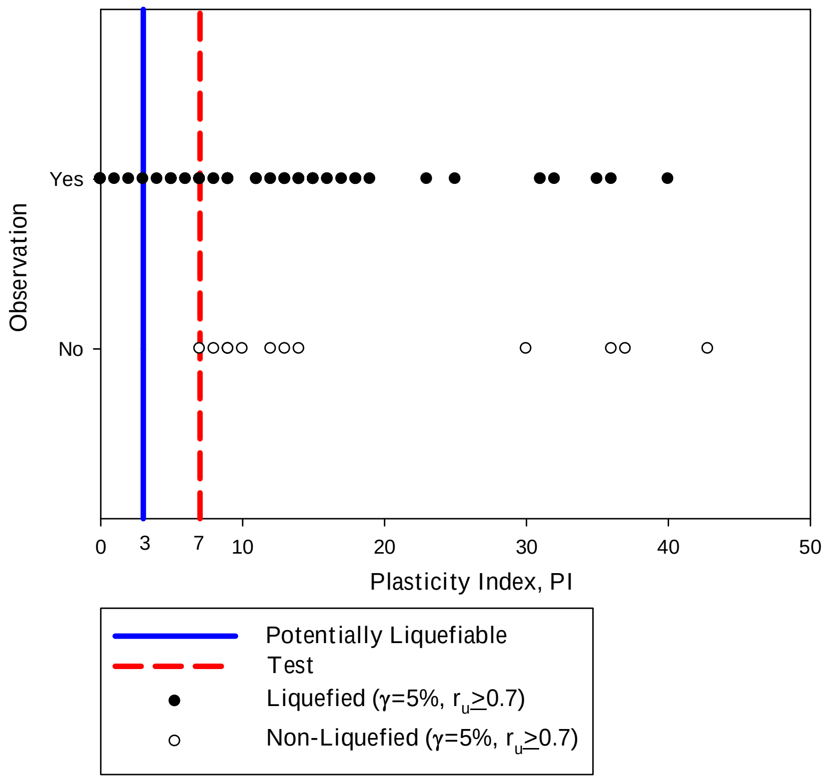

Figure A.15 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.7 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.15 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq 0.7$

γ=5 %, r_u≥0.7			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	22	13	50
NO	0	1	16

vii) γ = 5.0% , $r_u \geq$ 0.9

a) Seed et al. (2003)

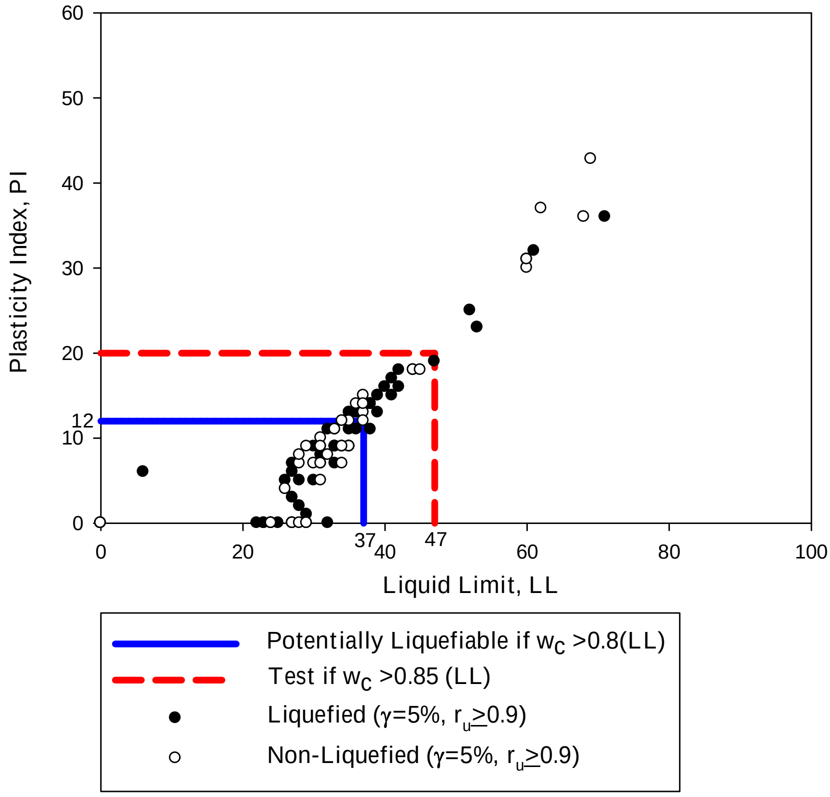

Figure A.16 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.9 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.16 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq$ 0.9.

γ=5 %, $r_u \geq$0.9			
Observed/ Seed et al.	YES	TEST	NO
YES	25	19	4
NO	30	6	12

b) Bray and Sancio (2006)

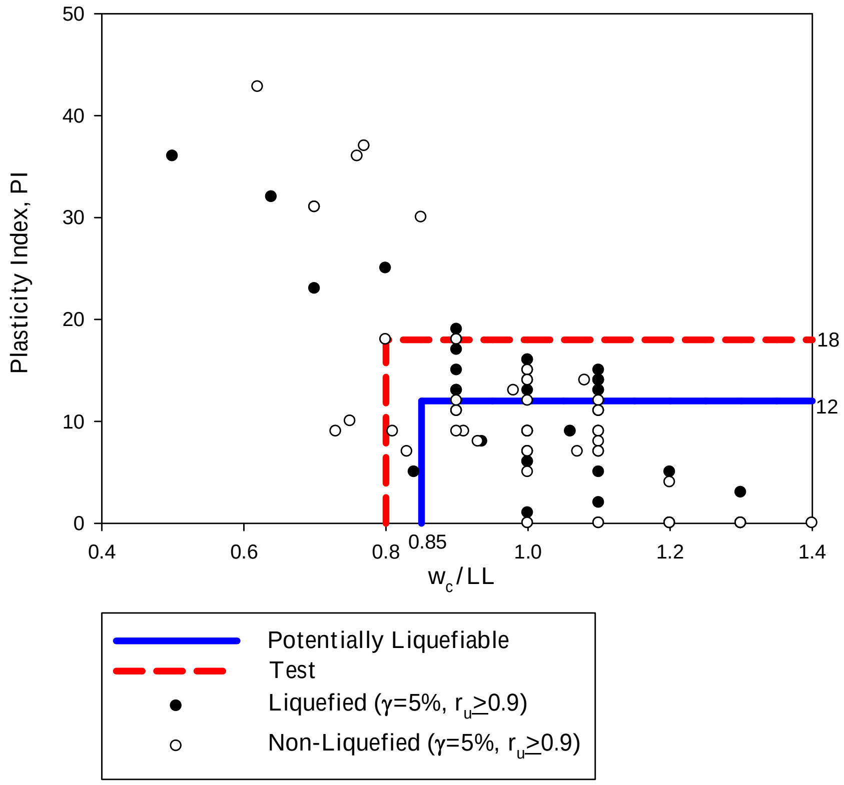

Figure A.17 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.9 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.17 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq$ 0.9.

γ=5 %, $r_u \geq$0.9			
Observed/ Bray & Sancio	YES	TEST	NO
YES	24	17	7
NO	28	8	12

c) Boulanger and Idriss (2004, 2006)

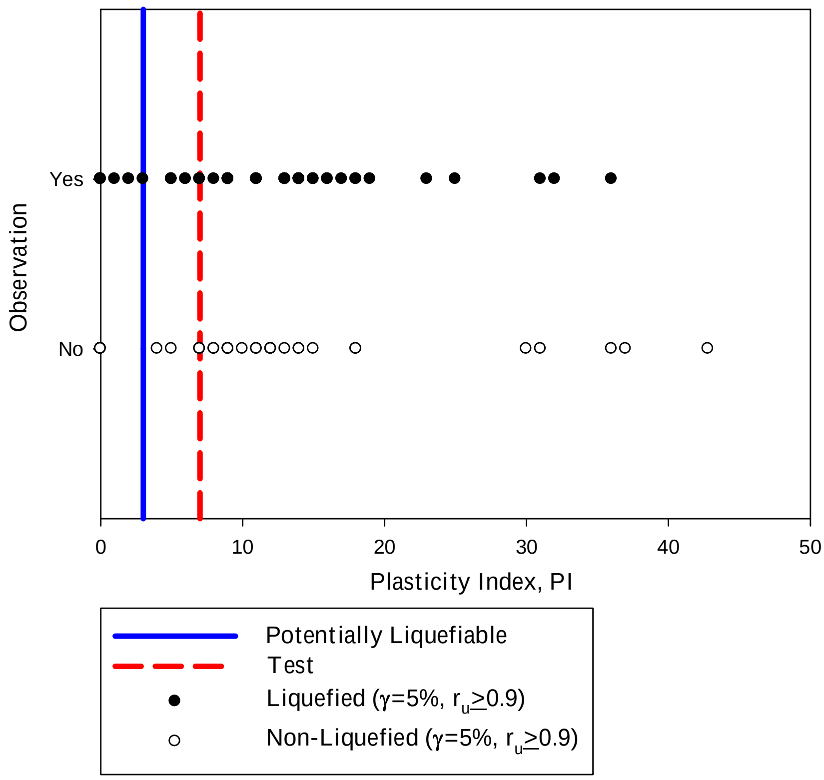

Figure A.18 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.9 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.18 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq$ 0.9

γ=5 %, $r_u \geq 0.9$			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	13	7	32
NO	9	7	33

viii) γ = 5.0% , $r_u \geq 1.0$

a) Seed et al. (2003)

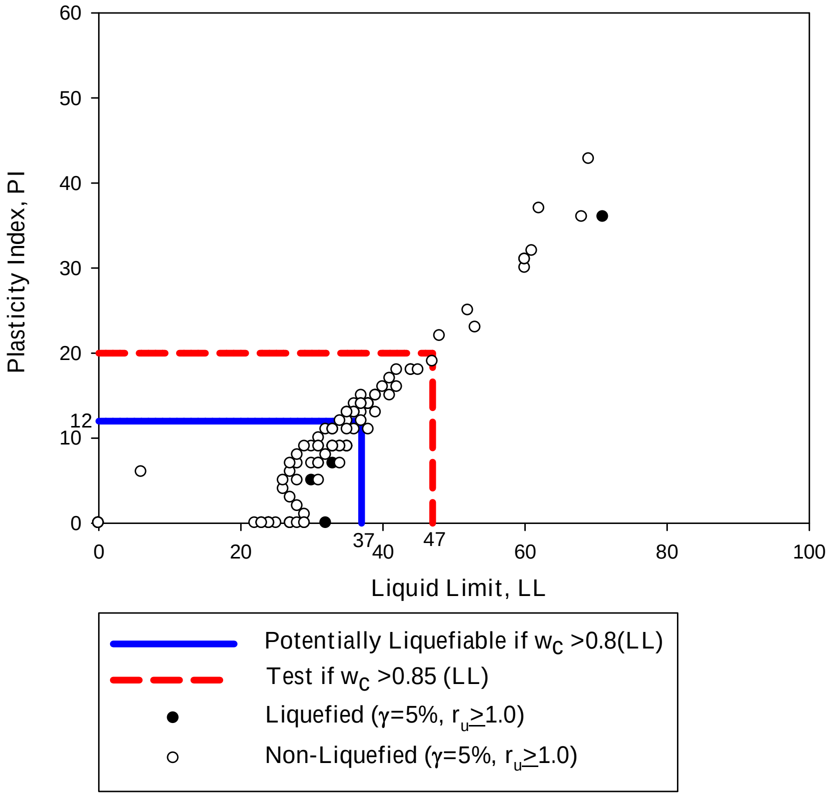

Figure A.19 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 1.0 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.19 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq$ 1.0.

γ=5 %, $r_u \geq 1.0$			
Observed/ Seed et al.	YES	TEST	NO
YES	2	1	1
NO	53	24	16

b) Bray and Sancio (2006)

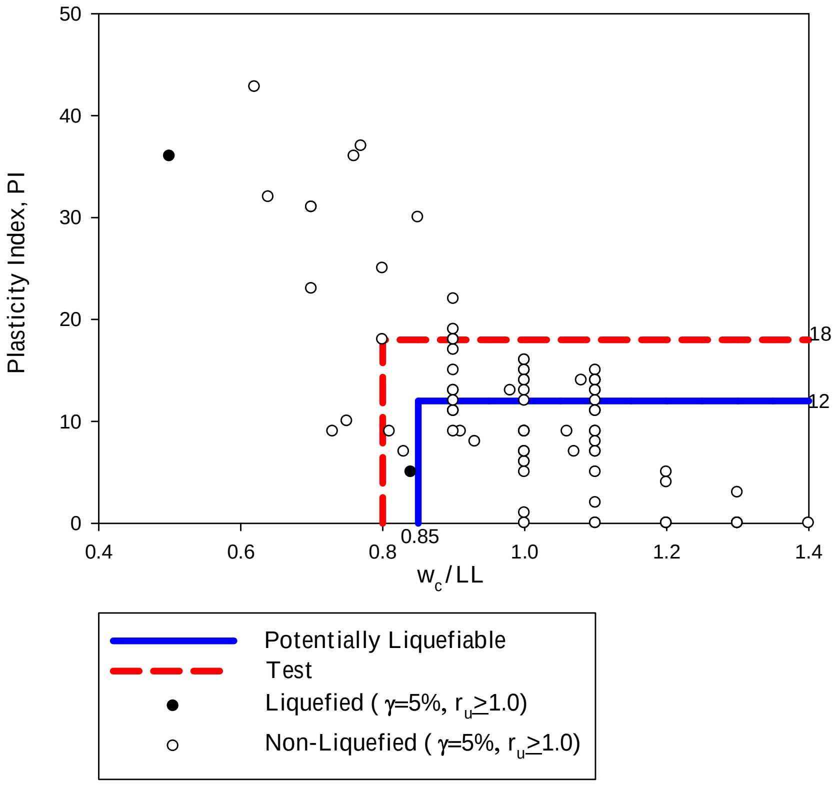

Figure A.20 Evaluation for liquefaction susceptibility condition (γ=5.0%, r_u≥ 1.0 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.20 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, r_u ≥ 1.0.

γ=5 %, r_u≥1.0			
Observed/ Bray & Sancio	YES	TEST	NO
YES	2	1	1
NO	50	24	19

c) **Boulanger and Idriss (2004, 2006)**

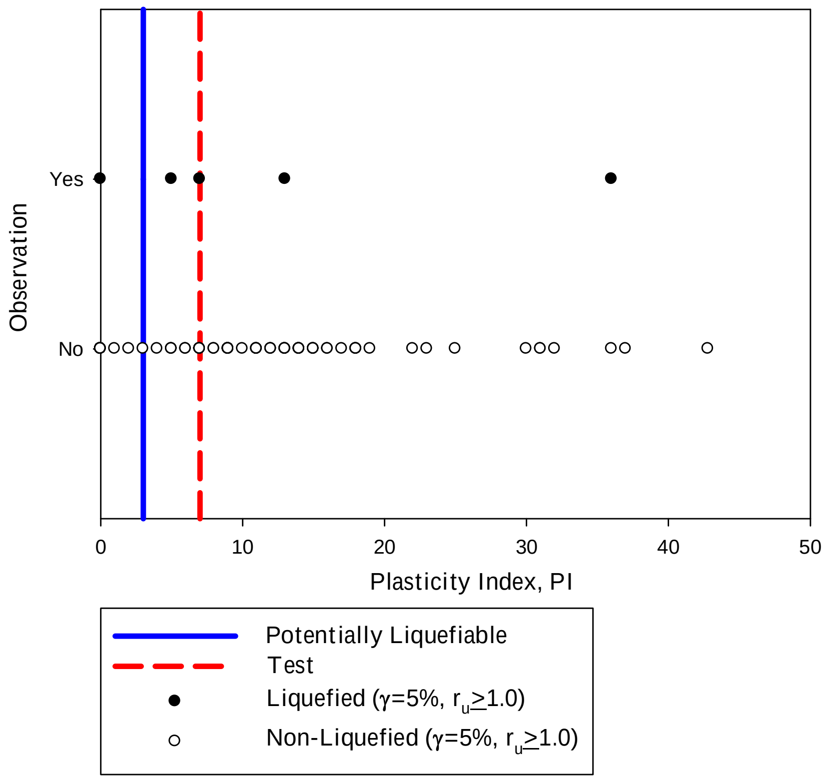

Figure A.21 Evaluation for liquefaction susceptibility condition (γ=5.0%, $r_u \geq$ 0.9 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.21 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 5.0 %, $r_u \geq 1.0$

γ=5 %, r_u≥1.0			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	1	1	2
NO	21	13	64

ix) γ = 7.5% , $r_u \geq 0.7$

a) **Seed et al. (2003)**

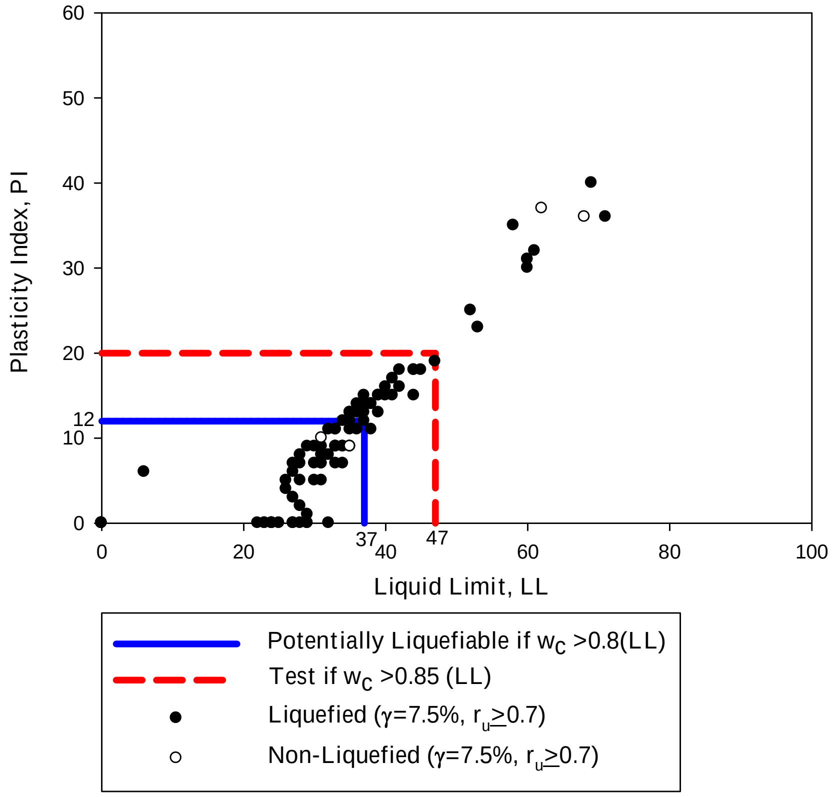

Figure A.22 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 0.7 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.22 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.7$.

γ=7.5 %, $r_u \geq 0.7$			
Observed/ Seed et al.	YES	TEST	NO
YES	50	22	11
NO	4	2	5

b) Bray and Sancio (2006)

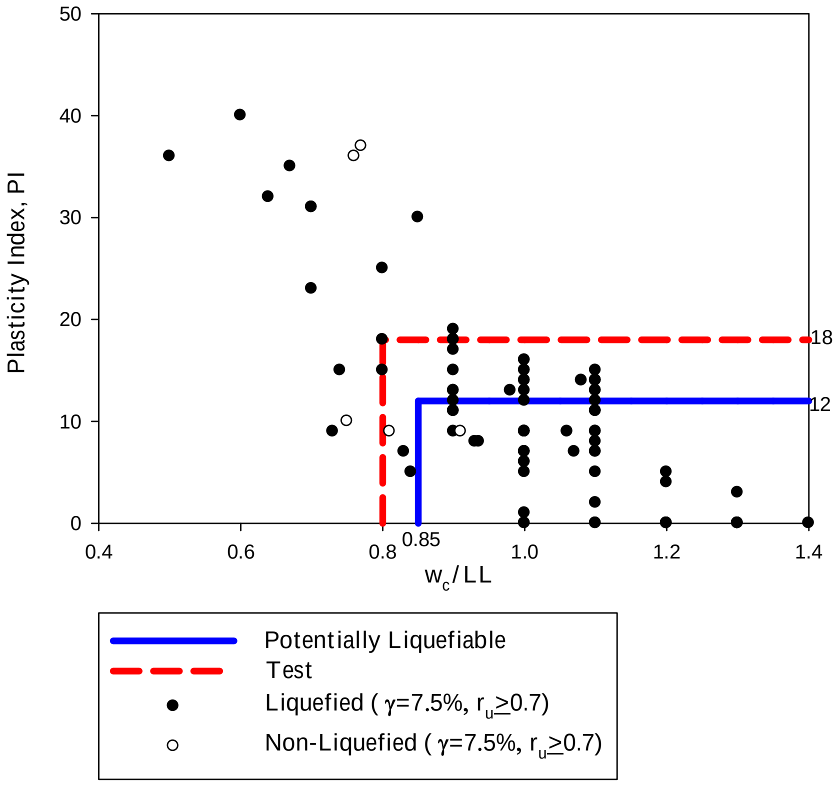

Figure A.23 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq 0.7$ condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.23 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.7$.

γ=7.5 %, $r_u \geq 0.7$			
Observed/ Bray & Sancio	YES	TEST	NO
YES	48	22	13
NO	3	2	6

c) Boulanger and Idriss (2004, 2006)

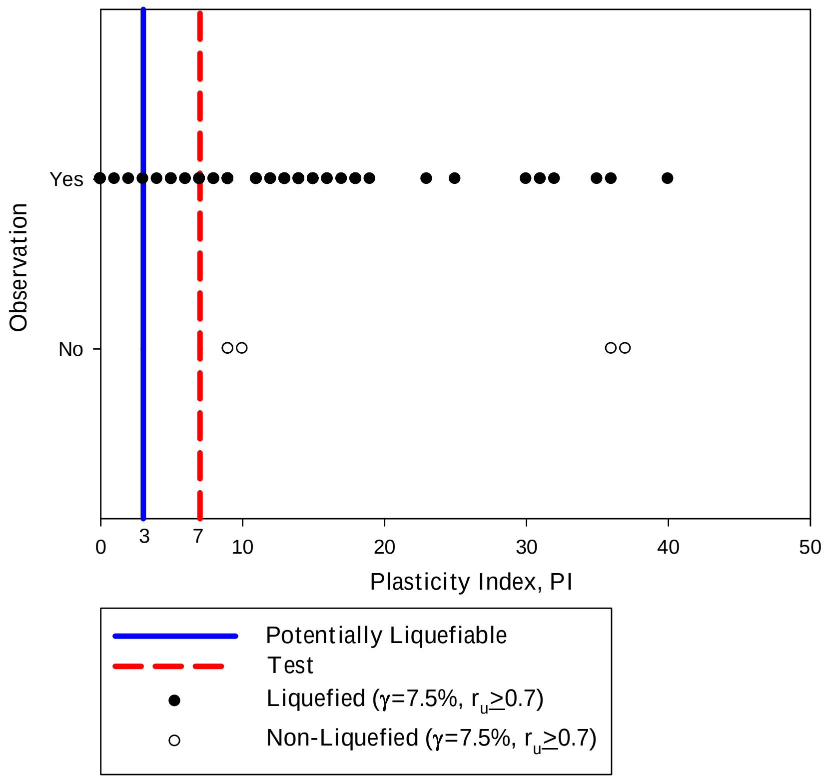

Figure A.24 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq 0.7$ condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.24 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.9$

γ=7.5 %, r_u≥0.7			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	22	13	53
NO	0	1	10

x) γ **= 7.5% , r$_u$ ≥ 0.8**

a) Seed et al. (2003)

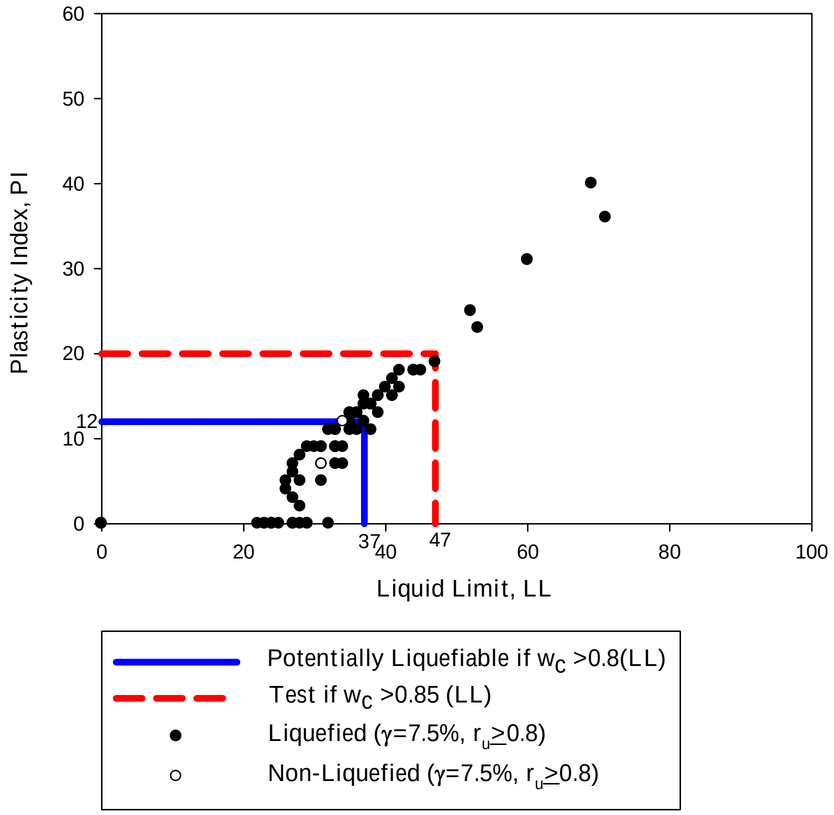

Figure A.25 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 0.8 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.25 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.8$.

γ=7.5 %, r_u≥0.8			
Observed/ Seed et al.	YES	TEST	NO
YES	48	22	11
NO	6	2	5

b) Bray and Sancio (2006)

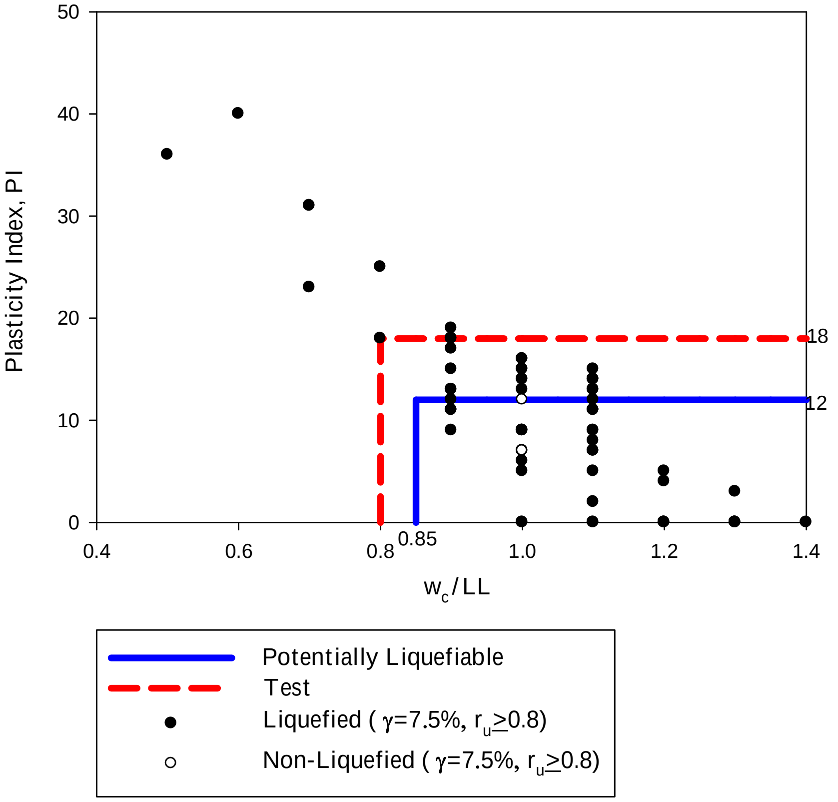

Figure A.26 Evaluation for liquefaction susceptibility condition (γ=7.5%, r_u≥ 0.8 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.26 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.80$

γ=7.5 %, r_u≥0.8			
Observed/ Bray & Sancio	YES	TEST	NO
YES	46	22	13
NO	5	2	6

c) Boulanger and Idriss (2004, 2006)

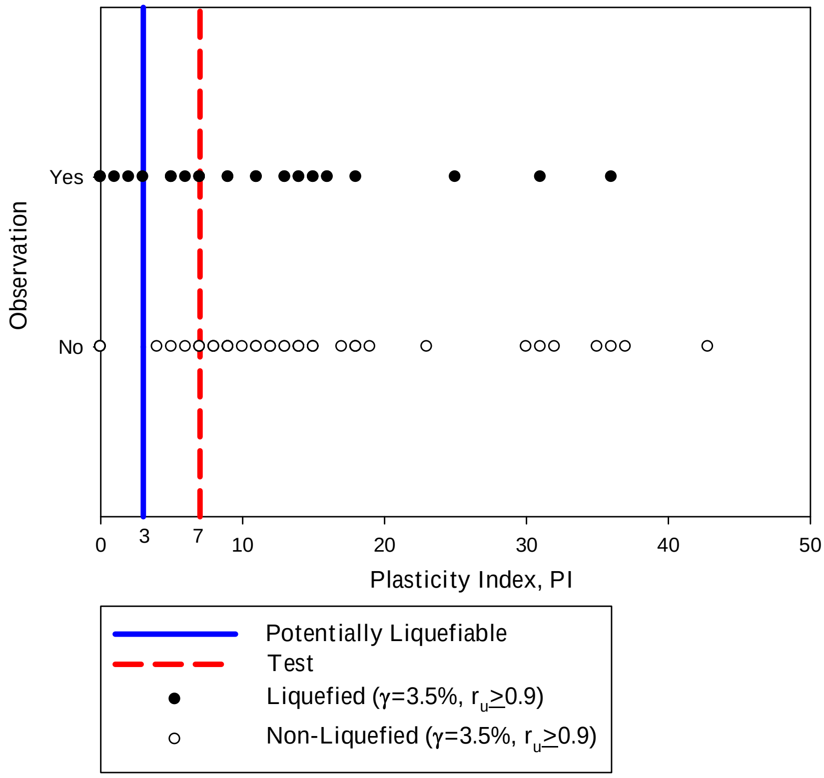

Figure A.27 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq 0.8$ condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.27 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.8$

γ=7.5%, r_u≥0.8			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	22	12	52
NO	0	2	11

xi) γ = 7.5% , $r_u \geq$ 0.9

a) Seed et al. (2003)

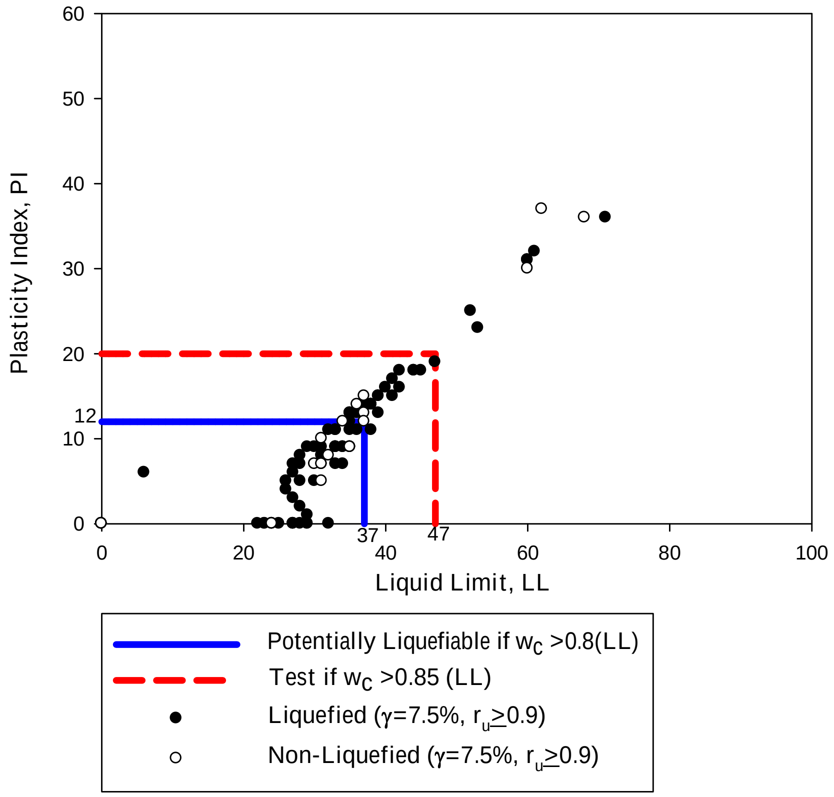

Figure A.28 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 0.9 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.28 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.9$

γ=7.5%, $r_u \geq$0.9			
Observed/ Seed et al.	YES	TEST	NO
YES	43	21	7
NO	11	3	5

b) Bray and Sancio (2006)

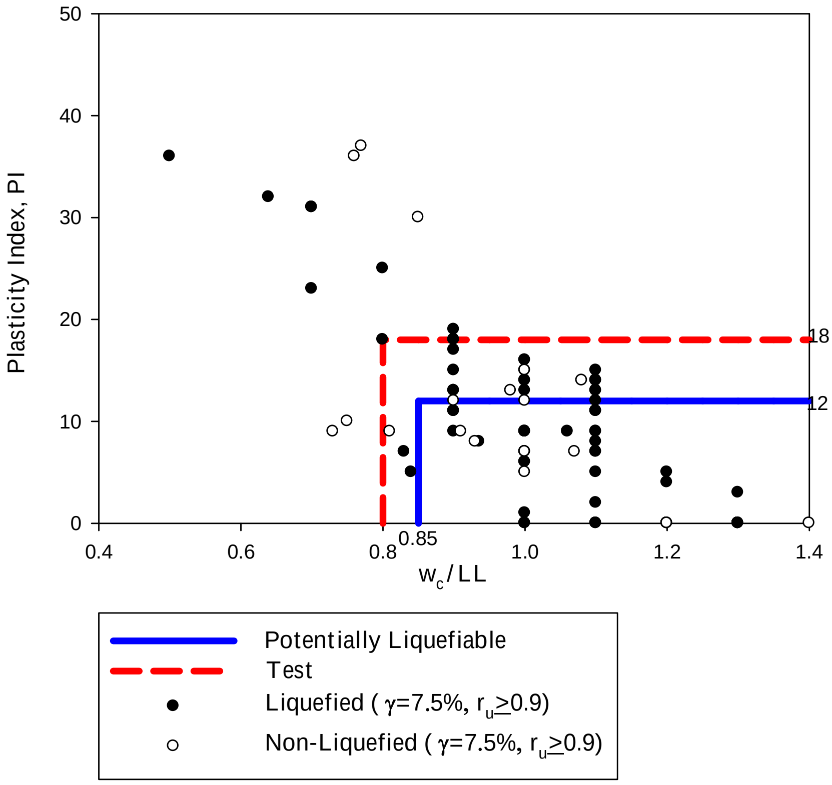

Figure A.29 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 0.9 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.29 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.9$.

γ=7.5 %, $r_u \geq$0.9			
Observed/ Bray & Sancio	YES	TEST	NO
YES	42	20	9
NO	9	3	7

c) Boulanger and Idriss (2004, 2006)

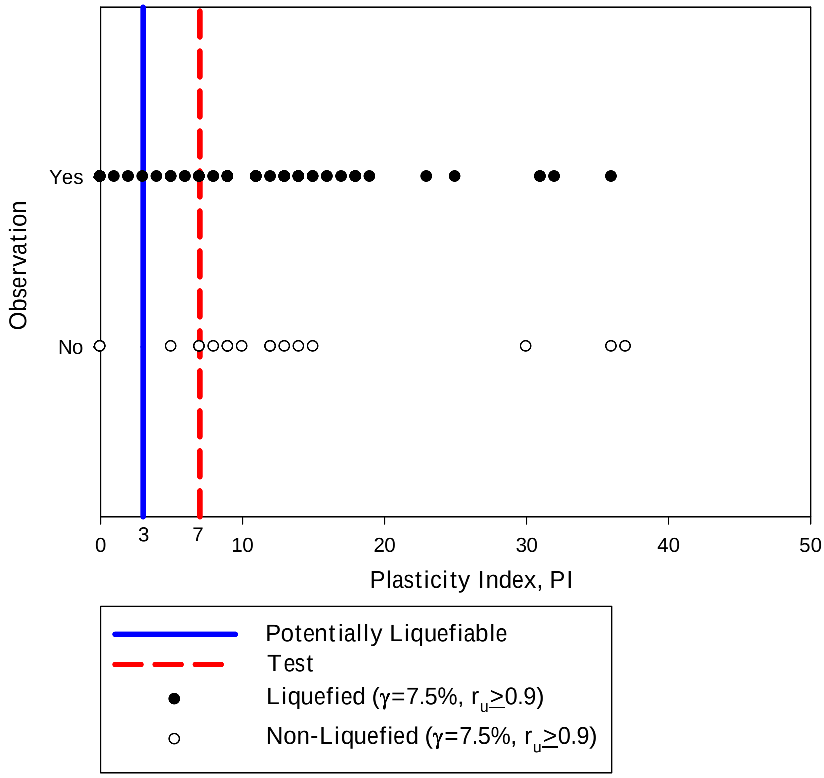

Figure A.30 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 0.9 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.30 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 0.9$

γ=7.5%, $r_u \geq$0.9			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	19	10	46
NO	3	4	13

xii) γ = 7.5% , $r_u \geq$ 1.0

a) Seed et al. (2003)

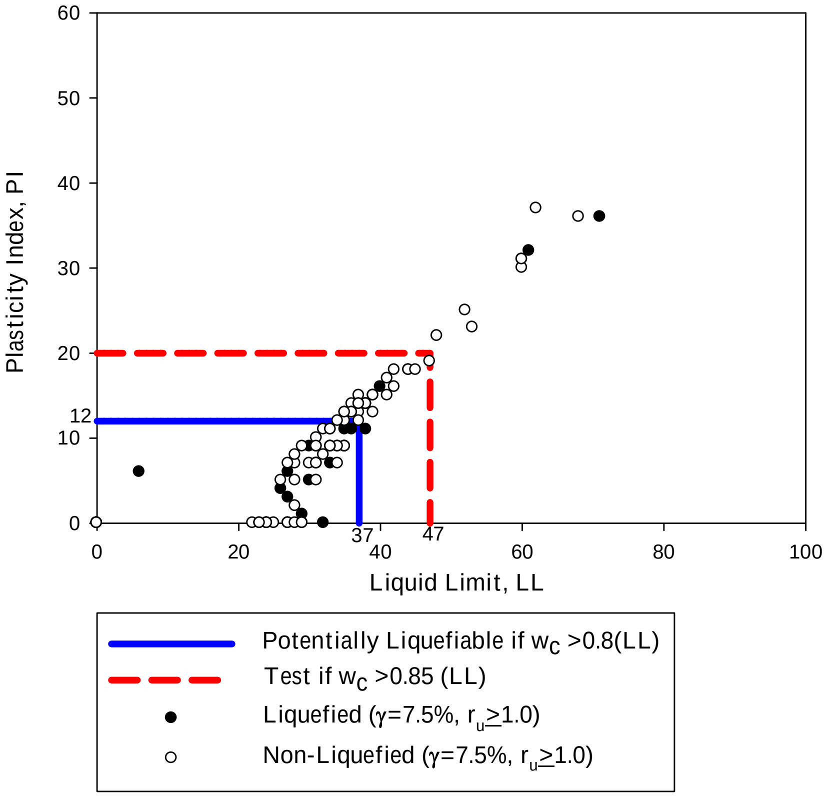

Figure A.31 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 1.0 condition) with comparison by Seed et al. (2003) liquefaction susceptibility criteria

Table A.31 Comparision of liquefaction susceptibility criteria of Seed et al. (2003) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq$ 1.0.

γ=7.5 %, $r_u \geq$1.0			
Observed/ Seed et al.	YES	TEST	NO
YES	14	3	2
NO	39	21	11

b) Bray and Sancio (2006)

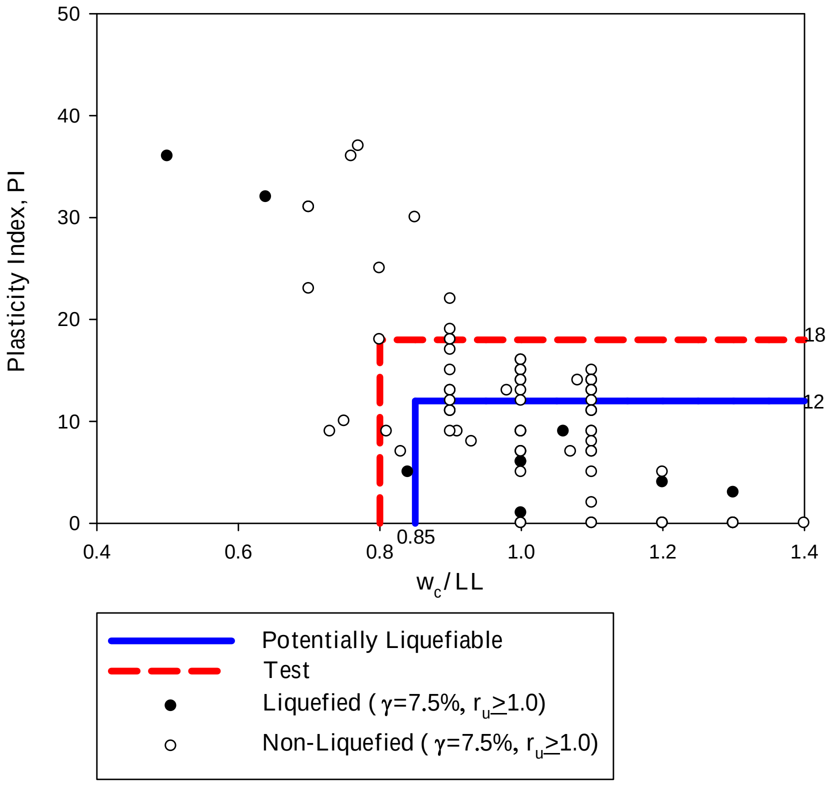

Figure A.32 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 1.0 condition) with comparison by Bray and Sancio (2006) liquefaction susceptibility criteria

Table A.32 Comparision of liquefaction susceptibility criteria of Bray and Sancio (2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq$ 1.0.

γ=7.5 %, r_u≥1.0			
Observed/ Bray & Sancio	YES	TEST	NO
YES	14	2	3
NO	36	21	14

c) Boulanger and Idriss (2004, 2006)

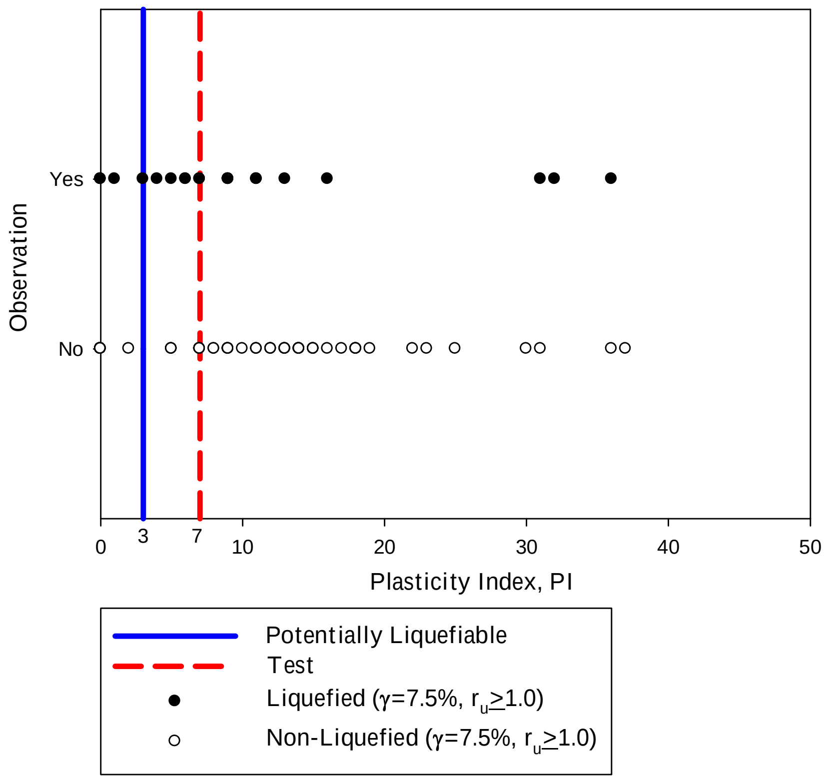

Figure A.33 Evaluation for liquefaction susceptibility condition (γ=7.5%, $r_u \geq$ 1.0 condition) with comparison by Boulanger and Idriss (2004,2006) liquefaction susceptibility criteria

Table A.33 Comparision of liquefaction susceptibility criteria of Boulanger and Idriss (2004,2006) with compiled database for liquefaction susceptibility condition γ= 7.5 %, $r_u \geq 1.0$

γ=7.5 %, $r_u \geq 1.0$			
Observed/ Boulanger&Idriss	YES	TEST	NO
YES	4	5	10
NO	18	9	49

APPENDIX B

CHART SOLUTIONS FOR EXCESS PORE WATER PRESSURE RATIO ON CPT DOMAIN

Appendix B presents the chart solutions obtained for assessment of excess pore pressure ratio corresponding to 3.5 and 5% double amplitude maximum shear strains for w_c/LL ratios of 0.8, 1.00 and 1.20.

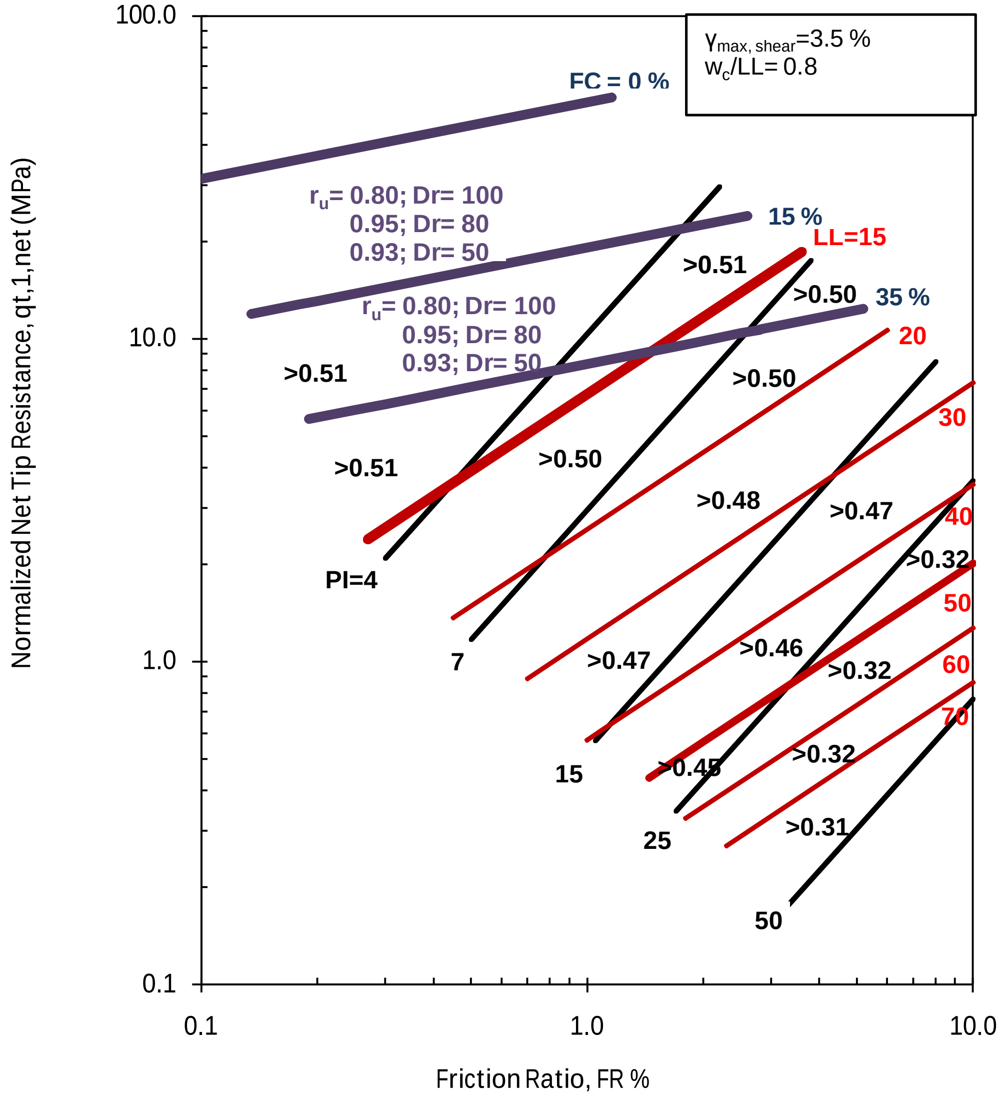

Figure B.1 Chart solution of r_u for γ_{max}=3.5% and w_c/LL = 0.80

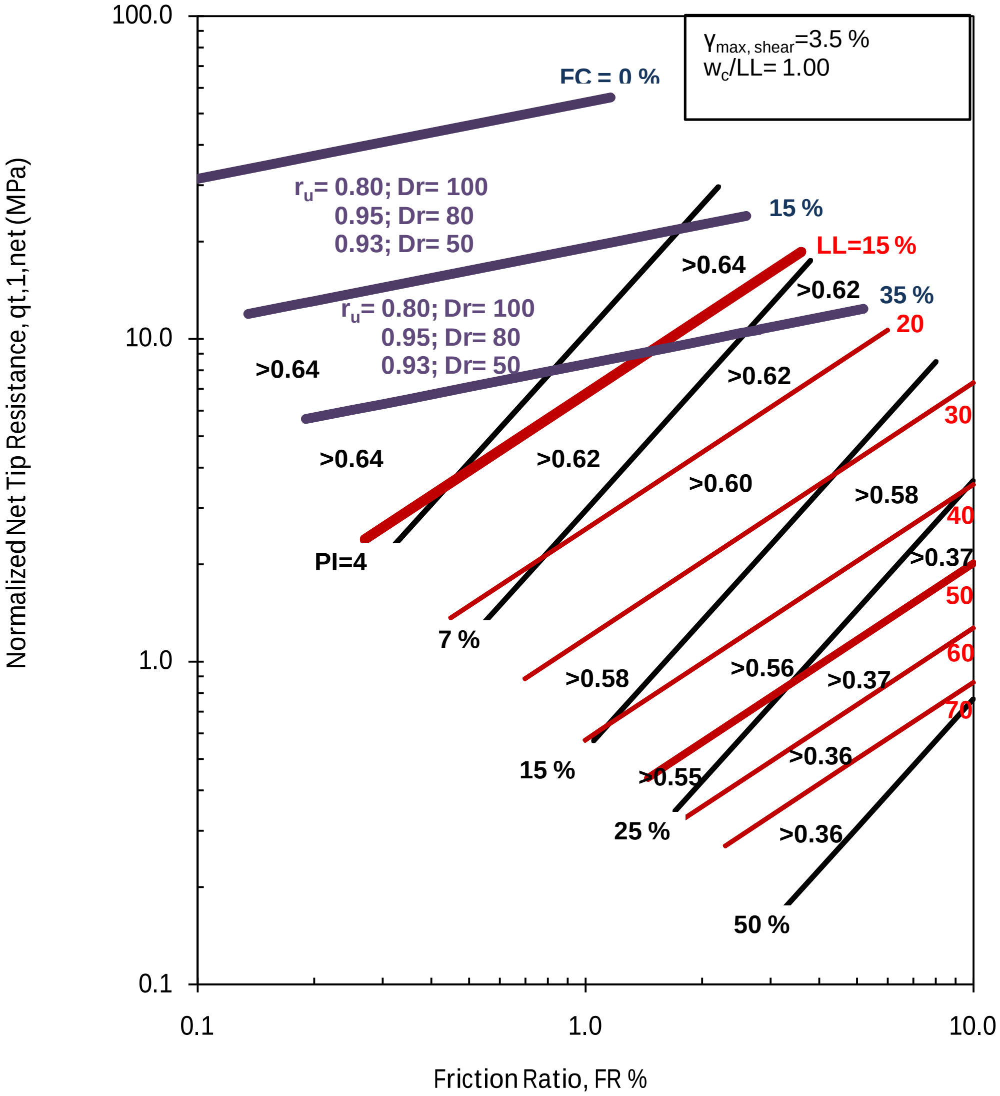

Figure B.2 Chart solution of r_u for γ_{max}=3.5% and w_c/LL = 1.00

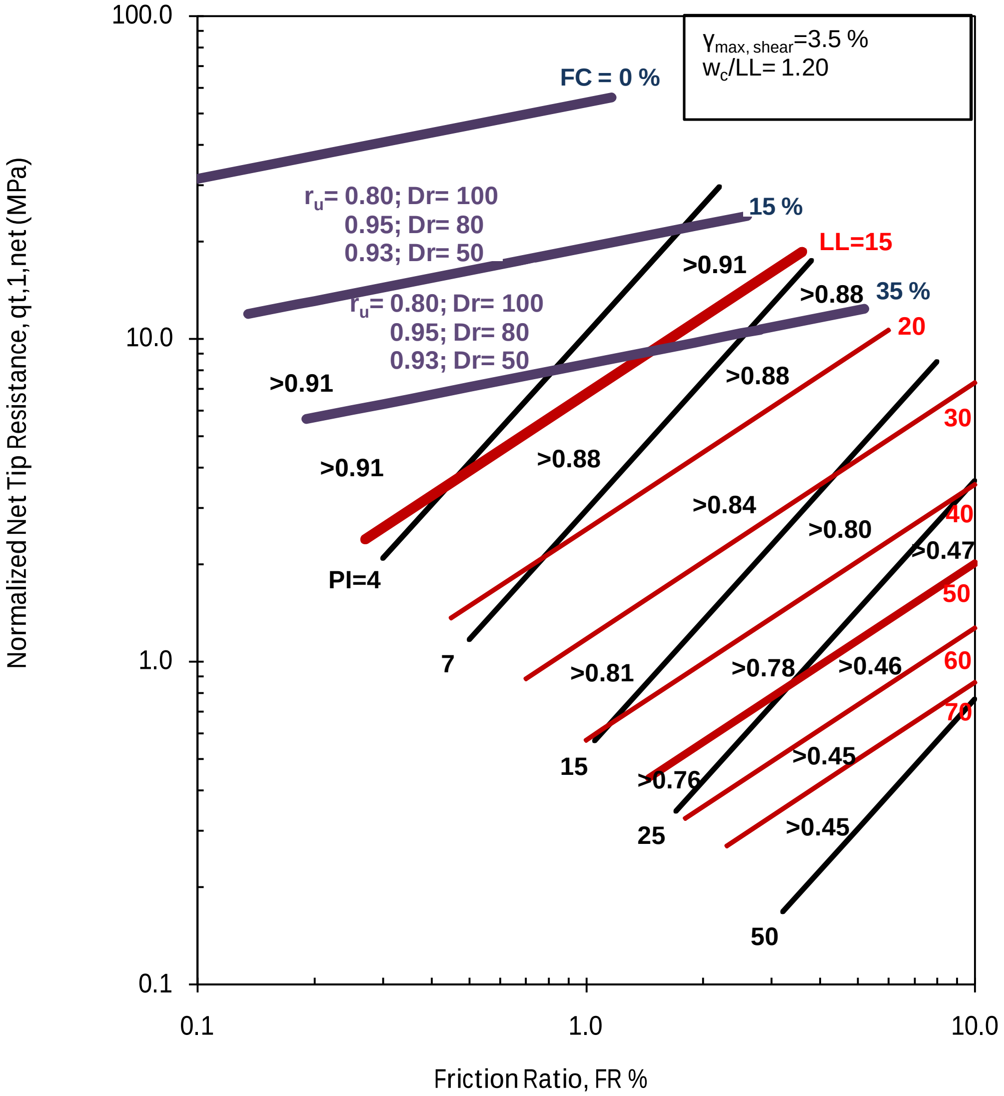

Figure B.3 Chart solution of r_u for γ_{max}=3.5% and w_c/LL = 1.20

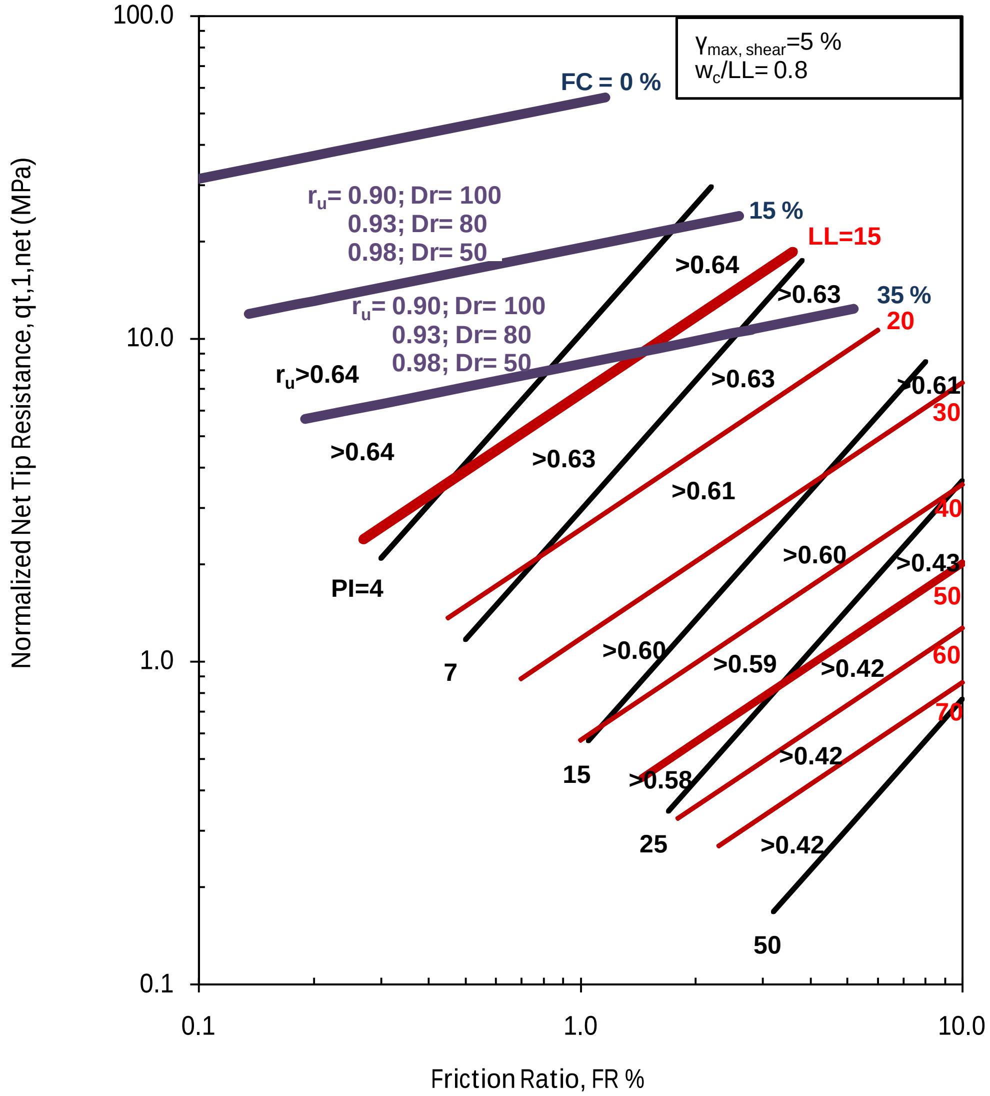

Figure B.4 Chart solution of r_u for γ_{max}=5% and w_c/LL = 0.80.

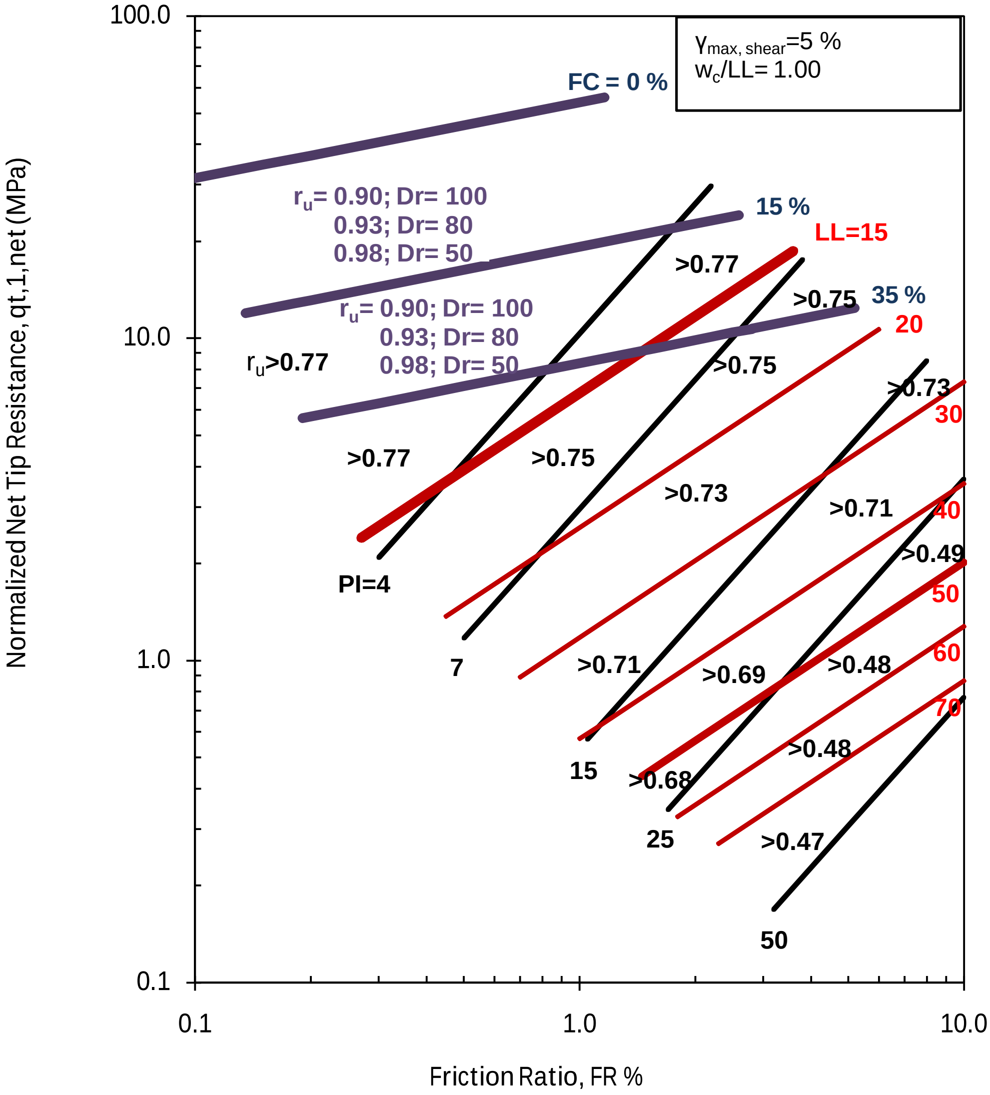

Figure B.5 Chart solution of r_u for γ_{max}=5.0% and w_c/LL = 1.00

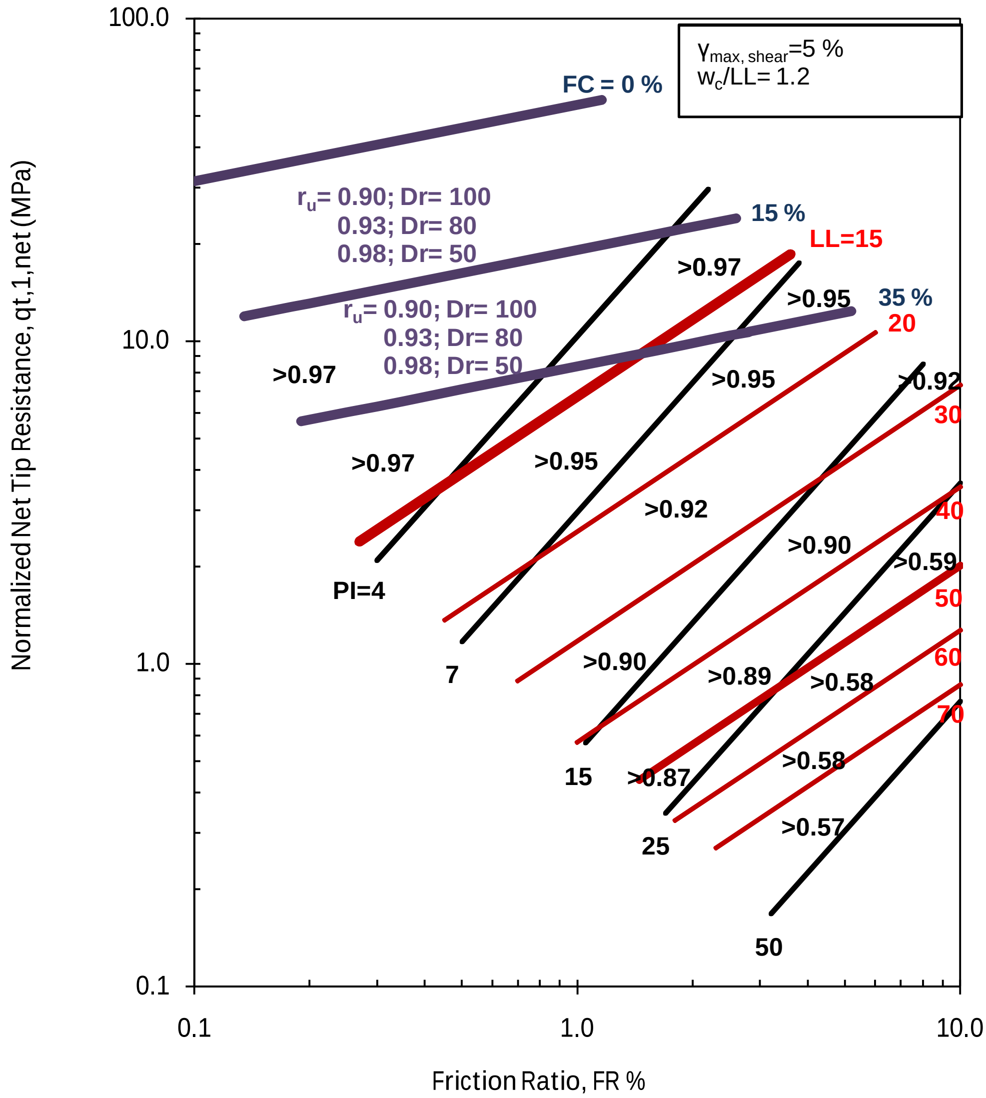

Figure B.6 Chart solution of r_u for γ_{max}=5.0% and w_c/LL = 1.20.