CPT-based evaluation of liquefaction potential for fine-grained soils

M. Pehlivan, H. T. Bilge, K. O. Cetin
Middle East Technical University, Ankara, Turkey

ABSTRACT: 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. In this study, results of cyclic tests performed on undisturbed specimens of ML, CL, MH and CH types 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 water pressure ratio (r_u) vs. shear strain (γ_{max}) response is observed, which is identified to be a function of PI, LL and (w_c/LL). Within the confines of this study, i) probabilistic based boundary curves identifying liquefaction triggering potential in the r_u vs. γ_{max} 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 & Ozan (2009). The proposed framework enabled CPT-based assessment of liquefaction triggering potential of fine-grained low plasticity soils, differentiating clearly both cyclic mobility and liquefaction type soil responses.

1 INTRODUCTION

Seismic liquefaction of soils, defined as significant loss in shear strength and stiffness due to increase in pore pressures, has been one of the major reasons for damage and loss of life during earthquakes. Since the 1964 Niigata and Great Alaska earthquakes, numerous research studies have been performed to better understand this phenomenon. Almost five decades have passed, and meanwhile the number of both case histories and high-quality laboratory test data has increased. Yet, more needs to be done to understand liquefaction potential of fine grained soil' which will be scope of this manuscript.

In their state-of-the-art work, Seed et al. (2003) introduced key components of liquefaction engineering, where identifying liquefaction susceptible soils was referred to as the starting point for assessment. Regarding liquefaction triggering susceptibility aspect, until Haicheng (1975) and Tangshan (1976) earthquakes, it was believed that only "clean sandy soils" with few amount of fines liquefy. Cohesive soils were considered to be resistant to cyclic loading due to cohesional component of shear strength. However, recent earthquakes of 1994 Northridge, 1999 Adapazari and

1999 Chi-Chi once again showed that silty and clayey soils can undergo seismically-induced soil liquefaction. Consistent with the advances in seismic soil liquefaction engineering, susceptibility assessments of fine grained soils revolved from pioneering Chinese Criteria 1979, to the methodologies of Seed & Idriss 1982, Seed et al. 2003, Bray & Sancio 2006). Considering the limitations of these studies, which will be presented later in this paper, an alternative framework with a theoretical background is proposed to assess the liquefaction susceptibility of fine-grained soils. The inspiration behind the proposed framework is due to the observation that cohesionless soils have a unique pore water pressure ratio (r_u) vs. shear strain (γ_{max}) response, and compared to saturated clean sands, a shift in pore water pressure ratio (r_u) vs. shear strain (γ_{max}) response is observed in cohesive soil samples. This shift is identified to be a function of PI, LL and (w_c/LL). Thus, (r_u) vs. shear strain (γ_{max}) domain is decided to be used to differentiate 'sand-like' and 'clay-like' responses. The proposed framework provides 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 & Ozan (2009).

After a brief review of existing methodologies, data compilation and process efforts, and development of proposed framework will be discussed in the following sections of this manuscript.

2 EXISTING LIQUEFACTION SUSCEPTIBILITY CRITERIA

Based on liquefaction-induced ground failure case histories compiled from predominantly fine grained soils sites after 1975 Haicheng and 1976 Tangshan earthquakes, Wang (1979) proposed liquefaction susceptibility assessment rules widely referred to as Chinese Criteria. Chinese Criteria and its improved versions have been widely used (e.g. Seed & Idriss 1982, Andrews & Martin 2000) in practice. However, the ground failure case histories after 1989 Loma-Prieta, 1994 Northridge, and especially 1999 Kocaeli and 1999 Chi-Chi earthquakes revealed that neither Chinese Criteria nor these improved versions can successfully discriminate potentially liquefiable and non-liquefiable fine grained soils. Inspired from this gap, Seed et al. (2003), Bray & Sancio (2006), and Boulanger & Idriss (2006) proposed new liquefaction susceptibility criteria based on field observations and laboratory test results. A summary of these criteria is presented in Table 1.

For the assessment of liquefaction triggering potential, first step is to determine whether the soil is potentially liquefiable or not. For this purpose, "Chinese Criteria" had been widely used for many years. However, contrary to Chinese Criteria, recent advances revealed that i) non-plastic fine grained soils can also liquefy, ii) PI is a major controlling factor in the cyclic response of fine grained soils. These criteria are then modified by Andrews & Martin (2000) for USCS-based silt and clay definitions. Bray et al. (2001) has concluded that the use of Chinese Criteria percent "clay-size" definition may be misleading and rather than the % of clay size material, their activities are judged to be more important. Seed et al. (2003) recommended a new criterion inspired from case histories and cyclic testing of "undisturbed" fine-grained soils compiled after 1999 Kocaeli and Chi-Chi earthquakes. These criteria classify saturated soils with a PI < 12 and LL < 37 as potentially liquefiable, provided that the w_c is greater than 80% of the LL (0.8·LL). Recently, Bray & Sancio (2006), based on mostly cyclic triaxial and some simple shear test results performed on Adapazari undisturbed fine grained soils developed their liquefaction susceptibility criteria, summarized in Table 1. Valid for both Seed et al. and Bray & Sancio methodologies, laboratory test-based liquefaction triggering definition was not clearly presented. Bray& Sancio (2006) adopted 4 % axial strain as liquefaction triggering criterion. However, tests were performed under CSR levels of 0.3, 0.4 and 0.5 and loading cycles were continued if and until this strain level was reached. Thus, their conclusions are judged to be constrained by CSR and durational levels adopted for their testing program. The most recent attempt for determining potentially liquefiable soils was by Boulanger & Idriss (2006). Based on cyclic laboratory test results and an extensive engineering judgment, they have recommended new criteria summarized in Table1. As part of this new methodology, deformation behavior of fine-grained soils are grouped as "Sand-Like" and "Clay-Like", where soils within the sand-like behavior region are judged to be susceptible to liquefaction and have substantially lower values of cyclic resistance ratio, CRR, than those within the clay-like behavior region. The main drawback of the methodology is the fact that the y-axis of Figure 1 is not to scale, thus a direct comparison between CRR of "clay-like" and "sand-like" responses is not possible. Also, very little, to an extent of none, is known about if and how identical or comparable "sand-like" and "clay-like" samples were prepared.

Table 1. A summary of available liquefaction susceptibility criteria for fine grained soils

Assessment Method	Potentially Liquefiable	Test for a Decision	Non-liquefiable
Chinese Criteria Wang (1979)	- FC ≤15% - LL ≤35% - $w_c \geq (0.9*LL)\%$		Otherwise
Andrews and Martin (2000)	- Clay content, CC<10% - LL < 32%	- CC<10% & LL≥32% -CC ≥ 10% & LL < 32%	- CC≥10% & LL≥32%
Seed et al. (2003)	- PI < 12% - LL < 37% - $w_c/LL > 0.8$	- 12 < PI < 20 - 37 < LL < 47 - $w_c/LL > 0.85$	Otherwise
Bray and Sancio (2006)	- PI<12% $w_c/LL > 0.85$	- 12 < PI < 18 - $w_c/LL > 0.80$	Otherwise
Boulanger and Idriss (2006)	- PI < 3%	- 3≤ PI ≤ 6	- PI ≥ 7

Although these studies are judged to be improvement over earlier studies, they suffer from one or more of the following issues: (i) there is no unique definition of liquefaction and hence, each criterion is developed based on different understandings regarding what liquefaction response is, (ii) the amplitude of cyclic loading is not specified in γ_{max}- or r_u-based exceedence of threshold definition; as a consequence there exist ambiguity under which cyclic stress conditions these criteria are applicable, and iii) most of these studies fail to differentiate cyclic liquefaction and mobility type soil responses.

3 DATABASE COMPILATION EFFORTS

For the purpose of discriminate between the responses of cohesionless and cohesive soils, cyclic test results of both types of soils were studied. The databases studied and compiled consist of tests performed on: i) laboratory reconstituted clean sands (Wu et al. 2003 and Bilge 2005) and ii) "undisturbed" fine-grained soils (Pekcan 2001, Sancio 2003, and Bilge, in prep.). The compiled database is composed of 158 cyclic test results including r_u vs. γ_{max} histories, Atterberg limits along with moisture content of specimens, consolidation and applied cyclic shear stress conditions. Table 2 briefly summarizes the data sources used in this study, and compiled data is presented on r_u vs. γ_{max} domain in Figure 2. More detailed information regarding these data sources and details of data processing can be found in the original references.

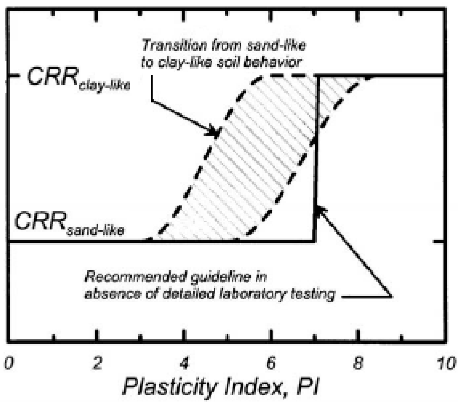

Figure 1. Criteria for discriminate between sand- and clay-like soil behavior (Boulanger & Idriss 2006)

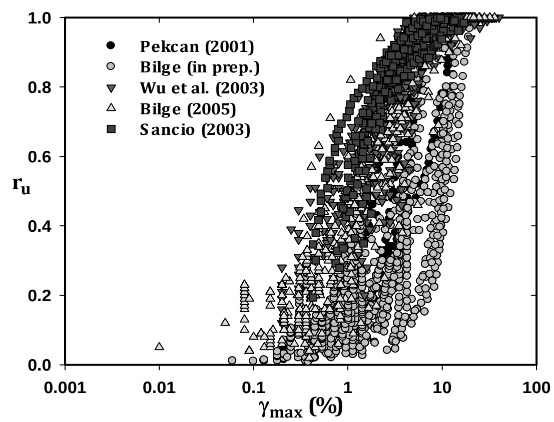

Figure 2. Compiled database in r_u vs. γ_{max} domain

Table 2. Summary of the data sources used in this study

Type	Data Source	# of data	Test Type	Tested Material
Coarse-grained	Wu et al. (2003)	50	Simple Shear	Monterey No.0/30 Sand
	Bilge (2005)	36	Cyclic Triaxial	Kizilirmak Sand
Fine-grained	Pekcan (2001)	7	Cyclic Triaxial	"undisturbed" Adapazari
	Sancio (2003)	15	Cyclic Triaxial	"undisturbed" Adapazari
	Bilge (in prep.)	50	Cyclic Triaxial	"undisturbed" Adapazari, Ordu

4 DEVELOPMENT OF PORE WATER PRESSURE GENERATİON MODEL

As discussed earlier, the shift in (r_u) vs. shear strain (γ_{max}) response of cohesive soils relative to cohesionless ones is identified to be a function of PI, LL and (w_c/LL). Thus, (r_u) vs. shear strain (γ_{max}) domain is decided to be used to differentiate 'sand-like' and 'clay-like' responses. The proposed framework provides liquefaction susceptibility boundary curves as a function of soil index parameters (PI, LL, w_c/LL). Selection of a limit state model capturing the important features of the observed behavior is the first step for development of a probabilistic model. The limit state function has the general form of $g = g$ **(x, Θ)** where x is a set of descriptive parameters and Θ is set of unknown model coefficients. Inspired by previous studies and observed trends from tests, it is concluded that for cohesive soils, key parameters affecting r_u response are γ_{max}, PI, LL and w_c/LL. Inspired mainly by the recent study of Cetin & Bilge (in prep.), given for cohesionless soils, various functional forms have been tested (Pehlivan 2009) and consistent with maximum likelihood methodology the following functional form is selected as the limit state model as it results in greater likelihood value and smaller model error, which are the indications of a superior model.

$$\hat{g}(r_u, \gamma_{\max}) = \ln(r_u) - \ln[1 - \exp(\alpha)] \pm \varepsilon_{\ln(r_u)} \tag{1}$$

$$\alpha = \frac{\gamma_{max}}{\theta_1 - \theta_2 \cdot [ln(\theta_3 \cdot PI + 1)]^{\theta_4} - \theta_5 \cdot [ln(\theta_6 \cdot LL + 1)]^{\theta_7} + \theta_8 \cdot [ln(\theta_9 \cdot w_c / LL + 1)]^{\theta_{10}}} \tag{2}$$

where ε is a random model correction term to account for possibilities of i) missing descriptive variables, and ii) imperfection of the adopted mathematical expression. 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. The standard deviation of ε (σ_ε) is unknown and must be estimated. Both the unknown coefficients and σ_ε were determined via maximum likelihood analysis and their corresponding values are presented in Table 3. Figure 3 presents the boundary curves developed for the mean values of compiled database, PI=22, LL=45, w_c/LL=0.82 along with $\pm$ one standard deviation (σ_ε) curves and compiled data. This figure revealed that proposed model and the suggested error bands captures the observed soil response successfully.

Rather than considering only soil index parameters, this correlation also accounts for the accumulated shear strain which is related to amplitude and duration of cyclic loading. Moreover, by this way the mechanisms governing cyclic response of soils can be taken into account. Cyclic stress-strain relationships of soils are usually defined through degradation of shear modulus as a function of cyclic shear strain. As pointed out previously (e.g. Seed & Idriss 1970) remolding (i.e. strain accumulation) and loss in effective stress (i.e. pore water pressure generation) play an integral role in stiffness degradation. The other factors affecting this degradation response have been studied by various other researchers (e.g. Vucetic & Dobry 1991). Founding on this theoretical background, a robust relationship between r_u and γ_{max} is developed and this relationship will be the basis of our framework which will be consequently valid for any liquefaction definition, take into account the significance of stress amplitudes and also be able to differentiate cyclic liquefaction and mobility type soil responses.

5 NEW LIQUEFACTION CRITERIA

Development of new liquefaction susceptibility criteria requires a definition for triggering of liquefaction. Considering both previous efforts and trends observed from available experimental data, for fine-grained soils liquefaction is defined as follows: For γ_{max} = 7.5%, if induced r_u is between 0.85 and 1.0 then soil is classified as potentially liquefiable (sand-like). If r_u is less than 0.7 at γ_{max} = 7.5%, then it is classified as potentially nonliquefiable (clay-like) and in between these limits further testing may be required. Validity of these criteria is assessed by using available test data and it was observed that the error in identification of cohesive soils susceptible to liquefaction was not greater than 10%. Figure 4 presents the proposed liquefaction susceptibility criteria for w_c/LL=1.0 condition on plasticity chart.

Table 3. Model coefficients

	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	0.485

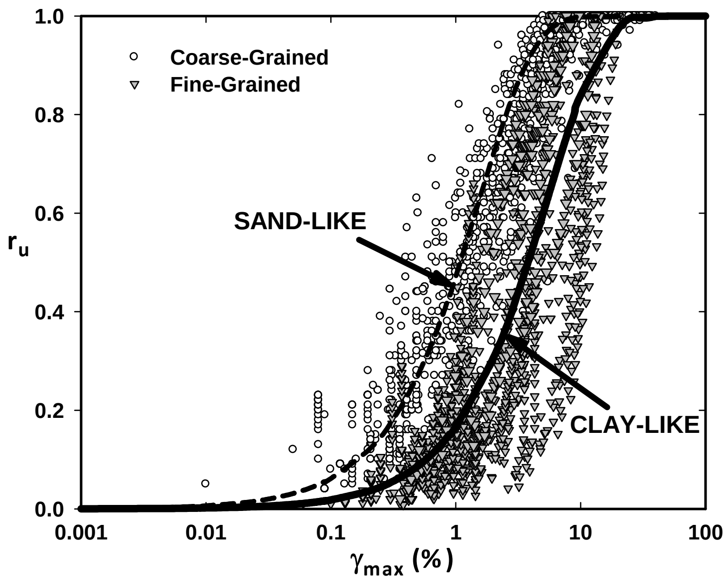

Figure 3. Proposed r_u generation model along with the compiled database

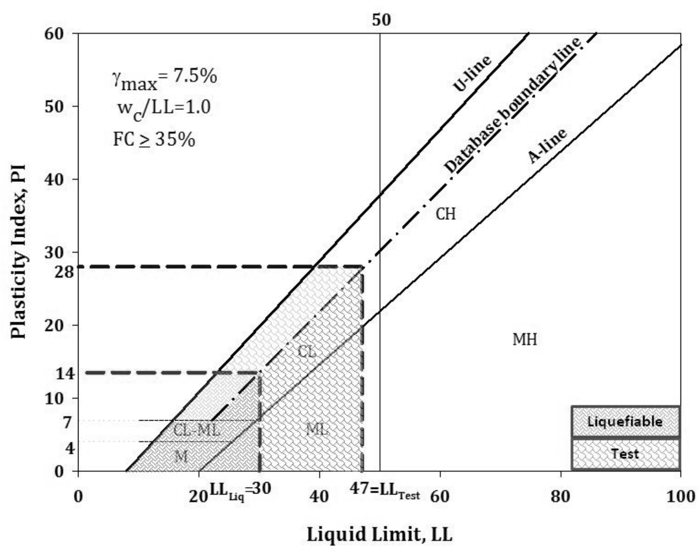

Figure 4. Liquefaction susceptibility criteria for w_c/LL=1.0 condition

In Figure 4, LL boundaries were estimated by solving the Equation 1 for γ_{max} =7.5% and w_c/LL=1.0. On the other hand, PI boundaries were defined according to the "database boundary line". This line is determined by considering the upper limits of LL and PI of the compiled database and it is defined as follows:

$$PI = 0.83 \cdot LL - 11.46 \quad (3)$$

6 ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY ON CPT DOMAIN

Although by using Equation 1, it is possible to assess liquefaction susceptibility for different combinations of γ_{max}, PI, LL, and w_c/LL values. Due to page limitations only the solution for γ_{max} =7.5% and w_c/LL =1.0 will be presented. Proposed Equation could be mapped on the CPT tip resistance and friction ratio domain by using Cetin & Ozan (2009) relationships. Figure 5 presents the minimum r_u levels induced by γ_{max} =7.5%. Similarly, the proposed liquefaction susceptibility margins are also mapped on CPT domain as shown in Figure 6 for γ_{max} =7.5% and w_c/LL = 1.0. For comparison purposes on the same figure, liquefaction susceptibility boundary of Robertson and Wride (1998) is also shown. Close agreement between these two fundamentally different methods is judged to be mutually supportive. However, it should be noted that such close agreement can not be achieved if w_c/LL is adopted as lower than 1.0. It is observed that as the selected γ_{max} decreases or w_c/LL increases boundary between of "liquefiable" and "test" moves toward right.

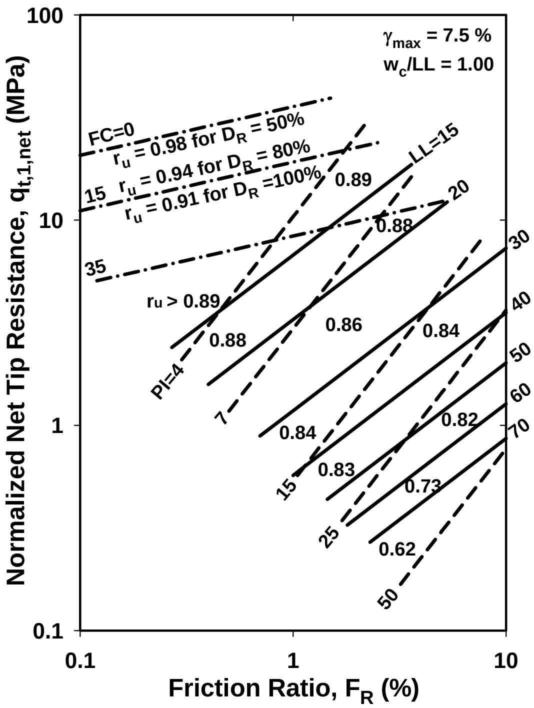

Figure 5. Minimum levels of ru for γ_{max} =7.5% and w_c/LL=1.0

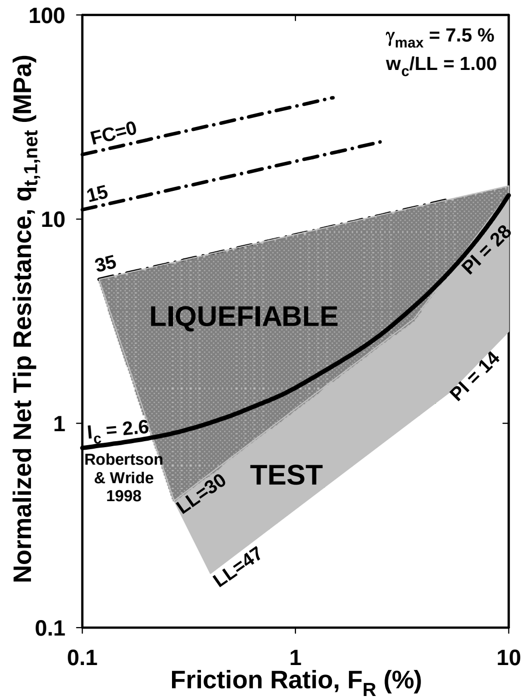

Figure 6. Proposed liquefaction susceptibility margins for γ_{max} =7.5% and w_c/LL=1.0

7 SUMMARY AND CONCLUSIONS

Within the confines of this manuscript, a new framework is proposed for the assessment of the liquefaction susceptibility of fine-grained soils considering the major drawbacks of the existing criteria. The proposed framework involves development of probabilistic-based boundary curves to define liquefaction triggering potential in r_u vs. γ_{max} domain as a function of PI, LL and w_c/LL. Then these boundaries were mapped on to the CPT domain ($q_{t,1,net}$ vs. F_R) consistent with the recent work of Cetin

& Ozan (2009).The proposed methodology is considered to provide a robust and defensible basis for assessment of liquefaction susceptibility of fine-grained soils, as it i) uses the relation between r_u vs. γ_{max} rather than using a specific liquefaction definition like its predecessors, ii) considers the level of cyclic loading through γ_{max}, iii) differentiates cyclic liquefaction and mobility type soil responses, and iv) proposes a practical approach to assess liquefaction susceptibility based on CPT data.

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