ASSESSMENT OF SOIL – STRUCTURE – EARTHQUAKE INTERACTION INDUCED SOIL LIQUEFACTION TRIGGERING

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

BY

BERNA UNUTMAZ

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY IN CIVIL ENGINEERING

DECEMBER 2008

Approval of the thesis:

ASSESSMENT OF SOIL – STRUCTURE – EARTHQUAKE INTERACTION INDUCED SOIL LIQUEFACTION TRIGGERING

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ABSTRACT

ASSESSMENT OF SOIL – STRUCTURE – EARTHQUAKE INTERACTION INDUCED SOIL LIQUEFACTION TRIGGERING

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December 2008, 252 pages

Although there exist some consensus regarding seismic soil liquefaction assessment of free field soil sites, estimating the liquefaction triggering potential beneath building foundations still stays as a controversial and difficult issue. Assessing liquefaction triggering potential under building foundations requires the estimation of cyclic and static stress state of the soil medium. For the purpose of assessing the effects of the presence of a structure three-dimensional, finite difference-based total stress analyses were performed for generic soil, structure and earthquake combinations. A simplified procedure was proposed which would produce unbiased estimates of the representative and maximum soil-structure-earthquake-induced cyclic stress ratio (CSR_{SSEI}) values, eliminating the need to perform 3-D dynamic response assessment of soil and structure systems for conventional projects. Consistent with the available literature, the descriptive (input) parameters of the proposed model were selected as soil-to-structure stiffness ratio (σ), spectral acceleration ratio (S_A/PGA) and aspect ratio of the building. The model coefficients were estimated through maximum likelihood methodology which was used to produce an unbiased match with the predictions of 3-D analyses and proposed simplified procedure. Although a satisfactory fit was achieved among the CSR estimations by numerical seismic response analysis results and the proposed simplified procedure, validation of the proposed simplified procedure further with available laboratory shaking table and centrifuge tests and well-documented field case histories was preferred. The proposed simplified procedure was shown to capture almost all of the behavioral trends and most of the amplitudes.

As the concluding remark, contrary to general conclusions of Rollins and Seed (1990), and partially consistent with the observations of Finn and Yodengrakumar (1987), Liu and Dobry (1997) and Mylonakis and Gazetas, (2000), it is proven that soil-structure interaction does not always beneficially affect the liquefaction triggering potential of foundation soils and the proposed simplified model conveniently captures when it is critical.

Keywords: Soil-structure-earthquake interaction, soil liquefaction, cyclic stress ratio, maximum likelihood, nonlinear regression.

ZEMİN – YAPI – DEPREM ETKİLEŞİMİ TARAFINDAN TETİKLENEN ZEMİN SIVILAŞMASININ BELİRLENMESİ

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Tez Yöneticisi: Doç. Dr. K. Önder Çetin

Aralık 2008, 252 sayfa

Üzerinde yapı bulunmayan, -serbest saha koşulları altında olarak da tanımlananeğimsiz zemin profillerinde deprem yükleri altında sıvılaşma tetiklenme potansiyelinin belirlenmesine yönelik genel bir görüş birliği bulunmasına rağmen, üst yapı etkisi altındaki temel zeminlerinin sıvılaşma tetiklenme potansiyeli ile igili görüş ayrılıkları ve çeşitli zorluklar bulunmaktadır. Bina altlarındaki bu temel zeminlerinin sıvılaşma potansiyellerinin değerlendirilmesi, statik ve deprem durumlarındaki gerilme durumunun belirlenmesi ile mümkündür. Binanın varlığının etkisinin belirlenebilmesi amacı ile, senaryo zemin, üstyapı ve deprem kayıtları kombine edilerek sonlu farklar yöntemine dayalı, 3-Boyutlu toplam gerilme analizleri gerçekleştirilmiştir. Sıvılaşma tetiklenme potansiyelinde üst yapının etkisini belirlemek amacı ile ortalama ve maksimum cevrimsel gerilme oranlarını tanımlayan (CSR_{SSEI,ortalama} ve CSR_{SSEI,maksimum}) basitleştirilmiş bir yöntem önerilerek. 3-Boyutlu analizlere ihtiyaç duyulmadan bina temellerinde sıvılaşma tetiklenme potansiyellerinin belirlenebilmesi amaçlanmıştır. CSR_{SSEI,ortalama} ve CSR_{SSEI,maksimum} değerlerinin belirlenmesinde gelişmiş olasılıksal yöntemler kullanılmış ve bu etkilesimi parametreleri σ , değerlerin vapı-zemin S_A/PGA ve yapının yükseklik/genişlik oranlarına bağlı olarak hesaplanması amaçlanmıştır. Modelde yer alan katsayılar, 3 boyutlu analiz sonuçları ve önerilen yöntem ile bulunan değerler arasında tarafsız bir ilişki kurabilmeyi sağlayan maksimum olabilirlik yöntemi kullanılarak belirlenmiştir. Sayısal analiz ve basitleştirilmiş yöntem sonuçlarının uyum içerisinde oldukları görülmesine rağmen, önerilen yöntem 1999 Türkiye depremleri sonrasında derlenen vaka örnekleri ve literatürdeki santrifüj ve sarsma tablası deneyleri ile kalibre edilerek gecerliliği kanıtlanmıştır. Bu değerlendirmeler sonucunca, önerilen basitleştirilmiş yöntemin tüm davranışsal eğilimler ile uyum içerisinde olduğu, davranışı temsil eden büyüklüklerin çoğunu doğru belirlediği görülmüştür.

Sonuç olarak özetlemek gerekirse, Rollins ve Seed (1990) tarafından söylenenin aksine ve kısmen Finn ve Yodengrakumar (1987), Liu ve Dobry (1997) ve Mylonakis ve Gazetas (2000) tarafından da belirtildiği üzere üst yapının sıvılaşma tetiklenme potansiyeli açısından her zaman olumlu yönde etkisi olmadığı belirlenmiş ve önerilen basitleştirilmiş yöntem bu etkileşimin ne zaman kritik olacağını başarıyla tahmin etmiştir.

Anahtar Kelimeler: Yapı – zemin – deprem etkileşimi, zemin sıvılaşması, tekrarlı kayma gerilme oranı, maksimum olabilirlik, doğrusal olmayan regresyon.

To My Parents

ACKNOWLEDGMENTS

I want to gratefully thank Dr. Kemal Önder Çetin, without whom I would not be able to complete my thesis. I wish to express my appreciation for his continuous and unconditional guidance and support at each step of this study. His unlimited assistance, patience and tolerance made my research come into this stage and should never be forgotten. My appreciation is not only his being a mentor in academic and professional life, but also making me feel as comfortable as a friend or a brother in the last six years.

Thanks are also due to the members of my dissertation committee: Dr. M. Yener Özkan and Dr. Levent Tutluoğlu, for their constructive suggestions and subjective comments throughout this research period.

I owe further thanks to Dr. Orhan Erol for his positive approach and considerable support during my graduate studies in Middle East Technical University.

I also want to thank my officemate H. Tolga Bilge for his enormous support, friendship and helps during the last six years. I cannot even imagine how difficult it can be to share a room with a lady who is trying to get her PhD for such a long time. However Tolga was always kind, positive and helpful during this long period. I am sure that it would be even more difficult to complete this dissertation without his considerable aids and support during both the research and writing steps.

I want to acknowledge the support of my dear friend Sevgi Türkkan who made me feel strong even at the most disappointing times. She has always been a generous friend whose door is always open and you know someone is always there. Special thanks are also due to Gökhan Çakan from him I have learned a lot in academic, professional and also in social life. I also want to thank Demirhan Şenel and Ozan Cem Çelik for their friendship during my life. They showed me that kilometers do not mean anything and they both made me feel that they are always with me. Of course the power of girl friends can not be forgotten here: Özlem Kasap, Çağla Meral, Ayşegül Askan and Nazan Yılmaz have always been with me during the years in METU and I am sure that they always be in my life.

I am grateful to my friends Anıl Yunatcı, Serkan Üçer, Abdullah Sandıkkaya and all the others in Geotechnical Engineering group of Middle East Technical University and Nadide Seyhun, Özlem Karaırmak, Pınar Onay and all the others in ÖYP group of Kocaeli University for all the fun and continuous support during this long time period.

Funding for these studies was provided by Scientific Research Development Program, and this support is gratefully acknowledged.

Last, but not least, I would also like to acknowledge the immeasurable support and love from my family who never left me alone. I am sincerely grateful to my mother Asuman Unutmaz and to my father İsmail Unutmaz for always being with me and for their efforts that encouraged me to realize my goals. I also want to thank my sister Nevra Unutmaz Yaldız, brothers Uran Unutmaz and Kasım Yaldız for being with me. Nevra and Uran deserve more thanks for their endurance and patience for not only this limited time period but throughout of my life...

Thanks to all.

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LIST OF ABBREVIATIONS

a _{max}	: Maximum ground acceleration
a(t)	: Ground acceleration at time "t"
В	: Width of structure
CRR	: Cyclic resistance ratio
CRR ₁	: CRR at the reference state (SPT correlation, τ_s = 0 and $\sigma_v{'}$ = 1 atm)
CSR	: Cyclic stress ratio
CSR _{FF}	: Free field cyclic stress ratio
CSR _{SSEI}	: Soil-Structure-Earthquake-Interaction induced cyclic stress ratio
CSR _{SSEI,rep}	:Representative CSR _{SSEI}
CSR _{SSEI,max}	: Maximum CSR _{SSEI}
СРТ	: Cone penetration test
d, D	: Depth from ground surface
D _R	: Relative density
FC	: Fines content
FS_L	: Factor of safety against liquefaction
G	: Shear modulus
g	: Gravitational acceleration

h	: Depth from the ground surface
$\frac{h}{B}$: Aspect ratio of the structure
heffective	: Effective height of structure
K _α	: Correction factor for the level of static horizontal shear stress
K _{OCR}	: Correction factor for over-consolidation effects
K_{σ}	: Correction factor for vertical effective confining stress
L	: Length of the foundation
LL	: Liquid limit
$M_{\rm w}$: Moment magnitude of earthquake
MSF	: Magnitude scaling factor
m_{σ}	: Vertical stress dissipation factor
m_{τ}	: Shear stress dissipation factor
Ν	: Number of storeys of the building
N _{1,60}	: Standard penetration test blow-counts
N _{cyc}	: Number of cycles
N _{liq}	: Number of cycles required to trigger liquefaction
OCR	: Over-consolidation ratio
Pa	: Atmospheric pressure
PGA	: Peak ground acceleration

PI	: Plasticity index
q _{c1}	: Cone penetration test tip resistance
r _d	: Stress reduction factor (mass participation factor)
r _u	: Excess pore pressure ratio
S_A	: Spectral acceleration
S _A /PGA	: Spectral acceleration ratio
SSEI	: Soil-structure-earthquake interaction
SSI	: Soil-structure interaction
SPT	: Standard penetration test
T _m	: Mean period of an earthquake
T _p	: Predominant period of an earthquake
T _{str}	: Fixed base (natural) period of a structure
T _{soil}	: Natural period of a soil site
\widetilde{T}	: Lengthened period of a structure
u	: Pore pressure
U _{hydrostatic}	: Hydrostatic pore pressure
Vs	: Shear wave velocity
W	: Weight of structure
Wc	: Water content

X	: Horizontal distance relative to the centerline of the structure,
α	: Initial (static) shear stress ratio
ϵ_{v}	: Volumetric strain
$\Delta \sigma'_{v}$: Structural-induced vertical stress difference
φ	: Internal friction angle
γ	: Shear strain
γ_{dry}	: Dry unit weight of soil
γ_n	: Natural unit weight of soil
γ_{sat}	: Saturated unit weight of soil
σ	: Structure-to-soil stiffness ratio
$\sigma_{b}{}'$: Effective vertical stress at the base of a structure
$\sigma_{v}{}^{\prime}$: Effective vertical stress
σ_{v}	: Total vertical stress
$\sigma'_{\scriptscriptstyle SSI}$: Effective vertical stress induced by both the structure and the soil.
τ	: Shear stress
$ au_{av}$: Average cyclic shear stress
$ au_{base}$: Cyclic base shear stress
$ au_{soil}$: Cyclic shear stress in soil mass

- τ_0 : Initial static shear stress
- ξ_R : Relative state parameter index
- v : Poisson's ratio

CHAPTER 1

INTRODUCTION

1.1. RESEARCH STATEMENT

The aim of these studies includes the development of a simplified procedure for the assessment of seismic liquefaction triggering of soils beneath structure foundations. Within this scope, three dimensional, numerical, soil-structure interaction analyses were performed to simulate both static and seismic stress state and performance. Founded on the results of these analyses, a probabilistically-based simplified procedure is defined. This simplified procedure is then verified by well-documented field case histories of liquefaction-induced building foundation failures, as well as centrifuge and shaking table test results.

1.2. LIMITATIONS OF PREVIOUS STUDIES

Liquefaction of soils, defined as significant reduction in shear strength and stiffness due to increase in pore pressure, continues to be a major cause of structural damage and loss of life after earthquakes (e.g.; the 1964 Alaska, 1964 Niigata, 1983 Nihonkai-Chubu, 1989 Loma Prieta, 1993 Kushiro-Oki, 1994 Northridge, 1995 Hyogoken-Nambu (Kobe), 1999 Kocaeli and 1999 Ji-Ji earthquakes). Various researchers have tried to quantify the risk of seismic soil liquefaction initiation through the use of both deterministic and probabilistic techniques based on laboratory test results and/or correlation of in-situ "index" tests with observed field performance data. Seed and Idriss (1971) proposed a widely accepted and used methodology, commonly known as "simplified procedure", where cyclic stress ratio (CSR), and overburden-, fines-, and the procedure-corrected Standard Penetration Test (SPT) blow-counts (N_{1,60}) are selected as the load and capacity terms, respectively, for the assessment of seismic soil liquefaction initiation.

The simplified procedure is originally proposed for free field level site conditions, where vertical and horizontal directions are the major and minor principal stress directions, and seismically-induced shear stresses oscillate along the horizontal plane. Unfortunately, these assumptions are not satisfied for soils beneath structure foundations due to i) presence of foundation loads complicating the static stress state, ii) kinematic and inertial interaction of the superstructure with the foundation soils and seismic excitation. Figure 1.2-1 and Figure 1.2-2 schematically illustrate the differences in static and seismic stress states for soils under free field and structure-induced loading conditions, respectively.

Addressing the effects of the different static stress state in liquefaction initiation response, series of corrections, formerly known as K_{α} and K_{σ} , were proposed later to the original procedure. In the literature, there exist contradicting arguments regarding if and how the presence of an overlying structure and foundation element affects liquefaction triggering potential and how these corrections should be applied. Thus, within the confines of this thesis, it is intended to resolve this controversial, yet important issue.

1.3. SCOPE OF THE STUDY

Following this introduction, in Chapter 2 an overview of existing studies focusing on liquefaction definitions, liquefaction triggering, potentially liquefiable soils, post.



Figure 1.2-1. Schematic view of free field stress conditions before and during seismic excitation



Figure 1.2-2. Schematic view of stress conditions of soil-structure system before and during seismic excitation

4

liquefaction strength, post liquefaction deformations and effects of structures on soil liquefaction triggering is presented

In Chapter 3, effects of overlying structures on liquefaction triggering potential of foundation soils, and an overview of existing studies regarding soil-structureearthquake interaction (SSEI) from liquefaction point of view are presented. Additionally, existing methods assessing liquefaction triggering potential for foundation soils are discussed.

In Chapter 4, numerical modeling aspects of SSEI are discussed. Generic soil and structural systems, modeling parameters, one and three dimensional verifications, processing of SSEI analyses results are presented in detail.

Chapter 5 presents the derivation of the proposed probabilistically-based simplified SSEI procedure. The predictions of the proposed simplified procedure are compared with the ones of 3-D static and seismic simulations. Variations of cyclic shear stresses with depth, as estimated by numerical simulations and simplified procedure are illustrated.

Chapter 6 presents the verification and validation of the proposed simplified SSEI procedure with field case histories, and existing shaking table and centrifuge test results. Interpretation of the results is illustrated by a forward analysis.

Finally, Chapter 7 presents the summary and major conclusions of the thesis, in addition to the recommendations on how to use the proposed simplified methodology. Probable future work and limitations of this study is also presented in this chapter.

CHAPTER 2

AN OVERVIEW ON SEISMIC SOIL LIQUEFACTION

2.1 INTRODUCTION

In this chapter, an overview of available literature regarding seismic soil liquefaction engineering is presented. As part of the discussion on seismic soil liquefaction initiation, a brief review on i) liquefaction definitions and mechanisms, ii) simplified procedure, iii) potentially liquefiable soils, iv) post-liquefaction strength and deformations is presented. A detailed presentation of the available methods to assess soil-structure-earthquake interaction (SSEI) with special emphasis on seismic soil liquefaction triggering is discussed in the following chapter.

2.2 LIQUEFACTION DEFINITIONS and ITS MECHANISMS

The term "liquefaction" has been first used by Terzaghi and Peck (1948) to describe the significant loss of strength of very loose sands causing flow failures due to slight disturbance. Similarly, Mogami and Kubo (1953) used the same term to define shear strength loss due to seismically-induced cyclic loading. However, its importance has not been fully understood until 1964 Niigata earthquake, during which the significant causes of structural damage were reported to be due to tilting and sinking of the buildings founded on saturated sandy soils with significant soil liquefaction potential. Robertson and Wride (1997) reported that as an engineering term, "liquefaction" has been used to define two mainly related but different soil responses during earthquakes: flow liquefaction and cyclic softening. Since both mechanisms can lead to quite similar consequences, it is difficult to distinguish. However, the mechanisms are rather different, and will be discussed next.

2.2.1 Flow Liquefaction

In the proceedings of the 1997 NCEER Workshop, flow liquefaction is defined as follows:

"Flow liquefaction is a phenomenon in which the equilibrium is destroyed by static or dynamic loads in a soil deposit with low residual strength. Residual strength is defined as the strength of soils under large strain levels. Static loading, for example, can be applied by new buildings on a slope that exert additional forces on the soil beneath the foundations. Earthquakes, blasting, and pile driving are all example of dynamic loads that could trigger flow liquefaction. Once triggered, the strength of a soil susceptible to flow liquefaction is no longer sufficient to withstand the static stresses that were acting on the soil before the disturbance. Failures caused by flow liquefaction are often characterized by large and rapid movements which can lead to disastrous consequences."

The main characteristics of flow liquefaction are that:

- it applies to strain softening soils only, under undrained loading,
- it requires in-situ shear stresses to be greater than the ultimate or minimum soil undrained shear strength,
- it can be triggered by either monotonic or cyclic loading,
- for failure of soil structure to occur, such as a slope, a sufficient volume of the soil must strain soften. The resulting failure can be a slide or a flow depending on the material properties and ground geometry, and

 it can occur in any meta-stable structured soil, such as loose granular deposits, very sensitive clays, and silt deposits.

Flow liquefaction mechanism can be illustrated as shown in Figure 2.2-1.



Figure 2.2-1. Flow liquefaction

2.2.2 Cyclic Softening

Similarly, cyclic softening definitions and mechanisms, consistent with 1997 NCEER Workshop proceedings are summarized below:

"Cyclic softening is another phenomenon, triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic softening develop incrementally because of static and dynamic stresses that exist during an earthquake. Two main engineering terms can be used to define the cyclic softening phenomenon, which applies to both strain softening and strain hardening materials."

2.2.2.1 Cyclic Mobility

Cyclic mobility can be identified by the facts that:
- it requires undrained cyclic loading during which shear stresses are always greater than zero; i.e. no shear stress reversals develop,
- zero effective stress will not develop,
- deformations during cyclic loading will stabilize, unless the soil is very loose and flow liquefaction is triggered,
- it can occur in almost any sand provided that the cyclic loading is sufficiently large in size and duration, but no shear stress reversals occurs, and
- clayey soils can experience cyclic mobility, but deformations are usually controlled by rate effects (creep).

Cyclic mobility mechanism is illustrated as shown in Figure 2.2-2. Figure on the left shows the variation of shear stress during cyclic loading and the figure on the right is the development of the shear strain during this loading. As this figure implies, no zero effective stress develop during cyclic loading.



Figure 2.2-2. Cyclic mobility

2.2.2.2 Cyclic Liquefaction

Cyclic liquefaction can be identified by the facts that:

- it requires undrained cyclic loading during which shear stress reversals occur or zero shear stress can develop; i.e. occurs when in-situ static shear stresses are low compared to cyclic shear stresses,
- it requires sufficient undrained cyclic loading to allow effective stress to reach essentially zero,
- at the point of zero effective stress no shear stress exists. When shear stress is applied, pore water pressure drops as the material tends to dilate, but a very soft initial stress strain response can develop resulting in large deformations,
- deformations during cyclic loading can accumulate to large values, but generally stabilize when cyclic loading stops,
- it can occur in almost all sands provided that the cyclic loading is sufficiently large in size and duration, and
- clayey soils can experience cyclic liquefaction but deformations are generally small due to cohesive strength at zero effective stress.
 Deformations in clays are often controlled by time rate effects.



Figure 2.2-3. Cyclic liquefaction

Cyclic liquefaction mechanism is illustrated as shown in Figure 2.2-3. The figure on the left shows the variation of stress state during cyclic loading,

whereas the figure on the right illustrates modulus degradation. As the figures imply, zero effective stress state develops and thus results in zero shear strength for non-cohesive soils. Strains (deformations) during cyclic loading may reach to higher values as presented in the right figure."

2.3 SEISMIC SOIL LIQUEFACTION ENGINEERING

The first step in liquefaction engineering is to determine if soils of interest are potentially liquefiable or not. If they are concluded to be liquefiable, following steps are defined as the determination of post–liquefaction strength, post–liquefaction stability, post –liquefaction deformations and displacements. Consequences of these deformations and, if necessary, mitigation measures need to be also addressed. A summary of these analyses steps which will be discussed next are shown in Table 2.3.1.

1	Assessment of the likelihood of "triggering" or initiation of soil liquefaction.
2	Assessment of post-liquefaction strength and overall post-liquefaction stability.
3	Assessment of expected liquefaction-induced deformations and displacements.
4	Assessment of the consequences of these deformations and displacements.
5	Implementation (and evaluation) of engineered mitigation, if necessary.

Table 2.3-1. Liquefaction engineering steps

2.3.1 Potentially Liquefiable Soils

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" summarized in Table 2.3-2 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) plasticity index is a major controlling factor in the cyclic response of fine grained soils. These criteria are then modified by Andrews and Martin (2000) for USCS-based silt and clay definitions, as shown in Table 2.3-3.

Potentially Liquefiable Soils		
Fines Content (< 0.005 mm)	≤ 15%	
Liquid Limit (LL)	≤ 35%	
Water Content (w _c)	≥(0.9xLL)%	

Table 2.3-2. Chinese Criteria proposed by Seed and Idriss (1982).

Table 2.3-3. Modified Chinese Criteria by Andrews and Martin (2000).

	Liquid Limit < 32%	Liquid Limit \ge 32%
Clay Content (< 0.002 mm) < 10%	Potentially Liquefiable	Further studies required considering plastic non-clay sized grains
Clay Content (< 0.002 mm) $\geq 10\%$	Further studies required considering non-plastic clay sized grains	Non-Liquefiable

Bray et al. (2001) has concluded that the Chinese criteria may be misleading in the concept of percent "clay-size". According to their findings, percent of clay minerals

and their activities are more important than the percent of "clay-size". They give the example of fine quartz particles which may be smaller than 2 - 5 mm, but they me largely non-plastic and may be susceptible to liquefaction, behaving as a cohesionless material under cycling loading. Recommendations of Bray et al. (2001) are presented in Figure 2.3-1.

Seed et al. (2003) recommended a new criterion inspired from case histories and cyclic testing of "undisturbed" fine grained soils compiled after 1999 Kocaeli-Turkey and Chi Chi-Taiwan earthquakes as shown in Figure 2.3-2. These criteria classify saturated soils with a plastic index (PI) less than 12 and liquid limit (LL) less than 37 as potentially liquefiable, provided that the soil natural moisture content is greater than 80% of the liquid limit (0.8·LL).



Figure 2.3-1. Potentially liquefiable soils (Bray et al., 2001)



Figure 2.3-2. Criteria for liquefaction susceptibility of fine-grained sediments proposed by Seed et al. (2003).

The most recent attempt for determining potentially liquefiable soils was by Boulanger and Idriss (2004). Based on cyclic laboratory test results and an extensive engineering judgment, they have recommended the new criteria summarized in Figure 2.3-3. 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 2.3-3 is not to scale, thus a direct comparison between cyclic resistance ratios 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"



Figure 2.3-3. Criteria for differentiating between sand-like and clay-like sediment behavior proposed by Boulanger and Idriss (2004).

As the concluding remark of this section, it is believed that Seed et al. (2003) and Bray et al. (2001) methodologies will continue to establish the state of practice until a performance (deformation) based comparisons of liquefaction triggering potentials are possible.

2.3.2 Seismic Soil Liquefaction Triggering

If the soil is judged to be potentially liquefiable, the next step involves the assessment of liquefaction triggering potential under seismic or cyclic loading. Two different models are available for evaluating liquefaction triggering potential:

 Methods calibrated based on field performance of soil sites shaken by earthquakes, where descriptive parameters were selected as standard penetration test (SPT) blow-counts, or cone penetration test (CPT) tip resistance, shear wave velocity (V_s), electrical properties, etc., and a measure of intensity of earthquake shaking (cyclic stress ratio, cyclic strain ratio, accelerogram energy, etc.)

 Laboratory-based methods based on cyclic testing of "undisturbed" or reconstituted soil samples.

In practice, the first method is widely used in liquefaction triggering assessment of free field soil sites. Laboratory-based methods are used rarely as it is difficult or impossible to obtain "undisturbed" soil samples from cohesionless soil deposits and realistically simulate field stress and earthquake loading conditions. Following section describes the evaluation of liquefaction triggering potential for free field soil sites.

2.3.2.1 Liquefaction Triggering Assessment for Free Field Soil Sites

2.3.2.1.1 Cyclic Stress Ratio, CSR

Seed and Idriss (1971) proposed cyclic stress ratio, CSR, which is defined as the average cyclic shear stress, τ_{av} , developed on the horizontal plane of a soil layer due to vertically propagating shear waves normalized by the initial vertical effective stress, σ'_{v} , to incorporate the increase in shear strength due to increase in effective stress.

$$CSR = \frac{\tau_{av}}{\sigma'_{v}} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_{v}}{\sigma'_{v}} \cdot r_{d}$$
(2 - 1)

Corollary, CRR is used for free field cases where no initial (static) shear stresses exist on the horizontal plane and $\sigma'_{v0} = 100$ kPa, and for an earthquake of moment magnitude, $M_w = 7.5$. Later, more normalization/correction terms were introduced to Equation (2 – 1) to incorporate the effects of magnitude (duration) of the earthquake shaking (magnitude scaling factor, MSF), nonlinear shear strength-effective stress relationship (K_{σ}), initial static driving shear stresses (K_{α}), etc. which will be discussed in detail in the following chapter (Chapter 3). However, the stress reduction factor, r_d will be discussed herein, as it is critical for the assessment of CSR.

2.3.2.1.2 Estimating Cyclically-induced Shear Stresses for Free Field Level Sites

According to the simplified procedure proposed by Seed and Idriss (1971), shear stresses for a rigid soil body due to vertically propagating shear waves at the soil depth "h" and for the time "t" can be calculated as:

$$\tau(t)_{rigid} = \gamma_n \cdot h \cdot \frac{a(t)}{g} \tag{2-2}$$

where γ_n is the natural unit weight of soil, a(t) is the ground surface acceleration at time "t", and g is the gravitational acceleration. A schematic view is presented in Figure 2.3-4. Due to the fact that soil behaves as a deformable body and shear stresses develop within this deformable body, the shear stresses will be less than those predicted by Equation (2 - 2). For this reason, a stress reduction factor, r_d , needs to be incorporated to model this reduction.

$$\tau(t)_{deformable} = \gamma_n \cdot h \cdot \frac{a(t)}{g} \cdot r_d \qquad (2-3)$$

To convert irregular forms of seismic shear stress time histories to a simpler equivalent series of uniform stress cycles, an averaging scheme is required. Based on laboratory test data, it has been found that reasonable amplitude to select for the "average" or equivalent uniform stress, τ_{av} , is about 65% of the maximum shear stress, τ_{max} , as:

$$\tau_{av} \approx 0.65 \cdot \gamma_n \cdot h \cdot \frac{a_{\max}}{g} \cdot r_d \tag{2-4}$$

where a_{max} is defined as peak ground acceleration.



Figure 2.3-4. Procedure for determining maximum shear stress, $(\tau_{max})_r$, (Seed and Idriss, 1982)

2.3.2.1.3 Stress reduction factor, r_d

Non-linear mass participation factor (stress reduction factor), r_d , was first introduced by Seed and Idriss (1971) as a parameter describing the ratio of cyclic stresses for a flexible soil column to the cyclic stresses for a rigid soil column. $r_d = 1.00$ corresponds to either a rigid soil column response or the value at the ground surface, and this ratio degrades rapidly with depth. In that study, they have proposed a range of r_d values estimated by site response analyses results covering a range of earthquake ground motions and soil profiles. The average curve was then proposed for the upper 12 m (40 ft) for all earthquake magnitudes and soil profiles. This average curve is presented in Figure 2.3-5.

Liao and Whitman (1986) have proposed the following equations in which z is the depth from the ground surface in meters.

$$r_d = 1.00 - 0.00765z$$
 for $z \le 9.15m$ (2 - 5a)

$$r_d = 1.174 - 0.0267z$$
 for $9.15 \le z \le 23.0$ m (2 - 5b)



Figure 2.3-5. Proposed r_d by Seed and Idriss, 1971

Shibata and Teperaksa (1988) introduced an alternative form of equation as given below:

$$r_d = 1.00 - 0.015z \tag{2-6}$$

An approximate value, presented in Equation (2 - 7) was proposed by NCEER (1997).

$$r_d = \frac{1.0 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.0 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^2}$$
(2 - 7)

In 1999, Idriss improved the work of Golesorkhi (1989) and concluded that for the conditions of most practical interest, the parameter r_d could be adequately expressed as a function of depth and earthquake magnitude. He proposed the following equations:

For $z \le 34$ m:

$$\ln(r_d) = \alpha(z) + \beta(z)M_w \qquad (2 - 8a)$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$$
 (2 - 8b)

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$$
 (2 - 8c)

For z > 34 m:

$$r_d = 0.12 \exp(0.22M_w) \tag{2-8d}$$

where " M_w " is the moment magnitude of the earthquake. Plots of r_d for different magnitudes of earthquake calculated by using Equations (2 – 8) are presented in Figure 2.3-6.

Cetin et al. (2004) introduced a closed form solution, where r_d depends on not only the level of shaking and depth but also the stiffness of the soil profile (represented by V_s). This solution is presented in Equations (2 – 9) and (2 – 10).

For d < 20 m

$$r_{d} = \left[\frac{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (-d + 0.0785 \cdot V_{s,12m}^{*} + 7.586)}}{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (0.0785 \cdot V_{s,12m}^{*} + 7.586)}}\right]$$
(2 - 9)



Figure 2.3-6. Variations of stress reduction coefficient with depth and earthquake magnitude (Idriss, 1999)

For $d \ge 20 \text{ m}$

$$r_{d} = \left[\frac{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot \left(-d + 0.0785 \cdot V_{s,12m}^{*} + 7.586\right)}}{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot \left(0.0785 \cdot V_{s,12m}^{*} + 7.586\right)}}}\right] - 0.0046(d - 20)(2 - 10)$$

where

 a_{max} : peak ground acceleration at the ground surface (g)

 M_w : earthquake moment magnitude

d : soil depth beneath ground surface (m)

 $V_{s,12m}^{*}$: equivalent shear wave velocity defined as:

$$V_s^* = \frac{H}{\sum \frac{h_i}{V_{s,i}}}$$
(2 - 11)

where, H is the total soil profile thickness (m), h_i is the thickness of the ith sub-layer (m), and $V_{s,i}$ is the shear wave velocity within the ith sub-layer (m/s).

2.3.2.1.4 Capacity term, CRR

A comparison of the level of CSR as the load (demand) term and cyclic resistance ratio, CRR as the capacity term helps concluding about liquefaction triggering possibility. Field performance of sands and silty sands during actual earthquakes have shown that there is a good correlation between the resistance of soil to initiation or "triggering" of liquefaction under earthquake shaking and soil penetration resistance (Seed et al., 1983; Tokimatsu and Yoshimi, 1981). For example, increase in relative density increases both the penetration resistance and liquefaction resistance potential; increase in time under pressure also increases both the penetration resistance and liquefaction resistance potential. Based on these observations and more, Seed et al. (1983) presented an empirical correlation where N_{1.60} and CSR were chosen as the capacity and demand parameters, respectively. With the addition of new data in 1984, Seed et al.(1984a), proposed the liquefaction triggering curves which are presented in Figure 2.3-7 based on case history data from 1975 Haicheng and 1976 Tangshan (China), 1976 Guatemala, 1977 Argentina, and 1978 Miyagiken-Oki (Japan) and U.S. earthquakes. Seed et al. (1984a) relationship has been widely accepted and used in practice although it is rather dated.



Figure 2.3-7. Liquefaction boundary curves recommended by Seed et al. (1984a)

Cetin et al. (2004) introduced new chart solutions similar to Seed et al. (1984a) deterministic curves for soil liquefaction triggering by using higher-order probabilistic tools. Moreover, these new correlations were based on a significantly extended database and improved knowledge on standard penetration test, site-specific earthquake ground motions and in-situ cyclic stress ratios. These charts can be seen in Figure 2.3-8 and Figure 2.3-9. For comparison purposes Seed et al. (1984a)'s deterministic boundary is also shown on the figures.



Figure 2.3-8. Recommended probabilistic SPT-based liquefaction triggering correlation for M_w =7.5 and σ_v '=1.0 atm. (Cetin et al. 2004)



Figure 2.3-9. Deterministic SPT-based liquefaction triggering correlation for M_w =7.5 and σ_v' =1.0 atm. with adjustments for fines content. (Cetin et al. 2004)

Curves in Figure 2.3-7 through Figure 2.3-9 represent the resistance of soils to liquefaction referred by cyclic resistance ratio, (CRR). Knowing the demand term, CSR (estimated by simplified procedure) and the capacity term, CRR (from the figures or empirical correlations) it is easy to determine if the soil body in concern will liquefy or not, i.e. if CSR > CRR, the soil is concluded to liquefy and vice versa.

2.3.3 Post Liquefaction Strength

If soils are concluded to be liquefied during an earthquake, then corollary question is whether a liquefaction-induced instability is expected or not. This requires the estimation of post liquefaction shear strength of soils. There exist two methods for the evaluation of post liquefaction (or residual) strength of liquefied soils: i) empirical correlations and ii) laboratory tests.

Empirical correlations attempt to develop a relationship between soil density state (generally in terms of penetration resistance) and residual strength. They are based on back analyses of liquefaction induced ground failure case histories (Seed, 1987; Davis et al, 1988; Seed and Harder, 1990; Robertson et al., 1992; Stark and Mesri, 1992; Ishihara, 1993; Wride et al., 1999; Olson and Stark, 2002; Olson and Stark, 2003).

The alternative approach involves the evaluation of the residual strength by laboratory tests (Castro, 1975; Castro and Poulos 1977; Poulos et al., 1985; Robertson et al., 2000). Originally, Castro (1975) proposed a laboratory-based approach which required performing a series of consolidated-undrained tests for the estimation of undrained steady-state strength of 'undisturbed' soil samples. However, soon it is realized that steady state shear strength is a function of in situ void ratio and small variations in void ratio may lead to large differences is steady state shear strengths (Poulos et al., 1985).

Vaid and Thomas (1995) presented experimental studies of the post liquefaction behavior of sands in triaxial tests. In their study, cyclic loading leading to liquefaction is studied by tests performed on samples with relative densities ranging from loose to dense consolidated at a range of confining stresses.

Among these two methods, empirically based one is widely accepted and used due to the fact that it is based on actual case histories. Due to high sensitivity of residual shear strength to small variations of void ratio and difficulties in simulating field stress and loading conditions, laboratory-based techniques are not widely used in engineering analyses. Thus, within the confines of this thesis, emphasis is given to the discussion of empirically-based methods, which are presented next.

Quantifying post-liquefaction strength has been the subject of numerous research studies. Seed and Harder (1990), have calculated post-liquefaction values of $s_{u,(critical,mob)}$ by back analyses of flow failure embankments during earthquakes. The values of $s_{u,(critical,mob)}$ were correlated with pre-earthquake $(N_1)_{60}$ values of the sand that had liquefied.

Stark and Mesri (1992) re-analyzed 20 case histories of liquefaction-induced failure of embankments and proposed a linear correlation between the undrained residual strength ratio S_u / σ'_{v0} and the equivalent normalized SPT blow count for clean sand, $(N_1)_{60-cs}$:

$$\frac{s_u}{\sigma'_{v0}} = 0.0055 \cdot (N_1)_{60-cs}$$
(2 - 12)

where $(N_1)_{60-cs} = (N_1)_{60} + \Delta N_1$

Olson and Stark (2003), proposed the use of yield strength ratio back calculated from static liquefaction flow failure case histories. Yield strength ratio is calculated depending on the penetration resistance (either SPT-N or q_c) by using the following equations:

for $q_{c1} \le 6.5$ MPa

$$\frac{s_u(yield)}{\sigma'_{v0}} = 0.205 + 0.0142(q_{c1}) \pm 0.04 \qquad (2 - 13)$$

for $N_{1,60} \le 12$

$$\frac{s_u(yield)}{\sigma'_{v0}} = 0.205 + 0.0075[(N_1)_{60}] \pm 0.04 \qquad (2 - 14)$$

If liquefaction is triggered, the liquefaction shear strength ratio is estimated from the following relationships as proposed by Olson and Stark (2002):

for $q_{c1} \le 6.5$ MPa

$$\frac{s_u(LIQ)}{\sigma'_{v0}} = 0.03 + 0.0143(q_{c1}) \pm 0.03$$
 (2 - 15)

for $N_{1,60} \leq 12$

$$\frac{s_u(LIQ)}{\sigma'_{v0}} = 0.03 + 0.0075[(N_1)_{60}] \pm 0.03$$
 (2 - 16)

Idriss and Boulanger (2007) presented a new set of recommended SPT-and CPTbased relationships for estimating the ratio of residual shear strength to initial vertical effective stress, S_r / σ'_{v0} , for liquefied non-plastic soils after revisiting the case histories compiled over the past 20 years. The values of S_r / σ'_{v0} for case histories are plotted as a function of $(N_1)_{60cs-Sr}$ in Figure 2.3-10.



Figure 2.3-10. Residual shear strength ratio S_r / σ'_{v0} versus equivalent cleansand SPT corrected blow count (Idriss and Boulanger, 2007)

The relationships shown in this figure can be interpreted with the following expressions:

When void redistribution is significant:

$$\frac{s_{r}}{\sigma'_{v0}} = \exp\left(\frac{\left(N_{1}\right)_{60cs-Sr}}{16} + \left(\frac{\left(N_{1}\right)_{60cs-Sr}}{21.2}\right)^{3} - 3.0\right) \le \tan\phi' \qquad (2 - 17)$$

When void redistribution is negligible:

$$\frac{s_{r}}{\sigma'_{v0}} = \exp\left(\frac{(N_{1})_{60cs-Sr}}{16} + \left(\frac{(N_{1})_{60cs-Sr}}{21.2}\right)^{3} - 3.0\right)$$

$$\times \left(1 + \exp\left(\frac{(N_{1})_{60cs-Sr}}{2.4} - 6.6\right)\right) \le \tan\phi'$$
(2 - 18)

For CPT-based correlation of residual strength, curves presented in Figure 2.3-11 are proposed. Similarly these relationships are closely approximated by Equations (2 -19 and 2-20):



Figure 2.3-11. Residual shear strength ratio S_r / σ'_{v0} versus equivalent cleansand CPT normalized corrected tip resistance (Idriss and Boulanger, 2007)

When void redistribution is significant:

$$\frac{s_{r}}{\sigma'_{v0}} = \exp\left(\frac{q_{c160cs-Sr}}{24.5} - \left(\frac{q_{c160cs-Sr}}{61.7}\right)^{2} + \left(\frac{q_{c160cs-Sr}}{106}\right)^{3} - 4.42\right) \le \tan\phi' \quad (2 - 19)$$

When void redistribution is negligible:

$$\frac{s_{r}}{\sigma'_{v0}} = \exp\left(\frac{q_{c160cs-Sr}}{24.5} - \left(\frac{q_{c160cs-Sr}}{61.7}\right)^{2} + \left(\frac{q_{c160cs-Sr}}{106}\right)^{3} - 4.42\right) \times \left(1 + \exp\left(\frac{q_{c160cs-Sr}}{11.1} - 9.82\right)\right) \le \tan\phi'$$
(2 - 20)

2.3.4 Post Liquefaction Deformations

After confirming that liquefaction-induced instability is of no engineering risk, next step is to determine how much the soil site is expected to deform. Current state of knowledge on the determination of post liquefaction deformations is not as satisfactory as liquefaction triggering assessment studies. This is mainly due to the fact that it is observed that the failures after liquefaction are catastrophic and accompanied by large deformations which make back-analysis or documentation very difficult. Major failure mechanisms after liquefaction have been presented in Figure 2.3-12. Figure 2.3-13 (after Seed at el. 2001) shows the mechanisms of large deformations.



(c) Localized, Non-directionally Preferential Differential Lateral Displacements

Figure 2.3-12. Schematic Examples of Modes of "Limited" Liquefaction-Induced Lateral Translation (Seed et al., 2001)

I - Liquefied zone with low residual undrained strength



(e) Rotational and/or Translational Sliding

Figure 2.3-13. Schematic Examples of Liquefaction-Induced Global Site Instability and/or "Large" Displacement Lateral Spreading (Seed et al., 2001)

Liquefaction-induced soil deformations are classified into two components: volumetric and deviatoric. As the names imply, volumetric straining is due to changes in mean effective stress state, whereas deviatoric straining can be attributed to shear stresses which may cause significant change in the shape of the soil body. As soil layers shake, loose soil particles tend to compress to a denser state which in

turn cause settlement. Non-cemented saturated loose cohesionless soils are more susceptible to this type of deformations, compared to other types of soils.

Various researchers have tried to quantify liquefaction-induced (cyclic) soil deformations. For this purpose, i) numerical analyses tools such as finite difference and finite element analyses, ii) analytical methods and iii) empirical methods are widely used. New computing techniques such as fuzzy logic, artificial neural networks are also introduced in the calculation of the magnitudes of the deformations in recent years.

Finite difference and finite element methods along with powerful constitutive laws can be used in predicting seismically-induced deformations. Finn et al., (1986), France et al., (2000) have developed methodologies for estimating ground deformations using finite difference/element methods. Physical conditions of the site (geometric properties of the site, soil properties and etc.) are taken into account while developing the analytical methods. Newmark (1965), Towhata et al. (1992), Yegian et al. (1994), Bardet et al. (1999) have proposed different analytical procedures for the determination of liquefaction-induced ground deformations.

Empirical models predict the deformations through the use of deterministic techniques based on laboratory test results and/or correlation of in-situ "index" tests with observed field performance data. Simplified procedure's (Seed and Idriss, 1971) capacity and demand terms, $N_{1,60}$ and CSR, respectively, are generally used in predicting the magnitude of deformations. Tokimatsu and Seed (1984) recommended constitutive model and chart solutions for the estimation of cyclic soil deformations using the results of laboratory cyclic tests performed on clean sands and calibrated with case history performance data. Similarly, Ishihara and Yoshimine (1992) proposed correlations where normalized demand term was chosen as factor of safety against liquefaction, and defined relative density (D_R) or cone tip resistance (q_c), or $N_{1,60}$ as the capacity term. Shamoto et al. (1998) recommended similar chart solutions based on cyclic torsional shear tests for the estimation of post

cyclic soil deformations as well as semi-empirical correlations. Recently, Wu et al. (2003) proposed cyclically-induced limiting shear and post-cyclic volumetric strain correlations based on the results of cyclic simple shear test results. Wu and Seed (2004) verified this volumetric strain relationship with ground settlement field case history data from various earthquakes.

No method, yet, has been developed to predict the post-liquefaction ground settlement and lateral spreading in a unified manner. Theoretical investigations on the physical essentials and conditions under which both types of ground deformations can occur are few. However, significant ground settlements and lateral spreading occurred extensively during past strong earthquakes not only in the liquefied soil where static driving shear stresses are present, but also in liquefied level sandy ground with a sufficiently large extend. It is known that residual post-liquefaction stress-strain settlement and lateral spreading are not independent of each other, so, in principle, both cannot be evaluated separately.

As the deformation analyses are beyond the scope of this study, a brief overview of the available methods used to estimate cyclically-induced ground deformations in terms of settlement and lateral ground spreading is presented below.

2.3.4.1 Liquefaction-induced Ground Settlement

Several methods have been introduced for predicting the liquefaction-induced ground settlement so far. Some of these including Shamoto et al. (1998), Ishihara and Yoshimine (1992), Tokimatsu and Seed (1984) are summarized below:

Shamoto et al. (1998), have proposed a methodology with a physical basis and developed chart solutions for evaluating earthquake-induced ground settlement and lateral spreading. These chart solutions are shown in Figure 2.3-14 (a) and (b).

Ishihara and Yoshimine (1992), have concluded that the induced volumetric strain level after liquefaction is related not only to the density of soil but more importantly to maximum shear strain level which the sand has undergone during the application



Figure 2.3-14. Relationship between normalized SPT-N value, dynamic shear stress and residual shear strain potential (a) for FC= 10% (b) for FC= 20% (Shamoto et al., 1998)

of cyclic loads. In order to find the ground settlements, first of all, the relationship between the volume change of saturated sands and maximum shear strains needs to be known. Figure 2.3-15 shows the proposed relationships. The next step is the evaluation of maximum shear strains, which the sand will undergo during the application of shaking in an earthquake. To find the maximum shear strain, factor of safety against liquefaction, F_l needs to be determined and shear strain then is read from Figure 2.3-16. After combining Figure 2.3-15 and Figure 2.3-16, a relationship between F_l and ε_v can be plotted as shown in Figure 2.3-17. It should be noted that if these curves are to be used for practical purposes, the axial strain values need to be converted to shear strain values by Equation (2 - 21);

$$\gamma_{\max} = 1.5 \times \varepsilon_{\max} \tag{2-21}$$

Volumetric strains are estimated for each layer in a soil deposit and the amount of settlement on the ground surface can be obtained by adding the vertical displacements produced in each layer of the deposit.



Figure 2.3-15. Post-liquefaction volumetric strain plotted against maximum shear strain. (Ishihara, 1996)



Figure 2.3-16. Relation between factor of safety and maximum shear strain (Ishihara, 1996)



Figure 2.3-17. Chart for determining volumetric strain as a function of factor of safety against liquefaction (Ishihara, 1996)

Tokimatsu and Seed (1984) adopted the simplified procedure's capacity and demand terms, $N_{1,60}$ and CSR, respectively and performed laboratory triaxial tests on clean sands which were then calibrated with the case history data They recommended a constitutive model, as well as set of chart solutions, for the estimation of cyclic soil deformations. The charts for deviatoric and volumetric strains can be seen in Figure 2.3-18 (a) and (b) respectively.



Figure 2.3-18. Chart for determining cyclic deformations as a function of N_{1,60}(a) Deviatoric strains, (b) Volumetric Strains (Tokimatsu and Seed, 1984)

As can be seen from previously discussed studies, although all of them use the same basis, there is a wide scatter in the numerical values. Another important aspect of these deformation estimations is that they are developed for free field, level soil site cases with no structures on them. Thus, liquefaction induced seismic deformation potential for cases with structures need to be further studied.

2.3.4.2 Lateral Ground Spreading

The lateral ground spreading can be estimated empirically or analytically using the methods proposed by Hamada et al. (1986) and Towhata et al. (1992) as well as other investigators.

Hamada et al. (1986) introduced a simple and easy to use empirical equation for the determination of lateral ground deformations, which is presented in Equation (2-22). This equation is based on limited number of cases and its use should be limited to the cases with similar conditions.

$$D_{H} = 0.75 \cdot H^{1/2} \cdot \theta^{1/3} \tag{2-22}$$

where

D_H : predicted horizontal ground displacement (m)

- H : thickness of the liquefied zone (m) (if more than one sub-layer liquefies, H is the distance from the top of the top liquefied layer to the bottom of the bottom liquefied layer)
- θ : larger slope of either the ground surface or liquefied zone lower boundary (%)

Shamoto et al. (1998) proposed chart solutions for predicting lateral ground displacements as mentioned previously. Predicted lateral displacements by these chart solutions need to be multiplied by a factor of 0.16 in order to predict lateral displacements of non-sloping ground.

Youd et al. (2002) modified the model of Bartlett and Youd (1992 – 1995) and introduced a new model which was developed by using multi-linear regression on an extensive case history database. The model was developed separately for free face and gently sloping ground conditions.

For free face:

$$Log D_{H} = -16.713 + 1.532 \cdot M_{w} - 1.406 \cdot Log R^{*} - 0.012 \cdot R + 0.592 \cdot Log W + 0.540 \cdot Log T_{15} + 3.413 \cdot Log (100 - F_{15}) - 0.795 \cdot Log (D50_{15} + 0.1mm)$$
(2 - 23)

For gently sloping ground:

$$Log D_{H} = -16.213 + 1.532 \cdot M - 1.406 \cdot Log R^{*} - 0.012 \cdot R + 0.338 \cdot Log S + 0.540 \cdot Log T_{15} + 3.413 \cdot Log (100 - F_{15}) - 0.795 \cdot Log (D50_{15} + 0.1mm)$$
(2 - 24)

where

- D_H : horizontal ground displacement (m)
- M_w : Magnitude of the earthquake
- R : horizontal distance to the nearest seismic source or fault rupture (km) and $R^* = R + 10^{(0.89M - 5.64)}$
- W : free face ratio = H/L(%), H is the height of the free face (m) and L is the distance to free face from the point of displacement (m)

- T_{15} : thickness of the saturated layers with $N_{1,60} < 15$ (m)
- F_{15} : average fines content for particles finer than 0.075 mm, within T_{15} (%)
- $D50_{15}$: average D_{50} within T_{15} (mm)

This model involves parameters of earthquake, site and soil conditions. However, it is applicable to mostly large deformation cases rather than lateral spreading cases with limited deformation potential. Free face equation is used when the free face ratio is between 5 and 20%. The equation for slopes is valid when W≤1%. In addition to this, the model is used for the ranges $6 \le M_w \le 8$, $0.1 \le S \le 6\%$, and $1 \le T_{15} \le 15$ m. Another limitation of this method is the fact that it is not applicable to soil sites with significant gravel or silt contents.

The settlement and lateral spread equations presented in this study may give different results as they have different assumptions and limitations. A dependable result may be obtained by a cross-check between all the answers.

2.4 CONCLUDING REMARKS

In this chapter, an overview of the previous studies on liquefaction definitions, liquefaction triggering, "simplified procedure", potentially liquefiable soils, post liquefaction strength, available post liquefaction deformation methods and available methods concerning the effects of soil-structure interaction on soil liquefaction triggering are presented. As discussed earlier, there is somewhat a consensus on liquefaction triggering potential, post liquefaction strength and post liquefaction deformations for free field level sites, whose horizontal planes are free of static shear stresses. However, there is no concesus on liquefaction triggering assessment for soil sites with initial shear stresses and/or extra overburden stresses. The next chapter discusses the available methodology used in liquefaction triggering assessment of foundation soils and an overview of the effects of soil-structure-earthquake interaction from the point of liquefaction triggering potential which constitutes the basis of this thesis.

CHAPTER 3

SOIL – STRUCTURE INTERACTION WITH EMPHASIS ON SEISMIC SOIL LIQUEFACTION TRIGGERING

3.1 INTRODUCTION

In this chapter, a detailed overview of existing studies on soil-structure-earthquake interaction (SSEI) and its effects on liquefaction triggering is presented. Although the extent of knowledge available in the literature regarding this issue is limited, the available studies, including rocking and sliding behavior of the foundations, are attempted to be summarized and presented. The differences in static stress states and seismic responses of free field soil sites and soil sites with superstructures overlying them are discussed. The differences in static stress state and its effects on liquefaction triggering potential are attempted to be resolved through K_{α} and K_{σ} corrections applied on either CSR or CRR.

3.2 EXISTING STUDIES ON SOIL STRUCTURE EARTHQUAKE INTERACTION

Since the very early days of geotechnical earthquake engineering profession, the seismic response of soil and structure interacting sites are acknowledged to be different than that of free field soils sites. However due to complexities in the treatment of these differences, foundation soils are usually treated as if they were free field level soil sites with a major assumption that this treatment is conservatively biased.

A free field infinite elastic medium can sustain two kinds of waves of different velocities which represent different types of body motions. These two waves are usually referred by the terms:

- (a) Dilatational wave (primary wave, P-wave, compression wave, irrotational wave)
- (b) Distortional wave (secondary wave, S-wave, shear wave, equi-voluminal wave)

In an elastic half-space, it is possible to find a third solution for the equations of motion which correspond to a wave, whose motion is confined to a zone near the boundary of the half-space. This wave was first studied by Lord Rayleigh (1885) and later was described in detail by Lamb (1904). The elastic wave described by these two investigators is known as the Rayleigh wave (R-wave) and is confined to the neighborhood of the surface of a half-space. The influence of R-wave decrease rapidly with depth. If more than one interface exists (this interface may be soil layers with varying rigidities, slopes, foundation elements, etc.), waves may not be reflected back to the surface at each layer interface as they are reflected in a typical free field level site. This reflected energy is partially responsible for the complications in a soil – structure – earthquake interaction problem.

The methods discussed in the previous chapter are founded on "simplified procedure" and suffer from the limitation that they are applicable to liquefaction

triggering assessment of free field soil sites. Direct applicability of these methods to foundation soils underlying structural systems is not possible, unless, in the estimation of structural-induced cyclic stress ratio, CSR_{SSEI}, soil-structure and earthquake interaction is properly addressed. More specifically, compared to free field soil sites, the presence of an overlying structural system and its effects should be considered both statically and dynamically. Under static conditions i) extra overburden stresses act on the underlying foundation soils compared to the free field sites and ii) presence of non-zero shear stresses on horizontal planes change the pore pressure response as well as failure pattern beneath the super structure significantly. Similarly, during earthquake shaking, presence of iii) foundation elements (e.g. footing or mat) forms usually a sharp impedance contrast which causes deviations from the free field vertical propagation of shear waves, i.e. kinematic interaction, and iv) an overlying structure due to its inertia will exert additional cyclic shear stresses on to the foundation soils.

Although, its critical importance has been recognized for years, there are very limited number of studies tackling the effects of SSEI from both structural and geotechnical points of views. A summary of the select studies on SSEI with emphasis on seismic soil liquefaction triggering point of view is presented next.

Smith (1969) has worked on a series of laboratory test results and one field case history record which involve flexible structures bearing on cohesionless foundations. The foundations were treated as elastic and inhomogeneous. Both "Winkler" and elastic solid foundations are considered and it is shown that for the latter type physically reasonable distributions of the elastic modulus do not lead to very good predictions of the deflections of the structure, although the deflections within the foundation itself are in agreement with the observed values. On the basis of this observation, flexible plates bearing on an over-consolidated sand foundation, the nature of the inhomogeneity of such a foundation has been deduced and found to be dependent on **relative stiffness** of the composite system.

Veletsos and Meek (1974) has introduced a relative stiffness term, σ , which represents the ratio of structure-to-soil stiffnesses, as presented in Equation (3 – 1). It was concluded that i) σ (ratio of structure-to-soil stiffness), b) the ratio of the structure height to foundation radius (width) and c) the interaction of the fixed-base natural frequency of the structure to the frequency regions of the design spectrum were the critical factors controlling SSEI. By Veletsos and Meek, it was concluded that for σ values in the range of 3 to 20, soil-structure interaction becomes critical.

$$\sigma = \frac{V_{s,\text{final}} \times T_{str}}{h_{effective}}$$
(3 - 1)

Rainer (1975) has presented a simplified method of analysis for the determination of dynamic properties of single-story structures founded on flexible foundations. He applied the general equations for natural frequency, mode shapes, and modal damping to structures founded on an elastic half-space and on piles. The results of his parametric studies, including the effects of hysteretic soil material damping, are presented for these two cases. Some of his findings include that: i) the variation of the modal amplitude ratios show a rapid decrease of relative displacement and a similar increase of rocking displacement with increasing aspect and stiffness ratios. This observation points to the predominant influence that rocking has on structure-ground interaction effects of moderately slender or very slender structures founded on an elastic half-space, ii) for stiffness ratios k/G_r greater than about 2, changes in soil material damping ratios are reflected in almost identical increases in system damping ratios for structures founded on an elastic half-space.

Yoshimi and Tokimatsu (1977) have shown that as the width ratio, (B/D) of a structural system increases, the settlement ratio, (S/D) decreases. The data for 35 buildings after 1964 Niigata Earthquake is shown on Figure 3.2-1.


Figure 3.2-1. Settlement Ratio, S/D vs. Width Ratio, B/D, from 1964 Niigata Earthquake (Yoshimi and Tokimatsu, 1977)

Morris (1981) has performed series of centrifuge tests to study the seismic rocking behavior of rigid foundations on cohesionless dry soils. The experiments supported the observation that the dynamic modulus of sand is proportional to the square root of the confining pressure and the stiffness of square foundations also appears to be adequately predicted by current elastic theory. Embedment of a foundation is found to increase the rotational stiffness, but by less than elastic theory predicts, probably because the theoretical assumption of full side contact is not justified in practice. The effects of interaction between adjacent slender towers appear to be small.

Finn and Yodengrakumar (1987) performed model tests on centrifuge and verified their findings through finite element analyses. They obtained that excess pore pressure generation in soils near the external wedges are higher than the ones corresponding to free field region. They have shown that soil structure and earthquake interaction do not necessarily produce conservative responses from pore pressure (liquefaction) point of view.

Rollins and Seed (1990) performed and compiled a number of shaking table and centrifuge model tests, and mostly concluded that excess pore pressure generation is slower and lower beneath the buildings compared to free field equivalent depths.

However, some of the tests indicated that there are zones near model structures which are observed to be more susceptible to pore pressure generation than free field. Typical results of pore pressure ratio development are shown in Figure 3.2-2. As part of their study, Rollins and Seed proposed correcting the free field based estimated cyclic stress ratio for three factors: i) the presence of static shear stresses on horizontal planes, K_{α} , ii) effects of higher confining pressures, K_{σ} , iii) changes in over consolidation ratio, OCR, K_{OCR} .

Effects of static shear stresses and higher confining stresses were discussed in the preceding sections. The third correction factor, K_{OCR} is recommended to be used to correct cyclic stress ratio for over consolidation effects. The K_{OCR} values, proposed by different authors are shown in Figure 3.2-3.



Figure 3.2-2. Measured Excess Pore Pressure Ratio Development: a)Early Stage; and b) Later Stage (Yoshimi and Tokimatsu, 1977; from Rollins and Seed, 1990)



Figure 3.2-3. Over Consolidation Ratio vs. Correction Factor K_{OCR} (Rollins and Seed 1990)

According to Rollins and Seed (1990), a significant increase in the factor of safety is expected for the soils underneath the building if only the effects of vertical stress increase is taken into account with the assumption that the structure responses as a rigid block. However, this is usually not the case. Spectrum shape and the period of the structure are also the factors affecting the cyclic stress ratio values near a building. They have concluded that if spectral acceleration ratio, S_A/PGA is larger than about 2.40, the induced cyclic stress ratio value would be higher beneath the building than in free field. If it is less than about 2.40, then the cyclic stress ratio beneath the building and the potential for liquefaction due to this effect alone will be decreased. Also, the building response, i.e. the period of the structure appears to play a significant role in determining the stress conditions near a building during an earthquake. It can be expected that $3\sim4$ storey buildings (T_{str} = $0.3\sim0.6$ sec.) to be much more susceptible to liquefaction than 10~15 storey buildings. It can also be concluded that, the results of a free field analysis would appear to be overly conservative for structures of long periods ($T_{str} = 1-2$ sec), e.g.: high rise structures on medium dense sands ($D_R = 55\%$). They might also be expected to provide unconservative evaluations for some short period ($T_{str} = 0.1-0.5$ sec.), low rise

structures on loose sands to medium dense ($D_R = 35$ %) and for overcompensated structures.

Popescu and Prevost (1993) have performed centrifuge model tests and numerical simulations, however, lack of pore pressure transducers located at the edges of the model structure made it impossible to fully quantify the interaction effects on pore pressure generation. The agreement between test and numerical results was concluded to be good.

Hwang et al. (1994) performed two dimensional effective stress-based analysis of soil-structure system and reached to the following conclusion: the zone directly under the structure is less prone to liquefaction compared to zones outside the structure. In simpler terms, the zones outside the structure liquefy first, followed by the propagation of excess pore pressure to zones under the structure.

Yoshiaki et al. (1997) performed shaking table tests, using a sand box with a simple footing model, the results of which were then verified by also finite element analyses of the test setup. The conclusion derived after these tests was that the soils directly beneath the footing would be harder to liquefy than the free field soils, whereas the region near the external wedge of footing was easier to liquefy than the ones of the free field region.

Liu and Dobry (1997) performed shaking table tests with sand box containing model footings, and obtained results similar to, but more detailed, than those of Yoshiaki et al. (1997). They suggested a weak zone of liquefaction resistance locating around the line starting from the external end point of footing base and inclining 45° to the horizontal direction.

Stewart et al. (1999) presented a detailed summary of the existing literature on soil structure interaction problems and described analysis procedures and system identification techniques for evaluating inertial soil structure interaction effects on seismic structural response. Two sets of analyses, aiming to estimate period

lengthening ratio and foundation damping factors, as well as soil structure system identification procedures were presented for the evaluation of vibration parameters. A more detailed re-visit to period lengthening response is available in Chapter 5.

After having performed a series of direct shear tests to study the mechanical characteristics of soil-structure interface with a charged-couple-device camera to observe the sand particle movements near the interface, Hu and Pu (2004) concluded that elastic perfectly plastic failure mode along the smooth interface and strain localization in a rough interface were accompanied with strong strain-softening and bulk dilatancy. They proposed a damage constitutive model with ten parameters to describe the behavior of the rough interface. These ten parameters include relative roughness, relative density, maximum volumetric strain, ultimate friction angle, tangential elastic stiffness, normal elastic stiffness, critical friction angle, volume dilatancy coefficient and two curve fitting coefficients. Then, they verified their hypotheses with simple shear, direct shear tests and finite element modeling. They all give good agreement with the proposed methodology.

Travasarou et al. (2006) have performed 2-D soil-structure interaction analyses for two representative buildings in Adapazari, Turkey and showed that for stiff structures on shallow foundations, seismic demand was considerably higher than free field adjacent to the perimeter of the structure and conversely for typical static building loads, the demand was reduced by 50% directly underneath the foundation of the structure.

Chen and Shi (2006) presented a simplified model for simulating unbounded soil in the vertical vibration problems of surface foundations using easily obtainable parameters such as equivalent stiffness factor, equivalent damping factor, equivalent mass factor, shear modulus of soil, soil density, shear-wave velocity of soil, characteristic length, i.e., the radius of a circular foundation, vertical static stiffness for the foundation-soil system, dynamic magnification factor, dynamic dissipated energy factor for the foundation soil system, dimensionless frequency in and dimensionless mass ratio.

Smith-Bardo and Bobet (2007) started a laboratory testing program to study the settlement and rotation response of rigid square footings under combined axial load and moment. A total of 17 tests were performed on testing boxes filled with fine and well-graded gravel in which the size of the footing, footing embedment, axial load, and load eccentricity were changed. The analytical model proposed was based on normalized response as an input, and it was calibrated to account for the change in soil stiffness with confinement. The formulation captures the inherent nonlinear deformations of the soil with load and the coupled nature of settlements and rotations of footings under axial load and moment. The test results showed that the foundation models exhibited a very ductile response with rotation capacities in excess of 5% ($\theta_u \leq 0.05$ radians) and settlements close to one-tenth the plate size. Thus, indicating that under strong ground motions a great potential for energy dissipation may be anticipated due to plastification of the soil in buildings with vulnerable foundations.

More recently, *Tileylioglu et al. (2008)* benefited from a model test structure to evaluate inertial soil-structure interaction effects. The test structure consists of braced or unbraced steel columns of 4.06 m height supporting a 4m×4m×0.4m reinforced concrete roof slab and resting on a 4m×4m×0.5m reinforced concrete foundation with no embedment. The soil profile beneath the foundation has an average shear wave velocity of 200 m/s in the upper 15 m. They have concluded that observed levels of SSI are reasonably well predicted by the available theoretical models although the level of shaking was not high (small strain conditions).

In summary, although there is only limited number of studies available and small scale model tests are not sufficient to obtain rigorous quantitative data, it can be concluded from earlier studies that, three zones having with different liquefaction potentials exist around a building foundation on loose sandy soil sites. They can be listed as high liquefaction potential for the zone around the edges of the structure, medium liquefaction potential for the free field zone and low liquefaction potential for the zone in the middle zone just beneath the building.

In addition to these studies, there exist a number of studies in the literature regarding the sliding and rocking response of structures, which are found to be useful to discuss. Sliding occurs if the resistance of soil to compression is large in comparison with the resistance to shear, then displacements of the foundation under the action of horizontal forces will occur mainly in the direction of the action of horizontal exciting forces. By definition, rocking is the generation of infinite vertical stress under the edge of the footing of the rigid foundation on an elastic half-space. Soils cannot sustain these stresses; therefore a soil support is not as stiff as the ideal elastic medium having the same elastic shear modulus. Thus, the actual maximum amplitude of rotation due to rocking of foundation will be somewhat higher and the frequency at this maximum amplitude will be lower than the pre-assumed values. The rocking vibrations mostly occur in high foundations under unbalanced horizontal components of exciting forces and exciting moments. Rocking is usually observed when foundation depths are shallow. However, when the foundation is embedded, the soil reacts not only on the horizontal foundation base area but also on the foundation side walls which results in a beneficial soil-inertia effect.

Yang et al. (1990) considered the response of multi degree of freedom structures with sliding supports subjected to harmonic and earthquake excitations and demonstrated that the higher modes of vibration can be neglected in the evaluation of base shear which means that good results can be obtained by treating the superstructures as single degree of freedom systems.

Mostaghel et al. (1983) considered the problem of response of a single degree of freedom structure supported on a sliding foundation and subjected to harmonic support motions. In their study, non-linear governing equations of motion are derived and it turns out that these equations are linear in each sliding and non-sliding phase and can be solved in closed forms in each phase.

The other aspect of shallow foundations that should be concerned is rocking which is introduced first by Housner (1963). In his study, an analysis is performed for the rocking motion of structures of inverted pendulum type and it is shown that there is a scale effect which makes tall slender structures more stable against overturning than might have been expected, and, therefore, the survival of such structures during earthquakes is not surprising.

Aslam et al. (1980) have examined the response of rigid block subjected to horizontal and vertical ground acceleration, with the option of elastic tie-down rods with a zero sliding assumption.

Various failure patterns have also been analyzed by Ishiyama (1982). A study on the dynamic behavior of a rocking rigid block supported by a flexible foundation which permits up-lift has been performed by Psycharis and Jennings (1983). Spanos and Koh (1984) have investigated the rocking response of a rigid block subjected to harmonic ground motion; assuming there is no sliding and the linear and nonlinear equations of motion have been solved numerically assuming zero initial conditions to identify likely steady-state patterns of response. Allen et al. (1986) have studied the dynamic behavior of an assembly of two-dimensional rigid prisms. Besides, the rocking response has been investigated both analytically and experimentally by Tso and Wong (1989). Hogan (1990) used the model, analyses and response of Spanos and Koh (1984) and performed a complete investigation of the existence and stability of single-impact sub-harmonic responses (1, n) (with $n \ge 1$), as a function of the restitution coefficient β . Psycharis (1990) analyzed the dynamic behavior of twoblock systems. Sinopoli (1987) has introduced a unilateral constraint to solve the impact problem using kinematic approach. The influence of nonlinearities associated with impact on the behavior of free-standing rigid objects subjected to horizontal base excitations has been studied by Yim and Lin (1991). Allen and Duan (1995) have examined the reliability of linearizing the equations of motion of rocking blocks. The criteria for initiation of slide, rock, and slide-rock rigid-body modes have been presented by Shenton (1996). The rocking response of free-standing blocks under

cyclonical pulses has been examined by Zhang and Makris (2001). Furthermore, Makris and Zhang (2001) have studied the rocking response and the overturning of anchored blocks under pulse type motions. Kim et al. (2001) have investigated experimentally the vibration properties of a rigid body placed on sand ground surface. Koh et al. (1986) have studied the behavior of a rigid block rocking on a flexible foundation. Modulated white noise has been used as a model of horizontal acceleration of the foundation. The statistics of the rocking response have been found by an analytical procedure which involves a combination of static condensation and stochastic linearization. Koh and Spanos (1986) have also presented an analysis of block random rocking.

In Spanos et al. (2001) the dynamic behavior of structures of two stacked rigid blocks subjected to ground excitation has been examined. They focused on the dynamic behavior of structures consisting of two rigid blocks; one serving as a base and another one on top of the base. Assuming rigid foundation, large friction to prevent sliding, and point contact during a perfectly plastic impact, the only possible response mechanism under base excitation is rocking about the corners of the blocks. It also presents a derivation of the exact (nonlinear) equations of motion for the system considered undergoing base excitation and a treatment of the impact problem by deriving expressions for the post-impact angular velocities and contains numerical results from the development and use of computer program for determining free vibration and seismic response of the system.

3.3 AN OVERVIEW ON SOIL STRUCTURE INTERACTION FROM LIQUEFACTION POINT OF VIEW

As is evident by the presented literature, the soil-structure interaction is a rather complex subject and there is not, yet, a consensus on how to accurately quantify the effects of the presence of structures on foundation soils subjected to both static and cyclic loading conditions. As summarized in Figure 3.3.1, the input parameters of



Figure 3.3-1. Summary of the elements of soil-structure-interaction

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this interaction can be listed under two main headings: the interaction input parameters corresponding to i) static and ii) dynamic states.

3.1.1. Static State Input Parameters

As briefly introduced in Chapter 1, presence of foundation loads complicates the static stress state. Compared to free field soil site conditions, depending on the type and magnitude of loading, major and minor principal stress directions start to deviate from vertical and horizontal directions, respectively. The static stress field is governed by the weight and geometry of the structure, relative rigidity of the foundation elements, and finally foundation soil type and state. Additionally, if one remembers that the dilatational response of soils is suppressed by the increase in mean effective stresses (e.g.: Bolton, 1986), application of a foundation load can simply affect the pore pressure response during both static and cyclic loading. Thus, for a rigorous assessment of cyclic response, it is necessary to accurately estimate the initial static stress field developed within foundation soils. Luckily, under static conditions, assessing stress state, including vertical, horizontal and shear stresses, is rather simple and elastic solutions (e.g.: Boussinesq, 1885, Fadum, 1948, Newmark, 1942, Love, 1929) can be conveniently used for this purpose.

3.1.2. Dynamic State Input Parameters

In addition to the static ones, cyclically-induced shear stresses are applied on foundation soils due to inertial interaction of the superstructure with the foundation soils and earthquake. In simpler terms, structures subjected to cyclic loading exhibit a sliding and/or rocking response, which also increase the shear demand on foundation soils. Similarly, as discussed earlier, presence of a structural foundation element changes the impedance contrast compared to free field conditions, thus propagation of shear waves is interfered, which is commonly referred as kinematic interaction.

Addressing the effects of the different static stress states in liquefaction initiation response, series of corrections, formerly known as K_{α} and K_{σ} , were proposed to the original simplified procedure. In the literature, there exist contradicting arguments regarding if and how the presence of an overlying structure and foundation affects liquefaction triggering potential and how these corrections should be applied. Thus, within the confines of this thesis, it is intended to resolve this controversial, yet important issue.

As summarized in Figure 3.3-1 assessing liquefaction triggering potential beneath structures is a complicated issue and requires the determination of a number of input parameters. The proposed assessment steps are summarized in Figure 3.3-2.



Figure 3.3-2. Steps in liquefaction prediction beneath structures

The main difficulty in calculating the liquefaction potential beneath a structure is the estimation of shear stresses at the location of interest. It should be noted that the shear stresses are induced not only from inertia of the soil and structure, but also their complex interaction with each other and the earthquake excitation. Even if the shear stresses induced by the inertia of the soil and structural masses are determined, how to add them up becomes another critical issue due to their possible out of phase nature. Cyclically induced shear stresses need to be normalized by the vertical effective at the point of interest, leading to "raw" CSR value. To estimate the equivalent CSR corresponding to zero initial (static) shear stress and vertical effective stress of 100 kPa, correction factors K_{α} and K_{σ} need to be applied on this "raw" CSR value. The details of these calculation steps will be discussed next.

3.3.1.1 Cyclically-induced Shear Stress of a Soil Mass, τ_{soil}

Cyclically-induced shear stresses in a soil medium can be estimated by using the simplified procedure of Seed and Idriss (1971) as discussed in detail in Chapter 2, Section 2.3.2.1. As a reminder, shear stresses can be approximated as:

$$\tau_{soil} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \gamma_n \cdot h \cdot r_d \tag{3-2}$$

where

 a_{max} : maximum acceleration at the ground surface (g)

 γ_n : unit weight of soil (kN/m³)

h : depth from the ground surface (m)

r_d : non-linear mass participation factor

In this equation, the factor 0.65 is used to convert the peak cyclic shear stress ratio to a cyclic stress ratio that is representative of the most significant cycles over the full

duration of loading. The parameter r_d is a stress reduction factor that accounts for the flexibility of the soil column and has been discussed thoroughly in Section 2.3.2.1.3.

3.3.1.2 Cyclically-induced Base Shear due to Overlying Structure, τ_{base}

Contrary to the "simplified procedure" alternative, there are number of simplified tools for the determination of base shear (τ_{base}) due to inertia of the overlying structure. They are specified in various design codes. Basically, horizontal base shear is calculated by the multiplication of the spectral acceleration with the mass of the structure and correcting this value with a coefficient due to different modal participation of the structural masses. This phenomenon is known as equivalent lateral force procedure. In this section, base shear formulations from NEHRP, Eurocode 8, International Building Code 2003 and Turkish Earthquake Code (2007) will be presented.

3.3.1.2.1 NEHRP Recommended Provisions for New Buildings and Other Structures

In NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1-Provisions, it is recommended that for a fixed base structure, the lateral forces applied in the horizontal direction shall sum to a total seismic base shear given as

$$\mathbf{V} = \mathbf{C}_{s} \cdot \mathbf{W} \tag{3-3}$$

where

C_s : Seismic response coefficient

W : Total dead and live load applicable portions of other loads

The seismic response coefficient C_s is determined using:

$$0.044 \le C_s = \frac{S_{DS}}{R/I} \le \frac{S_{D1}}{T(R/I)}$$

where

S_{DS} : design spectral acceleration in the short period range

R : response modification factor (presented in .Table 5.2.2 in NEHRP)

I : occupancy importance factor

 S_{D1} : design spectral response at a period of 1.0 second

T : fundamental period of the structure

The fundamental period of structures are estimated as:

$$T_a = C_r \cdot h_n^x \tag{3-4}$$

where *h* is the height of the structure in feet or meters and values for C_r and *x* are specified in Table 5.4.2.1 of NEHRP. For a reinforced concrete structure, $C_r = 0.0466$ and x = 0.9 in SI units. A more common method for determining structural period is also defined in NEHRP which is equal to number of storey divided by 10, i.e.:

$$T_a = 0.1 \cdot N \tag{3-5}$$

where N is the number of storey. The other method for determining the fixed base periods for masonry structures are

$$T_{a} = \frac{0.0019}{\sqrt{C_{w}}} \cdot h_{n} \qquad (3-6)$$

and the same form of equation for structures with concrete shear walls are:

$$T_a = \frac{0.0062}{\sqrt{C_w}} \cdot h_n \tag{3-7}$$

in which

$$C_{w} = \frac{100}{A_{B}} \sum_{i=1}^{n} \left(\frac{h_{n}}{h_{i}}\right) \cdot \frac{A_{i}}{\left[1 + 0.83\left(\frac{h_{n}}{D}\right)\right]}$$
(3 - 8)

where

 A_B : Base area

- h_n : height of building
- n : number of shear walls
- h_i : height of shear wall
- D : length of shear wall
- A_i : area of shear wall

These shear forces found by equivalent lateral force procedure is distributed vertically according to the formula below:

$$F_{x} = C_{yx} \cdot V \tag{3-9}$$

3.3.1.2.2 Eurocode 8

Eurocode 8 also suggests lateral force method of analysis in calculating base shear in horizontal directions. The seismic base shear force F_b , for each horizontal direction in which the building is analyzed, shall be determined by using the following expression:

$$F_{b} = S_{d}(T_{1}) \cdot m \cdot \lambda \qquad (3 - 10)$$

where

 $S_d(T_1)$: ordinate of the design spectrum at period T_1

 T_1 : fundamental period of vibration of the building for lateral motion in the direction considered

m : total mass of the building, above the foundation or above the top of a rigid basement

 λ : Correction factor, $\lambda = 0.85$ if $T_1 \le 2T_c$ and the building has more than two storey, or $\lambda = 1.0$ otherwise

Fundamental vibration period T_1 of the buildings, expressions based on methods of structural dynamics may be used. However, for buildings with heights up to 40 m, the value of T_1 may be approximated by:

$$T_1 = C_t \cdot H^{3/4}$$
 (3 - 11)

where

- C_t : 0.085 for moment resistant space steel frames, 0.07 for moment resistance space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures;
- H : height of the building, in m, from foundation or from the top of a rigid basement.

An alternative for structures with concrete or masonry shear walls, C_t may be taken as being:

$$C_{t} = \frac{0.075}{\sqrt{A_{c}}}$$
(3 - 12)

where

$$A_{c} = \sum \left[A_{i} \cdot (0.2 + (l_{wi} / H))^{2} \right]$$
 (3 - 13)

and

 A_c : total area of shear walls in the first storey, in m^2

 A_i : effective cross-sectional area of the shear wall *i* in the first storey of the building, in m²

 l_{wi} : length of the shear wall *i* in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0.9

Another alternative for estimation of T_1 is:

$$T_1 = 2 \cdot \sqrt{d} \tag{3-14}$$

where

d : lateral elastic displacement of the top of the building, in m, due to gravity loads applied in the horizontal direction.

This total horizontal seismic base shear force can be distributed in vertical direction through the building height by using the methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

3.3.1.2.3 International Building Code, 2003

In International Building Code, IBC-2003, provisions given in ASCE 7 is suggested. In this procedure, a simplified analysis for seismic design of buildings is proposed. The seismic base shear in a given direction shall be determined in accordance with the following equation:

$$V = \frac{1.2 \cdot S_{DS}}{R} W$$
(3 - 15)

where

V : seismic base shear,

S_{DS} : the design elastic response acceleration at short period,

R : the response modification factor from Table 1617.6.2 from at IBC-2003,

W : the effective seismic weight of the structure including dead and live loads discussed in detail in IBC-2003.

This horizontal load is distributed at each level by using:

$$F_x = \frac{1.2 \cdot S_{DS}}{R} w_s \qquad (3 - 16)$$

where

 w_s : the portion of the effective seismic weight of the structure, W, at Level x

3.3.1.2.4 Turkish Earthquake Code, TEC

In the Turkish design code for seismic design, TEC, the base shear force is taken equal to the total equivalent earthquake load, V_t , and is calculated as given below:

$$V_{t} = \frac{WA(T_{1})}{R_{a}(T_{1})} \ge 0.10A_{0}IW$$
 (3 - 17)

where

 T_1 : natural period of the structure, in 1st mode calculated as:

$$T_1 \cong T_{1a} = C_t \cdot H^{3/4}$$
 (3 - 18)

W : weight of the structure

 A_0 : effective ground acceleration coefficient, presented in Table 3.3-1.

I : structure importance factor defined in Table 6.3 in TEC.

R_a : load dissipation factor which is calculated by equations below:

$$\begin{array}{l} R_{a}(T) = 1.5 + (R - 1.5) \cdot T / T_{A} & (0 \le T \le T_{a}) \\ R_{a}(T) = R & (T > T_{a}) \end{array}$$
(3 - 19)

For the weight of the structure, combinations of dead and live loads will be utilized as described in the code.

Earthquake Zone	A ₀
1	0.40
2	0.30
3	0.20
4	0.10

Table 3.3-1. Effective ground acceleration coefficient, A₀

3.3.1.3 K_{σ} and K_{α} Correction Factors for Assessing the Effects of Static Stress State on Liquefaction Triggering

Simplified solution proposed by Seed and Idriss (1971) and the corresponding cyclic resistance ratio charts are valid for free field level site conditions, i.e. where initial (static) shear stresses are zero and for atmospheric pressure of 1 atm (= 100 kPa). However, this is always not the situation. Initial shear stresses are present in inclined sites and more importantly existence of structures on even level sites causes static shear stress too. To take these initial shear stresses (τ_0) into account, a correction factor, K_{α} was initially proposed by Seed (1983). In the same study, Seed has also proposed a correction factor for the overburden stress, namely K_{σ} which then became the research topic for various studies.

Cyclic resistance ratio values for the current state of stresses are found by multiplying cyclic resistance ratio at reference state, i.e. $\tau_0 = 0$ and $\sigma_{v'} = 1$ atm, by K_{α} and K_{σ} correction factors, i.e.:

$$CRR = CRR_1 \times K_{\sigma} \times K_{\alpha} \qquad (3-20)$$

where

CRR : the cyclic resistance ratio (τ_{av}/σ_{v0}) at the current stress state (σ_{v0}) and τ_{s}

CRR₁ : the cyclic resistance ratio at the reference state (SPT correlation, $\tau_s = 0$ and $\sigma_{v}' = 1$ atm)

 K_{σ} : correction factor for the level of vertical effective confining stress, σ_{v0}

 K_{α} : correction factor for the level of static horizontal shear stress, $\left(\alpha = \frac{\tau_s}{\sigma_v}\right)$

 σ_{v0} : initial vertical effective confining stress

 τ_s : static shear stress on the horizontal plane

Although there is a consensus regarding how the extra overburden and initial shear stresses affect the liquefaction potential of foundation soils, the numerical values do not converge as mentioned earlier. Details of K_{α} and K_{σ} are presented in detail in the following sections.

3.3.1.3.1 K_{α} Correction

The undrained cycling response of soils is affected by the presence of initial static shear stresses. To assess this effect, K_{α} correction factor was introduced by Seed (1983) to extend SPT and CPT correlations to sloping ground conditions where shear stresses exist prior to earthquake shaking. Another area where K_{α} correction factor should be used is the foundation soils, which is the main purpose of considering this issue in this study. The presence of structures causes shear stresses to develop beneath the foundations. A dimensionless parameter, α , defined as the ratio of initial (static) shear stresses to the static vertical effective stress $\left(\alpha = \frac{\tau_s}{\sigma_v}\right)$ is used for the assessment of these initial (static) shear stresses. Studies on this subject have shown that K_{α} correction depends on relative density, confining stress, failure criteria and somewhat on the laboratory test device. Latest studies, which will be discussed in the following paragraphs, show that presence of initial shear stresses increases cyclic resistance ratio (CRR) of dense sands ($D_R > 50\%$) and decreases cyclic resistance ratio for looser sands ($D_R < 45\%$).

The studies of Seed (1983) mostly focus on soils with relative densities higher than 50 % and concluded that, K_{α} correction factors lead to a very large increase in cyclic resistance of soils.

Research studies by Yoshimi and Oh-Oka (1975), Vaid and Finn (1979), Tatsuoka et al. (1982), Vaid and Chern (1983, 1985), Szerdy (1985) and Jong and Seed (1988) produced very similar conclusions. They have concluded that presence of static shear stresses increases the cyclic resistance (K_{α} >1) for moderately dense or dense sandy soils for confining stresses less than about 3 tsf and decrease the cyclic resistance ratio of loose, sandy soils (K_{α} <1).

Seed and Harder (1990) developed a set of K_{α} correction factors for a range of relative densities consistent with the previous findings.

Boulanger and Seed (1995) verified many of the results previously developed by other researchers. Boulanger (2003a) has proposed a practical guideline for describing the combined effects of relative density and confining stress on K_{α} by relating Bolton's (1986) relative state parameter index, ξ_{R} . Expression derived in Boulanger (2003a) is:

$$K_{\alpha} = a + b \cdot \exp\left(\frac{-\xi_{R}}{c}\right)$$
(3 - 21)

$$\xi_{\rm R} = \frac{1}{Q - \ln\left(\frac{100 {\rm p'}}{{\rm P}_{\rm a}}\right)} - {\rm D}_{\rm R}$$
 (3 - 22)

$$a = 1267 + 636\alpha^{2} - 634 \cdot \exp(\alpha) - 632 \cdot \exp(-\alpha)$$
 (3 - 23)

$$b = \exp\left[-1.11 + 12.3\alpha^{2} + 1.31 \cdot \ln(\alpha + 0.0001)\right]$$
(3 - 24)

$$c = 0.138 + 0.126\alpha + 2.52\alpha^3 \tag{3-25}$$

In these equations;

 D_R : relative density

p' : mean effective normal stress;

Q : empirical constant which determines the value of p' at which dilatancy is suppressed and depends on the grain type (Q \approx 10 for quartz and feldspar, 8 for limestone, 7 for anthracite and 5.5 for chalk);

P_a : atmospheric pressure

and the values of α are limited to $\alpha \le 0.35$ and $-0.6 \le \xi \le 0$.

Figure 3.3-3 summarizes the recommendations of NCEER (1997) group. As the figure implies, K_{α} correction factor varies significantly in the range of 0.3 to 1.7. The trend can be summarized in a way that, existence of initial shear stresses increase cyclic resistance ratio (CRR) of dense sands (D_R >50%) and decrease cyclic resistance ratio for looser sands (D_R <45%).



Figure 3.3-3. K_{α} correction factors recommended by NCEER (1997)

Figure 3.3-4 presents the variation of α values in two dimensions along the midline of typical structure estimated by three dimensional static analyses. In this figure

x/B = 0 represents the center of the structure and x/B = 0.5 represents the edges of the structure, as shown in the figure with vertical dashed line. As the figure implies the α values reach their maximum values near the corners of the structures as expected.

Based upon currently available K_{α} corrections, it can be concluded that from only K_{α} point of view, due to the presence of an overlying structure, decrease in liquefaction triggering resistance of loose foundation soils is expected especially near the foundation edges.



Figure 3.3-4. Variation of " α " along the width of the structure at z/B = 0 and y/L = 0

Figure 3.3-5 shows K_{α} fields developed along the center line of a 4 story residential structure founded on a mat underlain by a sand layer with relative densities of 30% and 70%, respectively. The K_{α} correction factors in these charts have been determined by using the average values of the NCEER (1997) recommendations. As can be interpreted from Figure 3.3-5, K_{α} effects increase seismic demand (i.e. CSR) for "loose" soils and decreases seismic demand for "dense" soils.



Figure 3.3-5. K_{α} along the width of the structure, for (a)D_R = 30%, (b) D_R = 70% z/B = 0 and y/L = 0

3.3.1.3.2 K_{σ} Correction

The effects of the confining pressure on penetration resistance of clean sands have been acknowledged as part of the simplified procedure. The available SPT data used for the development of liquefaction triggering analyses procedures are, however, limited to vertical stress values less than about 550 kPa and the extension of C_N factor (normalizing factor for SPT N values to an effective overburden stress of 1 atm (100 kPa)) to larger values requires extrapolation of purely empirical expressions. Besides, results of cyclic laboratory tests showed that liquefaction resistance of a soil increase with confining stresses. However, this increase is not linear and it decreases with increased normal stress. To incorporate this non-linearity, Seed (1983) recommended the use of the correction factor K_{σ} for overburden pressures greater than 100 kPa. The K_{σ} correction factor developed by Seed (1983) was obtained by normalizing CSR values of isotropically consolidated cyclic triaxial compression tests to cyclic resistance ratio (CRR) values associated with an effective confining pressure of 100 kPa (1 tsf). For confining pressures greater than 100 kPa, the K_{σ} correction factor is less than unity and decreases with increasing pressure. Using the suggested value, the cyclic resistance at about 800 kPa becomes only the 40 to 60 percent of the cyclic resistance at 100 kPa.

Seed and Harder (1990) analyzed additional data and suggested a curve for K_{σ} values. This curve results in generally lower values than before. However, it should be kept in mind that there is a considerable scatter in Seed and Harder (1990) K_{σ} database and they have performed their tests on both "undisturbed" and reconstituted samples.

Vaid et al. (1985) and Vaid and Thomas (1994) performed constant-volume cyclic simple shear tests on clean sands and found a very little decrease in K_{σ} values.

The experimental data in Figure 3.3-6 (Vaid and Sivathayalan, 1996) shows that K_{σ} is dependent on D_R , with K_{σ} values at $\sigma'_v / P_a > 1$ decreases with increasing D_R and that K_{σ} is different for simple shear versus triaxial loading conditions.

Pillai and Muhunthan (2001) observed that CRR of clean sand was approximately constant for a given value of state parameter index (ξ_R) and K_σ depends on the critical state parameters of the sand. Boulanger (2003b) noted that the practical difficulties in determining critical state properties for most site-specific projects, and proposed a relative state parameter index (ξ_R) derived from the relative dilatancy index (I_{RD}) of Bolton (1986).

Boulanger (2003b) re-evaluated the effect of overburden stress on liquefaction triggering using a theoretical framework and introduced a ξ_R -based approach reducing conservatism imposed at high overburden stresses by some current C_N and K_{σ} relations. It is also shown that, the appropriate choice of D_R -independent C_N and

 K_{σ} relations (without need of D_R or ξ_R) can approximate the effect of σ'_v on predicted CRR.



Figure 3.3-6. Comparison of K_{σ} relations with data from reconstituted Fraser delta sand specimens (Vaid and Sivathayalan 1996) and various field samples (Seed and Harder 1990) (from Boulanger 2003b)

Boulanger and Idriss (2004) summarized the results for K_{σ} relations as expressed in equations below:

$$K_{\sigma} = 1 - C_{\sigma} \ln \left(\frac{\sigma'_{v0}}{P_a}\right) \le 1.0$$
(3 - 26a)

$$C_{\sigma} = \frac{1}{18.9 - 17.3D_{R}} \le 0.3$$
 (3 - 26b)

Boulanger and Idriss (2004) also expressed the coefficient C_{σ} in terms of $(N_1)_{60}$ or q_{c1N} as:

$$C_{\sigma} = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}}$$
(3 - 27)

$$C_{\sigma} = \frac{1}{37.3 - 8.27(q_{c1N})^{0.264}}$$
(3 - 28)

with $(N_1)_{60}$ and q_{c1N} values are limited to 37 and 211 respectively. The resulting K_{σ} curves, calculated using Equations (3 - 26) to (3 - 28) are shown in Figure 3.3-7 for a range of $(N_1)_{60}$ and q_{c1N} compared with the previous studies.



Figure 3.3-7. Comparison of derived K_{σ} relations to those recommended by Hynes and Olsen (from Boulanger and Idriss 2006)

3.4 SUMMARY AND CONCLUDING REMARKS

A detailed overview of the existing studies on SSEI from liquefaction point of view is presented in this chapter. The effects of the structures from the liquefaction triggering point of view are discussed with a detailed literature survey. On the basis of the summarized literature, it can be concluded that the structural-induced liquefaction triggering problem is a difficult and a complex issue and has not been satisfactorily addressed yet. The major issue is defined as the estimation of cyclically-induced shear stresses developed in soil layers and determination of shear stresses beneath the structures. The prediction of correction factors for initial static shear stresses (K_{α}) and extra overburden stresses (K_{σ}) are then discussed in detail with illustrations from various design codes and practice.

CHAPTER 4

NUMERICAL MODELING OF SSEI FROM LIQUEFACTION TRIGGERING POINT OF VIEW

4.1 INTRODUCTION

For the purpose of assessing soil-structure-earthquake interaction from liquefaction triggering point of view, series of 1-D and 3-D numerical simulations were performed. As the basis of these simulations, four different residential structures founded on four different generic soils profiles were assessed under static and dynamic (i.e.: shaken by four earthquake records) loading conditions. Within the scope of this chapter, the details of these numerical simulations along with post-processing of the results are presented.

4.2 NUMERICAL ANALYSES PROCEDURE

As mentioned previously, the basis of the proposed simplified SSEI assessment methodology rests on three dimensional (3-D) static and dynamic analyses of the SSEI problem from liquefaction triggering of foundation soils point of view, the results of which were further verified and calibrated with the field performance and centrifuge test results. For the purpose, three sets of numerical analyses were performed: i) dynamic free field analyses, ii) static analyses with the existing structure and iii) dynamic analyses of the soil-structure-earthquake interacting system.

Free field dynamic analyses, referred to in (i), were performed by using both 1-D and 3-D models for verification purposes. In simpler terms, due to the fact that most of the conventional liquefaction triggering models chose 1-D, total stress-based equivalent linear assessment methodology (e.g.: SHAKE 91) as the basis to determine seismically-induced shear stresses, free field analyses were performed by both 1-D SHAKE 91 and 3-D FLAC until a reasonable match is achieved between both models. Global mesh window and local mesh sizes were continuously altered until the subject match is achieved. Note that in both analyses equivalent linear model is used with exactly the same model parameters. A detailed outline of the analyses scheme is presented in Figure 4.2-1.



Figure 4.2-1. Analyses performed in this study

Comparisons of the results by these two models are given in detail in Section 4.4.4. After having achieved consistent results for the free field dynamic analyses by both models, the same soil mesh and window can now be used for the SSEI analyses, but this time with the overlying structure in place. Before the discussion of these SSEI analyses, static assessment of the soil-structure system needs to be presented for the purpose of better understanding the initial static vertical effective and shear stresses, which may significantly affect the pore pressure generation response during seismic loading, formerly recognized as K_{α} and K_{σ} factors. 3-D seismic response assessment of soil-structure-earthquake system followed these static analyses. The dynamic response analyses results of the soil-structure-earthquake interacting system along with the free field dynamic analyses results enabled us to rationally compare CSR values which may lead to the final conclusion if SSEI effects are critical from liquefaction point of view for the particular case studied. The components of the numerical analyses steps are discussed next.

4.3 GENERIC SOIL PROFILES

4 sets of soil profiles, composed of 30 meter deep, homogeneous, cohesionless soil layers with shear wave velocities varying as 100, 150, 200 and 300 m/s were developed for the analyses. The optimum choice of 30 meter as the depth of the soil profile has the advantages of reducing run times compared to deeper soil profile models and being deep enough to explore the interaction effects of the overlying structure. It should be noted, however, that due to variation in shear wave velocity values, the initial periods of the soil sites are not constant and changes from 0.40 to 1.2 second. Elastic and strength parameters adopted for the analyses are tabulated in Table 4.3-1. The reasoning behind the model parameter choices are discussed below.

Static analyses were performed with elastic-perfectly plastic constitutive model with Mohr – Coulomb failure criterion. However, for the sake of consistency with the 1-D equivalent linear assessment methodology (i.e.: SHAKE 91), 3-D dynamic analyses were performed with elastic equivalent linear constitutive model without Mohr Coulomb failure criterion applied.

Soil Profile ID	Soil Classification	Unit weight, γn, (kN/m ³)	Shear Wave Velocity (m/s)*	Shear Modulus, G, (MPa)**	Poisson's Ratio, U	Internal Friction Angle, φ, (⁰)
1	Very loose	18	100	18	0.35	27
2	Loose	18	150	40.5	0.35	29
3	Medium dense	18	200	72	0.35	32
4	Dense	18	300	162	0.35	40

Table 4.3-1. Drained parameters for generic soil profiles

*Assumed to cover a range of very loose to medium dense cohesionless soils

** Estimated by the elastic assumption of $G_{\text{max}} = \rho \cdot V_s^2$

4.4 FREE FIELD DYNAMIC ANALYSES

Free field dynamic analyses were performed by two different software and numerical methods: 1-D equivalent linear (which were performed using SHAKE 91 software) and 3-D equivalent linear finite difference-based (which were performed using FLAC-3D) site response analyses. These seismic assessments of the free field soil sites (without the structural system) were performed for the purpose of enabling direct comparison and calibration of the 3-D (FLAC-3D) model with 1-D (SHAKE 91) model. The results of these analyses will be presented in Section 4.4.4.

Following sections present the details of free field dynamic analyses.

4.4.1 Choice of Input Motions Used in the Analyses

Besides a good representation of dynamic soil properties at the sites of interest, a properly performed dynamic site response analysis requires the selection of suitable input strong ground motion records for the reason that soil-structure interaction is dependent on the frequency content of the excitation waveform coinciding with the resonant frequencies of the system components. The sites evaluated in this study were shaken by four different earthquake records namely i) 1999 Kocaeli Earthquake, $M_w = 7.4$, Sakarya (SKR) record, ii) 1989 Loma Prieta Earthquake, M_w = 7.0 Santa Cruz USCS Lick Observatory Station (LP) record, iii) 1995 Kobe Earthquake, $M_w = 6.9$, Chimayo Station (CHY) record and iv) 1979 Imperial Valley Earthquake, $M_w = 6.4$, Cerro Prieta (IMP) record. Details and general characteristics of these earthquakes are described in the following sections. All of these four earthquakes were filtered to have a maximum frequency of 15 Hz. This filtering process allows us to increase the dimensions of the mesh elements used in the finite difference model which consequently results in a remarkable decrease in computational time and also preservation of minimum 95% of the power dissipated during the earthquake.

4.4.1.1 1999 Kocaeli Earthquake

On August 17, 1999, a magnitude $M_w = 7.4$ earthquake struck the Kocaeli area in the Northwest Turkey. The earthquake epicenter was located at Izmit along the strike-slip North Anatolian Fault. Details on the fault rupture mechanisms and strong motion recordings can be obtained from the Kandilli observatory at <u>http://www.koeri.boun.edu.tr/earthqk/earthqk.html</u>. The strong motion record used in these analyses (SKR) is obtained from a rock site, Sakarya Station. It is located at a closest distance of 3.1 km from the surface projection of the rupture. Maximum acceleration of the record is 0.40 g and the time step is 0.03 seconds. Acceleration time history as well as the spectral values (displacement, velocity and acceleration) of the SKR-record is presented in Figure 4.4-1. During the finite difference analyses, first 20.9 seconds of the record which corresponds to 90% energy dissipation has been taken into account.



Figure 4.4-1. Acceleration time history and displacement-velocity-acceleration response spectra for 1999 Kocaeli Earthquake, Sakarya (SKR) record

4.4.1.2 1989 Loma Prieta Earthquake

The Loma Prieta earthquake was a major earthquake that struck the San Francisco Bay Area of California on October 17, 1989 at 5:04 p.m. The earthquake lasted approximately 15 seconds and was reported to be a moment magnitude 7.0 (surfacewave magnitude 7.1). The epicenter was located in the forest of Nisene Marks State Park, in the Santa Cruz Mountains (geographical coordinates 37.04° N 121.88°W), near unincorporated Aptos and approximately 16 km (10 miles) northeast of Santa Cruz. The quake was named for the nearby Loma Prieta Peak which lies 8 km (5 miles) to the north-northeast. The record used in this study is Santa Cruz USCS Lick Observatory Station (LP) record with closest distance to surface projection of rupture of 12.5 km. Time acceleration record and the spectral values (displacement, velocity and acceleration) are presented in Figure 4.4-2. Maximum acceleration of this record is 0.36 g and the time step is 0.005 seconds.



Figure 4.4-2. Acceleration time history and displacement-velocity-acceleration response spectra for 1989 Loma Prieta Earthquake, Santa Cruz USCS Lick Observatory Station (LP)
4.4.1.3 1995 Kobe Earthquake

January 17, 1995 Hyogo-Ken Nanbu Earthquake is the most damaging earthquake that has struck Japan. The earthquake was assigned a JMA magnitude of 7.2 by the Japan Meteorological Agency (JMA). Seismological analyses indicate a seismic moment of about 3 x 10^{26} dyne- cm, corresponding to a moment magnitude of 6.9 (Kikuchi, 1995). The hypocenter of the earthquake (34.6 N, 135.0 E, focal depth =10 km, origin time 5:46:52 JST; JMA) was located about 20 km southwest of downtown Kobe. The focal mechanism of the earthquake indicates right-lateral strike-slip movement on a vertical fault striking slightly east of northeast, parallel to the strike of the mapped faults. The record used in this study is Chimayo Station (CHY) record with closest distance to surface projection of rupture of 48.7 km. Time acceleration record and the spectral values (displacement, velocity and acceleration) are presented in Figure 4.4-3. Maximum acceleration of the record is 0.11 g and the time step is 0.01seconds.



(a)



Figure 4.4-3. (a) Acceleration time history and (b) displacement-velocityacceleration response spectra for 1995 Kobe Earthquake, Chimayo Station (CHY) record

4.4.1.4 1979 Imperial Valley Earthquake

On October 15th, 1979, a moment magnitude $M_w = 6.4$ earthquake occurred at about 29 km southeast of El Centro, California, USA. The location of the earthquake was 32° 37' N, 115° 19' W. The fault was a right-lateral strike slip. Rupture length was 30 km of the Imperial fault zone, 13 km of the Brawley fault zone and 1 km of the Rico fault. The record used in this study is the Cerro Prieto Station (IMP) record with closest distance to surface projection of rupture of 26.5 km. Time acceleration record as well as the spectral values are presented in Figure 4.4-4. Maximum acceleration of this record is 0.169 g and the time step of 0.01 seconds.



Figure 4.4-4. Acceleration time history and displacement-velocity-acceleration response spectra for 1979 Imperial Valley, Cerro Prieto Station (IMP) record

4.4.2 1-D Equivalent Linear Model Input Parameters

SHAKE 91 (Idriss and Sun, 1993) is the most commonly used, thus calibrated software for the 1-D equivalent linear analysis of dynamic soil response. It is a slightly modified version of program SHAKE (Schnabel, et al., 1973), which employs an incrementally linear (equivalent linear) total stress based analysis for the response estimation of horizontally layered visco-elastic soil systems subjected to vertically propagating shear waves. An equivalent linear method is used to model nonlinear dynamic soil moduli and damping as a function of shear strain. The

hysteretic stress-strain behavior of soils under harmonic cyclic loading is represented by an equivalent modulus, G, corresponding to the secant modulus through the end points of the hysteresis loop and an equivalent damping ratio, β , corresponding to the equivalent damping.

An iterative procedure is followed to find the shear moduli and damping ratios compatible with the computed shear strains. Initial estimates of the shear strains and compatible estimates of dynamic moduli and damping ratios are provided for the first iteration. The strain dependence of the shear modulus and damping in soil layers or sub-layers is accounted for by an equivalent uniform ("effective") strain computed in that same layer or sub-layer. The ratio of equivalent uniform shear strain to the calculated maximum strain is specified as an input parameter and the value of this ratio is adopted as n = 0.65. In these studies, the ratio n was taken as a function of magnitude as $n = (M_w-1)/10$. The iteration stops when the initial estimation converges to the resulting computed strain amplitudes.

In these studies, dynamic analyses of free field soil sites have been first performed by SHAKE 91. Within this scope, four different homogeneous soil profiles of depth 30 m were developed with a rock half space below. Properties of the soils used in these soil profiles were listed in Table 4.3-1. Modulus degradation and damping curves proposed by Seed et al. (1984b) were used for the analyses. These curves are presented in Figure 4.4-5 and Figure 4.4-6 respectively. For the rock half-space degradation and damping curves proposed by Schnabel (1973) are also presented in the same figures.



Figure 4.4-5. Modulus degradation curves for cohesionless soils by Seed et al. (1984b) and for rock by Schnabel (1973)



Figure 4.4-6. Damping curves for cohesionless soils by Seed et al. (1984b) and for rock by Schnabel (1973)

An example of input files for used in SHAKE 91 analyses is presented in Appendix C. The results of the one-dimensional free field dynamic analyses will be discussed in Section 4.4.4 in comparison with the three-dimensional free field dynamic analyses.

4.4.3 3-D Finite Difference Site Response Analyses

Three dimensional (3-D) dynamic analyses were performed by the software FLAC-3D. FLAC-3D is a three-dimensional explicit finite-difference program used for engineering mechanics computation purposes. As defined in FLAC-3D User's Manual (2005), the basis for this program is the well-established numerical formulation used by two-dimensional program, FLAC-2D. FLAC-3D extends the analysis capability of FLAC-2D into three dimensions, simulating the behavior of three-dimensional structures built of soil, rock or other materials that undergo plastic flow when their yield limits are reached. Materials are represented by polyhedral elements within a three-dimensional grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to applied forces or boundary restraints. The material can yield and flow and the grid can deform (in large-strain mode) and move with the material that is represented. The dynamic analysis option of FLAC-3D permits three-dimensional, fully dynamic analysis. The calculation is based on the explicit finite difference scheme to solve the full equations of motion, using lumped grid point masses derived from the real density of surrounding zones (rather than fictitious masses used for static solution). This formulation can be coupled to the structural element model, thus permitting analysis of soil-structure interaction brought about by ground shaking. The dynamic option extends FLAC-3D's analysis capability to a wide range of dynamic problems in disciplines such as earthquake engineering, seismology and so on. One of the important issues in 3-D analyses is choosing a correct form of a mesh and boundary conditions. Because when a seismic wave is introduced into the system, one should be sure that it will go through the soil body without distortion. Next sections will give explanations about mesh generation

and selection of material properties in 3-D analyses. An example of a FLAC-3D input file is presented in Appendix C.

4.4.3.1 Mesh generation and boundary conditions

One of the useful aspects of FLAC-3D modeling is the mesh generation, especially in dynamic analyses. Important aspects of dynamic modeling in *FLAC-3D* are i) dynamic loading and boundary conditions; ii) mechanical damping; and iii) wave transmission through the model.

Dynamic loading and boundary conditions used in FLAC-3D are summarized in Figure 4.4-7 modified from FLAC-3D User's Manual (2005). The dynamic input can be applied in one of the following ways i) an acceleration history; ii) a velocity history; iii) a stress (or pressure) history; or iv) a force history with the APPLY command. In this study the earthquake excitation is applied to the numerical model by an acceleration time history from the base of the model.



a) Flexible base

b) Rigid base



In static analyses, fixed or elastic boundaries (e.g., represented by boundary-element techniques) can be realistically placed at some distance from the region of interest. In dynamic problems, however, such boundary conditions cause the reflection of outward propagating waves back into the model and do not allow the necessary energy radiation. The use of a larger model can minimize this problem, since material damping will absorb most of the energy in the waves reflected from distant boundaries. However, this solution leads to a large computational burden. The alternative is to use quiet (or absorbing) boundaries. Although, several formulations have been proposed in the literature, the viscous boundary developed by Lysmer and Kuhlemeyer (1969) is used in *FLAC-3D*. In this study, free field boundaries were used which are placed at sufficient distances to minimize wave reflections and achieve free field conditions. This model is presented schematically in Figure 4.4-8.



Figure 4.4-8. Model for seismic analysis of surface and free field mesh in FLAC-3D (reproduced from FLAC-3D User's Manual)

Free field mesh used in dynamic analyses in these studies is also presented in Figure 4.4-9. The separated parts seen in the four edges and four corners of the mesh are the free field boundaries as illustrated in the figure.

The element dimensions of the mesh have been determined keeping in mind that numerical distortion of the propagating wave can occur in a dynamic analysis as a function of the modeling conditions. Both the frequency content of the input wave and the wave-speed characteristics of the system will affect the numerical accuracy of wave transmission. Kuhlemeyer and Lysmer (1973) showed that for accurate representation of wave transmission through a model, the spatial element size, l, must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave— i.e.,

$$\Delta l \le \frac{\lambda}{10} \tag{4-1}$$

where λ is the wavelength associated with the highest frequency component that contains appreciable energy. Maximum element size was selected to be 1 m which satisfies the condition stated in Equation (4 – 1).

4.4.3.2 Material properties

In the analyses, elastic equivalent linear model has been executed. Elastic parameters summarized in Table 4.3-1 were also used in FLAC-3D analyses. In addition to these elastic parameters, the equivalent linear model requires the estimation of the modulus degradation and damping curves. FLAC-3D has the capability of applying equivalent linear model in Version 3.0 and gives the permission of use of modulus degradation and damping. The same curves, presented in Figure 4.4-5 and Figure 4.4-6, were used in the FLAC-3D input files for dynamic analyses.



Figure 4.4-9. Model for free field seismic analysis of generic soil sites in FLAC-3D in these studies

4.4.4 Comparison of 1-D and 3-D Analyses Results

Motivation for assessing 1-D free field site response analyses was to verify the results of 3-D analyses. For this purpose, maximum acceleration, maximum shear stress, maximum shear strain and elastic spectrum obtained as a result of these two types of analyses are compared. These comparison curves are presented in Appendix B. For illustration purposes, one set from these curves are plotted in Figure 4.4-10, Figure 4.4-11, Figure 4.4-12 and Figure 4.4-13 respectively. The solid lines in these figures represent the results from 3-D analyses, and the open circles are the results after 1-D analyses. As can be seen from these graphs, results obtained from these two sets are close to each other. The principal reason of the small variations is the

fact that in one dimensional analysis, waves can propagate in only one (vertical) direction whereas in three dimensional analyses horizontal propagation is also allowed. The other reasons for these discrepancies may be the boundary conditions applied to the 3-D models and/or numerical differences in these two totally different types of analyses methods.



Figure 4.4-10. Variation of maximum acceleration with depth (Comparison of SHAKE91 and FLAC-3D results for $V_s = 150$ m/s and 1999 Kocaeli EQ)



Figure 4.4-11. Variation of maximum shear stress with depth (Comparison of SHAKE91 and FLAC-3D results for $V_s = 150$ m/s and 1999 Kocaeli EQ)



Figure 4.4-12. Variation of shear strain with depth (Comparison of SHAKE91 and FLAC-3D results for $V_s = 150$ m/s and 1999 Kocaeli EQ)



Figure 4.4-13. Comparison of elastic spectra for SHAKE91 and FLAC-3D results for V_s = 150 m/s and 1999 Kocaeli Earthquake

4.5 ANALYSIS WITH THE OVERLYING STRUCTURE

After having performed both 1-D and 3-D dynamic response analyses of free field sites and having achieved satisfactory consistency between both methods, now 3-D analyses of the soil and structural system can be performed with the calibrated mesh window and size. Initially, a static analysis of the structure-soil system was performed. As stated earlier, the reason behind these static analyses is to calculate the vertical and shear stresses induced due to the presence of the structure. The static analyses are executed till a static equilibrium is reached in the finite difference solution scheme. Then, the earthquake excitation is applied at the rock interface to assess the soil-structure-earthquake interaction problem. Details of the generic structural systems, static analyses of these structural systems with the soil beneath and dynamic analyses with structures are described in the following sections.

4.5.1 Generic Structural Systems

As soon as the results of one and three dimensional analyses are found to be similar, the next step involves the investigation of the effects of overlying super structures on seismic performance of foundation soils. For this reason, beam and column elements constituting the load bearing elements of the structure are modeled as founded on the soil profiles. Different types of generic buildings are modeled to study the effects of structural layout. From structural point of view, the change in structural stiffness and masses as well as the dimensions of the structures are known to be the main cause of variations in liquefaction triggering potential for foundation soils. For this reason, structures having four different natural periods, ranging between 0.2 and 0.5 seconds, have been developed. The structural element size and properties are shown in Table 4.5-1. The dimensions of the beam, column and foundation elements are kept constant with only changing the height of the structure for the purpose of achieving a longer initial natural period.

Structure ID	Period of the structure (s)	Number of stories	Beam Dimensions (cm)	Column Dimensions (cm)	Mat Foundation Thickness (cm)
1	0.22	2	30 x 30	50 x 50	30
2	0.38	4	30 x 30	50 x 50	30
3	0.30	3	30 x 30	50 x 50	30
4	0.50	5	30 x 30	50 x 50	30

 Table 4.5-1. Properties of structures used in the analyses

The structures used in the analyses are simple frame structures consisting of simple elastic beams and columns resting on mat foundations. The columns and beams are modeled with element '*beamSEL*' whereas the mat foundation is modeled with '*shellSEL*' in FLAC-3D. The *beamSELs* are two-noded, straight finite elements with six degrees-of-freedom per node and is presented schematically in Figure 4.5-1. They behave as linearly elastic material with no failure limit. They are rigidly connected to the grid such that forces and bending moments develop within the beam as grid deforms. Three-noded, flat finite elements, *shellSELs* are used to model the foundation system: mat foundation. Each *shellSEL* behaves as an isotropic, linearly elastic material with no failure limit and is presented schematically in Figure 4.5-2. *ShellSELs* are used to model the structural support provided by any thin-shell structure in which the displacements caused by transverse shearing deformations can be neglected.



Figure 4.5-1. BeamSEL coordinate system and 12 active degrees-of-freedom of the beam finite element



Figure 4.5-2. Shell-type SEL coordinate system and 18 degrees-of-freedom available to the shell finite elements

4.5.2 Static Analyses

After having defined the material properties for the soil and the structural elements, the static analyses were performed until the soil-structure system reaches to equilibrium. This equilibrium is maintained when the unbalanced force ratio in the system drops below a certain limit (defined as 1.0×10^{-5} in FLAC-3D). The finite difference mesh used in the analyses is presented in Figure 4.5-3. Here; it should be noted that the boundary conditions of this static analysis are different than those of the dynamic analyses. Static boundary conditions require all vertical, horizontal and rotational fixities at the edges of the soil profile, far enough from the structure to overcome a general instability problem.



Figure 4.5-3. Typical 3-D mesh for static analysis

As stated earlier, static analyses for structure and soil system are performed for the purpose of assessing the effects of initial static shear and overburden stresses. The ratio of the static shear stresses to vertical effective stress, widely known as α ratio, was calculated by dividing the static shear stresses estimated at the end of static stepping by the vertical effective stress corresponding to the same stage as previously defined in Chapter 3, Section 3.3.1.3. A schematic view showing the variation of α and K_{α} beneath the foundation is presented in Figure 4.5-4 and Figure 4.5-5 respectively for illustrative purposes under a typical 4-storey building resting on a medium dense sand. To obtain these values from FLAC-3D, the programming language, FISH has been utilized which is a special language used simultaneously within FLAC-3D. FISH has been embedded into FLAC-3D and it gives the users to

define new variables and functions which may be used to extend the use of FLAC-3D and to add user-defined features. For the purpose of obtaining α and K_{α} values, a FISH function has been developed and embedded into the input file of FLAC-3D which calculates the ratio of initial shear stress (α) and the correction factor for this initial shear stress (K_{α}) using that α value. In Figure 4.5-4 and Figure 4.5-5, the word "Gradient' in the legends refers to α and K_{α} values respectively Figure 4.5-4 agrees that, value of this initial static ratio gets its highest value at the edges (about 0.58 maximum) of and its importance diminish going away from the structure. The value of K_{α} is about unity as this soil type is medium dense (Figure 3.3-3).



Figure 4.5-4. Variation of α beneath a typical structure (Site # 3 & Str # 2)



Figure 4.5-5. Variation of Ka beneath a typical structure (Site # 3 & Str # 2)

Variation of vertical stresses beneath the structures is also examined throughout these static analyses. Figure 4.5-6 shows the variation of K_{σ} in the soil profile beneath the foundation. Similar to of α and K_{α} values, K_{σ} values are also obtained by a FISH function and are plotted in this figure. The word "Gradient' in the legend of these figures refers to K_{σ} values similar to the previous ones. K_{σ} values show a great variability at shallow depths.



Figure 4.5-6. Variation of K_{σ} beneath a typical structure (Site # 2 and Str # 4)

4.5.3 Dynamic Analyses

After having completed 3-D static analyses, dynamic analyses of the soil-structure system was performed for the purpose of investigating the effects of the structures on liquefaction triggering potential of foundation soils. The only difference between the 3-D free field and SSEI system analyses is the presence of the structure in the model. All other aspects, including boundary conditions, model parameters, applied earthquake excitations, etc, were kept the same. A typical finite difference mesh used in the analyses is presented in Figure 4.5-7. It should be noted that the boundary conditions in this figure are similar to the one in Figure 4.4-9 rather than the one in Figure 4.5-3.



Figure 4.5-7. Typical mesh used in dynamic analyses of soil-structure system

Results from the dynamic analyses are processed in such a way that the maximum shear stress during the excitation is recorded. The shear stress at the beginning of the excitation (initial/static shear stress) is subtracted from this maximum shear stress and the portion of the shear stress that is purely dynamic (i.e.: the oscillating part of the shear stresses) is obtained. These cyclic shear stresses are then normalized with the initial vertical effective stress to estimate the uncorrected CSR value which will later be corrected for K_{σ} and K_{α} effects. Details of the post-processing are explained in the following sections.

4.6 POST-PROCESSING OF SSEI ANALYSES RESULTS

After solving the numerical model under both static and dynamic loading conditions, the outputs of these analyses need to be processed to estimate the significant variables of the problems. Details of this post processing will be explained in the following sections.

4.6.1 Post-Processing of Static Analyses

Results of the static analyses were used in determining the initial conditions of the soil-structure system as defined in the preceding sections. Effects of the vertical effective stress (K_{α}) and initial shear stress (K_{α}) are calculated according to the results obtained from the static part. The outline followed during this process is summarized in Figure 4.6-1. As the figure implies, as the starting step, initial (static) is recorded select locations shear stress (τ_0) at of (x/B = 0.06, 0.31, 0.63 and etc.) and (d/B = 0.06, 0.31, 0.63 and etc.) beneath the structure, where x is the distance measured from the center of the structure, d is the depth from ground surface and B is the width of the structure or the foundation. Next comes the estimation of vertical effective stress (σ_v) at the same points. This effective vertical stress is the resultant stress due to the weight of both structure and soil body. This effective stress is used in two calculations: i) estimating the initial shear stress ratio $\left(\left(\alpha = \frac{\tau_0}{\sigma_v}\right)\right)$ and CSR ii) calculating the correction factor for overburden stress (K_{σ}). After determining the α field, the correction factor K_{α} is estimated as proposed by NCEER (1997). Procedures for the calculations of α and K_{α} have been discussed in detail in Chapter 3. For 'K_{σ}' values, expressions proposed by Boulanger and Idriss (2004) as given in Equation 3-26a and 3-26b in Chapter 3 are utilized.

With the values of K_{α} and K_{σ} estimated, corrections on the cyclic stress ratio can be applied.



Figure 4.6-1. Outline followed in processing static analyses results

4.6.2 Post-Process of Dynamic Analyses

Figure 4.6-2 summarizes the main steps followed in the post-processing of dynamic analyses calculations. Items in italic as shown in Figure 4.6-2 are the results obtained from the static part.



Figure 4.6-2. Outline followed in processing dynamic analyses results

For the assessment of the results of the dynamic analyses, initially the maximum shear stresses (τ_{max}) acting on the horizontal planes which were developed during the earthquake excitation needs to be calculated. These shear stresses may act in either positive or negative *x*-direction (in the direction from which the earthquake excitation is applied). It should be noted that τ_{max} is simply the sum of cyclic and the initial shear stresses acting on the horizontal plane. Knowing the initial shear stress

value (τ_0) after static analyses, oscillating portion of the shear stress, which is defined as dynamic shear stress (τ_{cyclic}), is obtained by simply subtracting these initial shear stresses (τ_0) from the maximum total horizontal shear stresses (τ_{max}). Absolute value of this purely cyclic shear stress is normalized with initial effective vertical stress, σ'_0 , to find the uncorrected (raw) structural induced cyclic shear stress value, CSR_{SSEI, α , σ} as defined in Equation (4-2). It should be noted that this maximum shear stress ratio is multiplied with the factor 0.65 to convert the maximum value to an equivalent uniform stress.

$$CSR_{SSEI,equivalent,\alpha,\sigma} = 0.65 \times \frac{|\tau_{\max} - \tau_0|}{\sigma'_0}$$
(4-2)

This equivalent CSR_{SSEI} value is corrected with K_{α} and K_{σ} obtained from the static analyses as defined in Equation (4-3).

$$CSR_{SSEI,equivalent,\alpha=0,\sigma=100kPa} = \frac{CSR_{SSEI,equivalent,\alpha,\sigma}}{K_{\alpha} \times K_{\sigma}}$$
(4-3)

This $CSR_{SSEI,equivalent,\alpha=0,\sigma=100kPa}$ value can now be conveniently used for the assessment of liquefaction triggering potential of the foundation soils and will be designated as CSR_{SSEI} beyond this point on.

An illustrative CSR_{SSEI} field estimated just beneath a structure (at a depth of 0.50 m) is plotted in Figure 4.6-3 for the generic site No. 1 (V_s = 100 m/sec) and generic structure No. 3 (T_{str} = 0.3 sec). As the figure implies, the CSR_{SSEI} value reaches to its maximum value near the edges of the structures.



Figure 4.6-3. CSR_{SSEI} values at the foundation soil for Site No. 1 and Str 3

4.7 CONCLUDING REMARKS

In this chapter, details of the numerical modeling scheme followed for the assessment of soil-structure-earthquake interaction problem is defined. The adopted constitutive models and numerical algorithm schemes as well as the geometry and characteristics of the soil profiles, structural systems and strong ground motions used are discussed. Generation of the finite difference mesh, analyses types, comparison of 1-D and 3-D analyses are presented. Properties of the generic soil profiles, procedure followed during the compilation of soil parameters is described in detail. Main characteristics of the earthquake excitations, how they are applied to the site in concern is explained. As the final remark, post-processing of the numerical analyses results was described. In the next chapter, simplified analyses procedure proposed on the basis of these results will be discussed.

CHAPTER 5

PROPOSED SIMPLIFIED PROCEDURE FOR SOIL-STRUCTURE-EARTHQUAKE INTERACTION ASSESSMENT

5.1 INTRODUCTION

In this chapter, a simple, practical to use, closed-form solution for determining liquefaction triggering potential of foundation soils is proposed. The proposed methodology is based on widely used demand term CSR, however with series of corrections applied to reliably represent the seismic liquefaction response of the soil-structure-earthquake interacting systems' foundation soils.

Shear stress fields beneath foundations show significant variations as discussed in the previous chapter. For example, there exist very little or no shear stresses acting on the horizontal planes just at the symmetry line of the foundation and it starts to somewhat gradually increase, followed by a sharp increase in the vicinity of edges. The scatter of these static shear stresses, even everything else is assumed as constant, creates differences in the estimated cyclic stress ratios which are actually corrected for K_{α} and K_{σ} effects. These scattered data result in a significantly non-uniform CSR field. Figure 5.1-1 illustrates a typical ratio of CSR_{SSEI} to CSR_{FF} field estimated

under a 3- story residential structure founded on loose - medium dense sand ($V_s = 200 \text{ m/s}$) deposits.



Figure 5.1-1. Variation of CSR_{SSEI}/CSR_{FF} field with depth

In this figure the corners of the structure has a higher CSR_{SSEI}/CSR_{FF} value which reflects detrimental effects of the structure on liquefaction triggering potential at/near the surface. When we go deeper, this ratio converges to 1.0 which indicates that the effect of the structure disappears at about 2.5 m for this individual case. As the figure implies, representing such a varying field with one or two, easy to estimate values is naturally difficult, however it will be attempted.

From engineering point of view, both average and maximum responses are considered to be meaningful. Thus, as a first attempt representative CSR_{SSEI} value (CSR_{SSEI,rep}) is defined as the average of the CSR_{SSEI} values within the foundation influence zone, discussion of which is presented later in this chapter. The complementary parameter is defined as the maximum CSR_{SSEI} value (CSR_{SSEI,max}) within this influence zone. Usually this maximum value of CSR_{SSEI} is observed in the vicinity of the foundation edges. It is believed that CSR_{SSEI,rep} and CSR_{SSEI,max} are good indicators for the assessment of bearing capacity after during and immediately after earthquake shaking and tilting potential of foundations, respectively.

Through series of numerical simulations, representative and maximum CSR values were estimated and a database is compiled. Founded on these both static and dynamic numerical analyses results, a simplified, probabilistically-based framework is developed for the purpose of estimating CSR.

Before going into details of the seismic numerical analyses, static analyses and postprocessing of their results are discussed and presented in terms of static shear stress ratio (α) and effective vertical stress (σ_v'). Consequently, use of these static results in dynamic analyses is explained in terms of CSR_{SSEI,rep} and CSR_{SSEI,max}.

5.2 STATIC STATE

Before the dynamic assessment of soil-structure-earthquake interaction problems, static state of the structural and soil systems should be assessed for the purpose of

estimating vertical and shear stress fields. These stress values are required for the estimation of K_{σ} and α . The α values are consequently used in the estimation of K_{α} . Many researches have focused on predicting static stress state and corollary correction factors and these were reviewed in Chapter 3. In this chapter, simplified methodologies will be proposed to estimate i) vertical effective stresses, ii) α values, iii) cyclically-induced shear stresses of the soil and structure interacting system. Moreover, results of numerical analysis will be compared with the predictions.

5.2.1 Static Vertical Effective Stress State

The vertical stresses just beneath the structure can be calculated precisely by summing up all the dead and may be some of the live loads imposed by the structure. Within the scope of this study, a simplified method is implemented for the estimation of the vertical stress at the foundation level and dissipation of this vertical stress through depth 'z'. The formulation for the value of vertical effective stress σ'_v at any depth *z* is:

$$\sigma'_{structure+soil}(z) = \sigma'_{soil}(z) + \frac{(\sigma_{str} \times B \times L)}{(B + m_{\sigma}z) \cdot (L + m_{\sigma}z)}$$
(5-1)

where

$\sigma'_{soil}(z)$: Effective vertical stress at depth z (kPa)		
σ_{str}	: Total foundation stress generated by the structure (kPa)		
В	: Width of the structure (m)		
L	: Length of the structure (m)		
m _σ	: Vertical stress dissipation factor (0.9 in this study)		
Z	: Depth from the ground surface (m)		

In this methodology, stress increase per storey of the structure is roughly assumed as 15 kPa. This structural-induced stress is added to the vertical effective stress of the soil to find the total effective stress at a certain depth. However, it should be noticed that the foundation influence zone expands by a factor of m_{σ} as loads are transferred to deeper soil layers. There are series of recommendations regarding the assessment of this influence zone: 1H:2V rule, 60° approximation rule, elastic solutions, Boussinesque's rule, etc. After statistical assessments, m_{σ} is estimated as 0.9 which produced the best fit with the results of the numerical analyses. This value is found to be very consistent with 1H:2V and 60° approximation rules. Figure 5.2-1 shows the comparisons of effective stress values calculated by numerical analyses and by the use of Equation (5 – 1). The simplified formulation presented in Equation (5 – 1) or any other empirical (such as 1H:2V rule, 60° approximation rules) or elastic solution based predictions can be conveniently used in the calculation of vertical effective stresses induced by the structure and the soil overburden.



Figure 5.2-1. Comparison of the effective stresses calculated as a result of numerical analyses and proposed formulation

5.2.2 Static Shear Stress State

Initial (static) shear stresses show significant variations underneath foundations as a function of foundation type and stiffness as compared to the stiffness soil layers and type of foundation soils. These stresses are usually zero (or very small) at the load symmetry line of the foundation and increase gradually towards the edges. The highest value of the shear stress is observed at or around the edges of the foundation. As expected, values of these shear stresses decrease significantly with depth and beyond one width depth from the foundation level, they can be concluded to be negligibly small. Figure 5.2-2 shows the dissipation of the structure, "x" is the distance from the center of the structure in these figures. Vertical dashed lines (x/B = 0.5) represent the edges of the structures. As the figures imply, shear stress values approach to zero at distances x/B > 1.0, approaching to the free field ground conditions.

Instead of calculating the shear stress beneath the foundations, it is attempted to represent the shear stress value beneath the foundations by the initial (static) shear stress ratio, α which was defined in Section 3.3.1.3.1 and also in Equation (5 – 2).

$$\alpha = \frac{\tau}{\sigma_{v}}$$
(5-2)

Figure 5.2-3 shows the variation of α field along the width of the structure at various depths. Similar to the behavior of the shear stress field, α reaches its peak value within the zone of $0.4 \le x/B \le 0.6$, beyond which it diminishes rapidly and approaches to a value of zero at a distance of 1.0 width from the centerline of the structure. α value decrease sharply with depth and beyond 1 B depth, it approaches to negligibly small values.



Figure 5.2-2. Dissipation of initial (static) shear stresses with depth (for D/B = 0.06, 0.19, 0.31, 0.56, 0.81, 1.06, 2.06 and 3.06 respectively)



Figure 5.2-3. Variation of initial (static) shear stress ratio along the structure (for D/B = 0.19, 0.31, 0.56, 0.81, 1.06, 2.06 and 3.06)

When this wide range of α values are considered from liquefaction triggering point of view, selection a unique representative value is rather difficult. Two alternatives exist: i) an 'average' value and ii) a 'maximum' value. Figure 5.2-4 shows schematically these average and maximum values. Horizontal thick dashed line shows the average (representative) value (not the actual value but a schematic representation) and the dot represents the peak (maximum) value for this particular cross-section. Within the scope of this chapter, it is attempted to estimate both of these values by simple yet reliable formulations, the details of which are discussed in the following sections.



Figure 5.2-4. Schematic view for 'average' and 'maximum' a

5.2.2.1 Representative α value:

Inspired from the bearing capacity concepts, α_{rep} is estimated by a weighted average scheme as given in Equation (5 – 3). In simpler terms, at a certain depth, the soil mass that carries more of the foundation stresses, $\Delta\sigma$, has a higher weighting on the estimation of the α field.

$$\alpha_{rep} = \frac{\sum_{i=1}^{N} \Delta L_i \cdot \Delta \sigma'_{v,i} \cdot \alpha}{\sum_{i=1}^{N} \Delta L_i \cdot \Delta \sigma'_{v,i}}$$
(5-3)

where;

- α_{rep} : Average (representative) α value along the foundation
- $\Delta \sigma'_{v,i}$: Structural-induced vertical stress difference
- ΔL_i : Length along which $\Delta \sigma'_{v,i}$ is influential
- α : α value along the length ΔL_i

 $\Delta\sigma'_{v}$ is defined as the difference between the structural-induced and the free field vertical effective stresses as presented in Figure 5.2-5. Next, α values are estimated for each ΔL increment as defined in Chapter 3. Similarly, $\Delta\sigma'_{v}$ values are estimated for the same distance increments which are defined as the difference between the structural-and soil induced and the free field vertical effective stresses. The ratio of

 $\sum_{i=1}^{N} \Delta L_{i} \cdot \Delta \sigma'_{v,i} \cdot \alpha \text{ to } \sum_{i=1}^{N} \Delta L_{i} \cdot \Delta \sigma'_{v,i} \text{ results in } \alpha_{\text{rep}} \text{ value.}$



Figure 5.2-5. Schematic view of $\Delta \sigma_v$ at depth 0.06B

A flowchart showing calculation steps of α_{rep} are presented in Figure 5.2-6.

A numerical illustration of these calculation steps is shown in Table 5.2-1. The ratio of the sum of the last two rows in Table 5.2-1 gives α_{rep} value.


Figure 5.2-6. Calculation steps for α_{rep}

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u (kPa) 5 σ'_v (kPa) 37 31 31 49 38 4 4 4 $\sigma'_{v,FF}$ (kPa) 4 4 4 4 4 4 4 4 $\Delta\sigma'_v$ (kPa) 33 28 27 45 34 0 0 0 α 0.069 0.106 0.032 0.065 0.657 0.210 0.091 0.040 SUM: $\Delta L \cdot \Delta \sigma'_v \cdot \alpha$ 2 3 1 3 22 0 0 0 31 $\Delta L \cdot \Delta \sigma'_v$ 33 28 27 45 34 0 0 0 31 $\Delta L \cdot \Delta \sigma'_v$ 33 28 27 45 34 0 0 0 31 $\Delta L \cdot \Delta \sigma'_v$ 33 28 27 45 34 0 0 0 167 α_{rep} $\frac{31}{167} = 0.186$ α_{rep} $\frac{31}{167} = 0.186$
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$\alpha_{\rm rep} = \frac{31}{167} = 0.186$

Table 5.2-1. Illustrative calculation of α_{rep}

A set of values illustrating the variation of α_{rep} with respect to depth is presented in Figure 5.2-7. Bolder lines represent the values estimated for a 4 story structure (T_{str} = 0.38 s) whereas others are for 2 story structures (T_{str} = 0.22 s).

As illustrated in Figure 5.2-7, α_{rep} values are relatively higher at depth range of $0 \le D/B \le 0.5$, without much affected from the type and weight of the structure and soil type and stiffness. Beyond D/B=0.5 to 1.0, α_{rep} values decrease rapidly to a value of almost zero at and beyond the depth of 3.0 B.

As presented by Figure 5.2-7, α_{rep} values exhibit a variation depending on the height of the structure (number of stories or period). Clearly, α_{rep} values are observed to be inversely proportional to the structural periods (number of storey). On the basis of this observation α_{rep} values are attempted to be empirically modeled with the help of the following expression:

$$\alpha_{rep} = \exp\left(\frac{z - 1.48}{-4.36}\right) \times \frac{T_{str}^{-0.12}}{10.03} \times N_{1,60}^{0.04}$$
(5-4)



Figure 5.2-7. Variation of α_{rep} with depth

where:

z : depth from ground surface

T_{str} : natural period of the structure

 $N_{1,60}$: overburden and energy corrected SPT-N blow count.

Model coefficients of the equation are to be predicted within a probabilistic framework described in Section 5.5.1.4. The predictions by Equation (5 - 4) are presented in Figure 5.2-8. Darker lines are drawn by using Equation (5 - 4) for limiting cases, where N_{1,60} = 2 and 84 and T_{str} = 0.22 and 0.60, whereas the lighter lines are the α_{rep} values estimated after numerical simulations.

The goodness of the predictions is compared in Figure 5.2-9. Almost all of the data falls within 1:2 and 2:1 lines with an R^2 value of 80%.



Figure 5.2-8. Comparison of the model prediction with available α_{rep} vs. depth



Figure 5.2-9. Comparison of α_{rep} values calculated as a result of numerical analyses and proposed formulation

5.2.2.2 Maximum α value:

The importance of the maximum value of the initial shear stress ratio, α_{max} was addressed earlier. Similar to the methodology followed in α_{rep} , the decrease of α_{max} values with depth is presented in Figure 5.2-10.



Figure 5.2-10. Variation of α_{max} values with depth

This figure implies that, following a similar trend with α_{rep} , α_{max} values are observed to be relatively higher within the depth range of $0 \le D/B \le 0.5$ without much affected from the type and weight of the structure and soil type and stiffness. Beyond D/B=0.5 to 1.0, α_{max} values decrease rapidly to a value of almost zero at and beyond the depth of 2.0 B.

With a slight modification to the empirical α_{rep} functional form, Equation (5-5) is achieved which is developed to be used for the simplistic estimation of α_{max} values.

$$\alpha_{\max} = \exp\left(\frac{z - 7.08}{-2.87}\right) \times \frac{T_{str}^{0.45}}{10.36} \times N_{1,60}^{-0.06}$$
(5-5)

The predictions of this equation for $T_{str} = 0.22$ and 0.60 and $N_{1,60} = 2$ and 84 are shown by a bolder line. It is observed that α_{max} fall within a narrower range compared to the values of α_{rep} .

The goodness of the predictions is compared in Figure 5.2-12. Almost all of the data falls within 1:2 and 2:1 lines with an R^2 value of 93%.



Figure 5.2-11. Comparison of the model prediction with available α_{max} vs. depth lines



Figure 5.2-12. Comparison of α_{max} values calculated as a result of numerical analyses and proposed formulation

5.3 DYNAMIC STATE

Dynamic soil-structure interaction from liquefaction point of view and its relative importance were discussed earlier. This section is devoted to the presentation of the proposed simplified procedure to assess the problem.

How to estimate the structural-induced cyclic shear stresses from the results of numerical analyses (CSR_{SSEI}) was discussed in detail in Chapter 4 (Figure 4.6-2). However, it is worth to express some of the observational trends. CSR_{SSEI} values vary significantly beneath the foundation. It is generally low at the mid section of the foundations and increases dramatically at the edges for the mat foundation. A typical variation of CSR_{SSEI} values beneath a mat foundation is shown in Figure 5.3-1 (a) and (b). In these figures x/B = 0 and x/B = 0.5 represent the center and

corner of the foundation, respectively. In Figure 5.3-1 (a), one can observe that CSR_{SSEI} values beneath the foundation are lower than the ones of the free field, clearly addressing the "positive" effect of the structural system on the liquefaction triggering potential. However, in Figure 5.3-1 (b), presence of the structural system adversely affects the soil liquefaction triggering response.



Figure 5.3-1. Typical graphs showing the variation of CSR_{SSEI}

There exist similar concerns regarding how to define the CSR field with a single value reliably. Similar to the solution proposed for the estimation of α_{max} and α_{rep} values, CSR field is attempted to be modeled through two values: a representative and a maximum value. Figure 5.3-2 illustrates schematically both the representative and maximum values. Horizontal dashed line is the representative CSR_{SSEI} value, CSR_{SSEI,rep}, and the dot at the peak point addresses the maximum CSRR_{SSEI} value, CSR_{SSEI,max}.



Figure 5.3-2. Typical CSR_{SSEI, rep} and CSR_{SSEI, max} value

Following sections will explain the determination of the representative and maximum CSR_{SSEI}, values by the numerical analyses and the proposed simplified procedure.

5.3.1 Representative CSR_{SSEI} Value

As presented in Figure 5.3-1, CSR_{SSEI} values show variations along a straight line under the structures. In this study, a representative CSR_{SSEI} value presenting the average response of the foundation soils in case of an earthquake is attempted to be defined similar to the methodology followed for estimating α values. Horizontal thick dashed line in Figure 5.3-3 shows this average (representative) CSR_{SSEI} value ($CSR_{SSEI, rep}$) schematically for a typical case.



Figure 5.3-3. Typical CSR_{SSEI, rep} value

This representative value can be calculated using the results of the numerical analyses as explained below or from a simplified expression that will be defined later on in this chapter.

This representative CSR_{SSEI} value is calculated in such a way that structural effects are considered, i.e. a higher contribution from points where structural-induced forces are higher (e.g. corners of the structures) and less contribution where these forces are smaller (i.e. free field). Equation (5-6) summarizes this calculation process.

$$CSR_{SSEI,rep} = \frac{\sum_{i=1}^{N} \Delta L_i \cdot \Delta \sigma'_{v,i} \cdot CSR_{SSEI}}{\sum_{i=1}^{N} \Delta L_i \cdot \Delta \sigma'_{v,i}}$$
(5-6)

where

$\Delta \sigma'_{v,i}$:	Structural-induced vertical stress difference
ΔL_i	:	Length along which $\Delta \sigma'_{v,i}$ is influential
CSR _{SSEI}	:	CSR_{SSEI} value along the length ΔL_i

It should be noted that this expression has a similar form with the α_{rep} equation with a slight modification for CSR_{SSEI,rep}. For this reason, outline presented in Figure 5.2-6 is also applicable to CSR_{SSEI,rep}. For illustration purposes, calculation of CSR_{SSEI,rep} value for the case shown in Figure 5.3-3 will be summarized here in Table 5.3-1. Like the case of representative α , CSR_{SSEI,rep} values are calculated by dividing the sum of the last two rows to each other.

5.3.2 Maximum CSR_{SSEI} Value

The importance of the maximum value of the cyclic stress ratio, $CSR_{SSEI,max}$ was addressed earlier. This $CSR_{SSEI,max}$ value is shown with a big dot in Figure 5.3-2. Similar to the representative case, maximum value of CSR_{SSEI} can also be calculated

from both the numerical analyses and from a simplified procedure. Details for these will be discussed in the following sections.

x/B	0.063	0.19	0.31	0.44	0.56	0.94	1.31	1.56	
$\Delta L(\mathbf{m})$	1.00	1.00	1.00	1.00	1.00	3.00	3.00	3.00	
τ (kPa)	9	10	12	11	10	3	3	3	
σ _v (kPa)	-63	-41	-25	-85	-53	-8	-9	-9	
u (kPa)					5				
σ' _v (kPa)	58	36	20	80	48	3	4	4	
σ' _{v,FF} (kPa)	4	4	4	4	4	4	4	4	
$\Delta \sigma'_{v} (kPa)$	54	32	16	76	44	-1	0	0	
CSR	0.104	0.178	0.400	0.091	0.130	0.594	0.520	0.543	SUM:
$\Delta L \cdot \Delta \sigma'_{v} \cdot CSR$	6	6	6	7	6	0	0	0	31
$\Delta L \cdot \Delta \sigma'_{v}$	54	32	16	76	44	0	0	0	222
								CSR _{rep}	$\frac{31}{222} = 0.14$

Table 5.3-1. Illustrative example calculation of CSR_{SSEI, rep}

It is easy to find the maximum CSR_{SSEI} value when compared to the representative one, as it does not require a weighted averaging scheme, instead the maximum value of CSR_{SSEI} is directly read from the outputs of the numerical simulations.

5.4 COMPARISON OF CSR_{SSEl,rep} & CSR_{SSEl,max} WITH EXISTING SSEI PARAMETERS

In the literature, two parameters with significant use in soil-structure-earthquake interaction problems exist. These are " σ " as proposed by Veletsos and Meek (1974) and "S_A/PGA" as chosen by Rollins and Seed (1990). This section will present the variation trends of CSR_{SSEI}/CSR_{FF} with these two existing parameters.

Veletsos and Meek (1974) has introduced a term, σ , which represents the ratio of structure-to-soil stiffness. The definition of σ is presented in Equation (5 – 7). It was concluded that a) σ (the ratio of the structure-to-soil stiffness), b) aspect ratio (the ratio of structure height to foundation radius or width) and c) the interaction of the fixed-base natural frequency of the structure to the frequency regions of the design spectrum were the critical factors controlling SSEI. If σ values fall into the range of 3 to 20, then soil-structure interaction is concluded to be critical.

$$\sigma = \frac{V_{s,\text{final}} \times T_{str}}{h_{effective}}$$
(5 - 7)

 σ , itself is not a parameter, which can directly be used to estimate the value of CSR_{SSEI}. Instead, it is an indicator of the effect of the structure on liquefaction triggering compared to the free field sites. On this basis; the ratio of CSR_{SSEI} to CSR_{FF} (free field CSR value) is thought to be more correlated to σ rather than CSR_{SSEI} itself. Figure 5.4-1 illustrates the variation of CSR_{SSEI} /CSR_{FF} with σ. Values greater than 1.0 indicates CSR_{SSEI} > CSR_{FF}; i.e. the presence of the structure affects negatively the liquefaction triggering potential of soil sites.



Figure 5.4-1. Comparison of CSR_{SSEI, rep} value with σ

As the figure implies, that consistent with the observations of Veletsos and Meek, for σ values in the range of 3 and 20, estimated CSR_{SSEI}/CSR_{FF} values are higher than one. In simpler terms, when soil structure interaction affects are critical, then they adversely affect liquefaction triggering response. As a conclusion, if σ is estimated to be in the range of 3 to 20, then CSR ratio is expected to be greater than 1. Beyond these limits, effects of SSEI on liquefaction triggering may not be critical.

The second parameter of concern is S_A/PGA as defined by Rollins and Seed (1990). A detailed discussion on this was presented in Section 3.2. In their study, they suggested that if spectral acceleration ratio, S_A/PGA is larger than about 2.40, the induced cyclic stress ratio value would be higher beneath the foundation than the one in the free field. If it is less than about 2.40, then the cyclic stress ratio beneath the foundation is less, thus liquefaction triggering potential of foundation soils decreases compared to the ones of free field. The same issue regarding the use of S_A/PGA as an indicator of relative response of soil and structure sites compared to free field response. Thus S_A/PGA can be a good indicator if SSEI is critical or not. Figure 5.4-2 illustrates the variation of CSR_{SSEI}/CSR_{FF} with S_A/PGA . Values greater than 1.0 indicates $CSR_{SSEI} > CSR_{FF}$; i.e. the presence of the structure affects negatively the liquefaction triggering potential of soil sites. As S_A/PGA increases, the CSR_{SSEI}/CSR_{FF} value increases.



Figure 5.4-2. Comparison of CSR_{SSEI, rep} value with S_A/PGA

Inspired by these previous attempts, it is decided to incorporate both σ and S_A/PGA as model input parameters. There exist a number of other parameters which need to be considered to capture the SSEI effects from liquefaction triggering point of view. They can be classified into four categories: i) structural (geometric) properties, ii) earthquake characteristics iii) soil properties and iv) joint parameters. These important parameters with potentials to be used are listed below:

- i) Structural (Geometric) Properties:
 - Equivalent radius, R
 - Length of the structure, L
 - Width of the structure, B
 - Height of the structure, h
 - Natural period of the structure, T_{str}
 - Aspect ratio, h/B
- ii) Earthquake Characteristics:
 - Peak ground acceleration, PGA
 - Mean period of the earthquake, T_m
 - Predominant period of the earthquake, T_p
 - Spectral acceleration, S_A
 - Predominant period of the acceleration record at the surface, T_{p,surface}
- iii) Soil Properties:
 - Initial shear wave velocity, V_{s,ini}

- Final shear wave velocity after the earthquake excitation, $V_{s,\text{final}}$
- Period of the soil site, T_{soil}
- Initial shear modulus, G_{s,ini}
- Final shear modulus, G_{s,final}
- iv) Joint Properties:
 - σ
 - $\frac{\sigma}{PGA}$
 - $\frac{T_{soil}}{T_m}$

• Res =
$$\frac{T_{bldg}}{\sqrt{T_{soil} \times T_p}}$$

•
$$\frac{T_{str}}{T_{soil}}$$

•
$$\frac{V_{s,final} \times H_{str}}{B_{str}}$$

•
$$\frac{T_{soil}}{T_m}$$

• $\frac{T_s}{T_p}$

•
$$\frac{T_{soil}}{T_p}$$

Initial and final impedance ratios

Multi dimension regression analyses were performed to find the correlations of the parameters listed above with the observed CSR_{SSEI,rep} response. To take into account the non-linear effects of these parameters, exponential and natural logarithm of all the terms were also introduced into the model. Results of this regression analyses is presented in Figure 5.4-3 from the most to the least correlated parameter:

As the figure implies, the significant parameters were found to be the ratio of structural period T_{str} to predominant period of the earthquake excitation, T_p ; the ratio of soil period, T_{soil} to again the predominant period of earthquake excitation, T_p . The third one is the equivalent radius, r, of the foundation. Next comes, the ratio of structural period T_{str} to mean period of the earthquake excitation, T_m followed by the ratio T_{soil}/T_m . Period of the soil, T_{soil} is the next important parameter. Aspect ratio of the structure (h/B) is the following important parameter. Importance of existing σ and S_A/PGA are not as high as the others. However, σ includes period of the soil and the structure as well. This is presented as below:

$$\sigma = \frac{V_{s,\text{final}} \times T_{str}}{h_{\text{effective}}}$$
(5 -7)

where
$$\frac{V_{s,final}}{h_{effective}}$$
 is approximately equal to $\frac{1}{T_{soil}}$.

Other earthquake parameters, T_p and T_m of the earthquake are somehow related to the peak ground acceleration of the earthquake excitation. Due to the fact that they incorporate the other significant model parameters and they have a non-dimensional form, S_A/PGA, σ and aspect ratio, h/B are selected to be the tentative descriptive parameters for the assessment of structural induced CSR values. Details of the simplified procedure for the estimation of CSR_{SSEI,rep} and CSR_{SSEI,max} will be discussed next.



Figure 5.4-3. Correlation of the intended parameters with CSR_{SSEI, rep}

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5.5 SIMPLIFIED PROCEDURE FOR CSR_{SSEI,rep} & CSR_{SSEI,max}

Lengthy and complex nature of the dynamic analyses generally makes them unfeasible to use them in the dynamic soil-structure assessments of low rise (up to 6 stories) residential structures. Keeping the main difficulties and burden in numerical soil-structure and earthquake analyses in mind, two different models for structural-induced cyclic stress ratios have been defined within the contents of this study; namely representative CSR_{SSEI} and maximum CSR_{SSEI}. These two cyclic stress values are denoted as CSR_{SSEI,rep} and CSR_{SSEI,max} respectively. It is believed that the former one may be used for general bearing capacity assessments whereas the latter one along with the representative value can be used for assessing the tilting potential of structures due to liquefaction-induced bearing capacity failures.

5.5.1 Calculation of CSR_{SSEI, rep} by Simplified Procedure

As defined earlier, this value is intended to be used as a weighted average of all the CSR_{SSEI} values beneath the structure. The calculation steps of the $CSR_{SSEI,rep}$ value by using dynamic response analyses were defined in Section 5.3.1. This section is dedicated to the discussion of the proposed simplified methodology for the estimation of $CSR_{SSEI,rep}$ values without a need to more complex dynamic assessments. For comparison purposes, predictions by the proposed simplified methodology were plotted against the dynamic response analysis.

CSR is originally defined as the ratio of shear stresses to effective normal stress on any plane of interest. Although this general definition has been widely used for free field sites where horizontal planes are usually more critical due vertically propagating nature of seismic waves, modifications in the form of K_{α} factors were later introduced to assess planes of interest which are not aligned with the principal directions (i.e.: planes where under static conditions shear stresses apply).Compared to the case of seismic response of free field sites, when a structure overlies foundation soils both under static and dynamic conditions, interaction of the structure and soil need to be addressed. Then the critical question is to determine the descriptive parameters of the problem. As discussed earlier, the most significant parameters with potential affect on soil structure interaction induced liquefaction

triggering are "
$$\sigma$$
", " $\frac{S_A}{PGA}$ " and the aspect ratio " $\frac{h}{B}$ "

As the starting point, these significant parameters are used to decide how seismically induced shear stresses due to overlying structure, $\tau_b(z)$ and overburden soil, $\tau_{soil}(z)$ need to be added. If one remembers that due to seismically induced shear waves traveling vertically towards the foundation level and transmitted to the super structure, these two components of shear stresses can be in-phase or out-off phase. The contributions by these two interdependent and related sources of shear stresses are proposed to be estimated by the following expressions.

$$CSR_{SSEI, representative} = \frac{f(\sigma) \cdot f\left(\frac{S_A}{PGA}\right) \cdot f\left(\frac{h}{B}\right) \cdot \tau_b + \tau_{soil}}{\sigma'_{SSI}}$$
(5 - 8)

where

$f(\sigma)$: Contribution of σ whose form will be discussed later

 $f\left(\frac{S_A}{PGA}\right)$: Contribution of $\frac{S_A}{PGA}$ whose form will be discussed later

$$f\left(\frac{h}{B}\right)$$
 : Contribution of $\frac{h}{B}$ whose form will be discussed later

τ_{b} : Base shear induced by the structure

- τ_{soil} : Shear stress in the soil due to seismic loading
- σ'_{SSI} : Effective vertical stress induced by both the structure and the soil.

This expression implies that, structural-induced shear stress value will be weighted by using "some" correlations which are factors of important SSEI parameters σ , $\frac{S_A}{PGA}$ and $\frac{h}{B}$. Parameters of the Equation (5 – 8) will be discussed in detail in the following sections.

5.5.1.1 Base shear calculations in the simplified model:

Inspired by the simplified base shear formulations proposed in various international and national design codes, which were discussed in detail in Section 3.3.1.2, a simplified procedure for determining base shear is proposed along with Equation (5 - 8), unless a more rigorous methods of base shear assessments are preferred.

$$\tau_{b,\max} = 0.80 \times W \times S_A \tag{5-9a}$$

$$\tau_{b,eq} = 0.65 \times \tau_{b,\max} \tag{5-9b}$$

$$\tau_b(z) = \frac{\tau_{b,eq}}{(B + m_\tau \cdot z) \cdot (L + m_\tau \cdot z)}$$
(5 -9c)

where:

- $\tau_{b,max}$: maximum shear stress beneath the structure
- $\tau_{b,eq}$: equivalent shear stress beneath the structure
- $\tau_b(z)$: shear stress at depth "z"
- W : weight of the structure
- S_A : spectral acceleration
- B : width of the structure (m)
- L : length of the structure (m)

 m_τ \qquad : shear stress dissipation factor

z : depth from ground surface (m)

 $\tau_{b,eq}$ gives the equivalent base shear value that will develop at the foundation level. Equation 5 – 9c is proposed for modeling the diminishing effects of super-structural induced seismic shear stresses with depth. Shear stress reduction factor m_{τ} is estimated as 1.6 which produced the best fit to the numerical analyses estimated CSR values. It should be noted that shear stresses spread to a wider area compared to vertical stresses. (Vertical stress distribution factor, m_{σ} was equal to 0.9 as discussed in Section 5.2.1).

Another important aspect in this formulation is that the weight of the structure is multiplied with the spectral acceleration (S_A) estimated at the first period of the structure. However, selection of which period to use is not trivial: alternatives are to use: the initial period of the structure or the period during shaking (i.e. after soil-structure interaction is taken into account). Due to its simplicity, fixed base natural period (i.e. the initial period) is considered as the first choice. However, Veletsos and Nair (1975) have proposed formulations for predicting the lengthening ratios which incorporates soil-structure interaction effects. This flexible base period (lengthened period) can be estimated by the following equation:

$$\frac{\widetilde{T}}{T_{str}} = \sqrt{1 + \frac{k}{k_u} + \frac{kh^2}{k_\theta}}$$
(5 - 10)

where

 \widetilde{T} : lengthened period of the structure,

 T_{str} : fixed base period of the structure, $\sqrt{k/m}$

k : equivalent stiffness of the system

- k_u : translational stiffness of the structure,
- k_{θ} : rotational period of the structure
- h : effective height of the structure

Stewart (2000) has reviewed these existing models and the results of his study are presented in Figure 5.5-1. This figure summarizes period lengthening ratios and damping ratios for different embedment ratios (e/r = 0 and e/r = 1) where e is the depth of embedment and r is the equivalent radius of the structure. These two sets of figures remark an average period lengthening ratio of 1.1.

Depending on the structural periods used, different sets of spectral acceleration values can be estimated. Alternatives tested are: i) fixed base natural period of the structure, ii) lengthened period of the structure according to Figure 5.5-1 and iii) lengthened period of the structure with a lengthening ratio of 1.1. By using these alternatives, dynamic analysis based values and the simplified proposed methodology based predictions are compared. This comparison will be presented in the section where the simplified procedure for $CSR_{SSEI,rep}$ is presented.

5.5.1.2 Shear stresses due to soil overburden, τ_{soil}

For the purpose of use in Equation (5 - 8), seismically-induced shear stresses due to soil layer at a depth of h can be estimated as given in Equation (5-11). This issue has been discussed in detail in Section 3.3.1.1 and will not be repeated herein.

$$\tau_{soil} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \gamma_n \cdot z \cdot r_d \tag{5-11}$$



Figure 5.5-1. Comparison of Period Lengthening Ratios and Foundation Damping Factors for SDOF Structure with Rigid Circular Foundation on Half-Space for Surface and Embedded Foundations ($\upsilon = 0.45, \beta = 5\%, \gamma = 0.15, \zeta = 5\%$) (Veletsos and Nair 1975; Bielak 1975)

5.5.1.3 Effective stress due to weights of the structure and the soil, σ_{SSI}

The normalizing value in Equation (5 - 8) is the effective stress at depth z, where seismically-induced shear stresses are to be estimated. It is proposed to be calculated by using Equation (5 - 1), which is presented below as a reminder. Details regarding this equation were presented in Section 5.2.1.

$$\sigma'_{structure+soil}(z) = \sigma'_{soil}(z) + \frac{(\sigma_{str} \times B \times L)}{(B+0.9z) \cdot (L+0.9z)}$$
(5-1)

5.5.1.4 Functions of
$$\sigma$$
, $\frac{S_A}{PGA}$ and $\frac{h}{B}$

The best descriptive functional forms of σ , $\frac{S_A}{PGA}$ and $\frac{h}{B}$ presented in Equation (5 – 7) were chosen after testing series of alternatives. Maximum likelihood methodology was utilized to decide on the best alternative. In this section, details of the probabilistic assessment framework including the maximum likelihood methodology are discussed.

The first step in developing a probabilistic model is to select a limit state expression that captures the essential parameters of the problem. The model for the limit state function has the general form $g = g(\mathbf{x}, \boldsymbol{\Theta})$ where x is a set of descriptive parameters and $\boldsymbol{\Theta}$ is the set of unknown model parameters. Inspired by prior research studies, as well as the trends in the presented databases, the following model is adopted as the limit state functions for structural-induced representative cyclic stress ratio (CSR_{SSEI,rep}). The same model with different coefficients is adopted for the maximum cyclic stress ratio (CSR_{SSEI,max}) as well and will be discussed in the next section.

$$g_{CSR_{SSEI,rep}}(\sigma, \frac{S_A}{PGA}, \frac{h}{B}, \tau_b, \tau_{soil}, \Theta) = \ln(CSR_{SSEI,rep}) - \ln\left(\frac{f(\sigma) \times f\left(\frac{S_A}{PGA}\right) \times f\left(\frac{h}{B}\right) \times \tau_b + \tau_{soil}}{\sigma'_{SSI}}\right) \pm \varepsilon_{CSR_{REP}}$$
(5 - 12)

where

$$f(\sigma) = \theta_{\sigma 1} \times \exp(\theta_{\sigma 2} \cdot (\sigma))$$
 (5 - 12a)

$$f\left(\frac{S_A}{PGA}\right) = \theta_{s1} \times \exp\left(\theta_{s2} \cdot \left(\frac{S_A}{PGA}\right)\right)$$
(5-12b)

$$f\left(\frac{h}{B}\right) = \exp\left(\theta_{h1} \cdot \left(\frac{h}{B}\right)\right) \tag{5-12c}$$

where $\theta_{i,\gamma_{\max}}$ and $\theta_{i,\varepsilon_{\gamma}}$ are the set of unknown model parameters.

The proposed model include a random model correction term (ε) to account for the facts that i) possible missing descriptive parameters with influence on cyclic deformations may exist; and ii) the adopted mathematical expression may not have the ideal functional form. It is reasonable and also convenient to assume that ε has normal distribution with zero mean for the aim of producing an unbiased model (i.e., one that in the average makes correct predictions). The standard deviation of ε , denoted as σ_{ε} , however is unknown and must be estimated. The set of unknown parameters of the model, therefore, is $\Theta = (\theta, \sigma_{\varepsilon})$.

Assuming the value of representative structural-induced cyclic shear strain ratio value obtained after the analyses to be statistically independent, the likelihood function for "k" analyses where exact ratio values are available (i.e.: values at the end of the analyses are available), can be written as the product of the probabilities of the observations.

$$L_{\text{CSR}_{\text{SSEI,rep}}}(\theta, \sigma_{\varepsilon}) = \prod_{i=1}^{k} P \Big[g_{\text{CSR}_{\text{SSEI,rep}},i}(.) = 0 \Big]$$
(5 - 13)

Suppose the values of σ , $\frac{S_A}{PGA}$, $\frac{h}{B}\tau_b$, τ_{soil} and σ_{SSI} at each data point are exact, i.e. no measurement or estimation error is present, noting that $g(...) = \hat{g}(...) + \varepsilon_i$ has the normal distribution with mean \hat{g} and standard deviation σ_{ε} , then the likelihood function can be written as:

$$L_{CSR_{SSEI,rep}}(\boldsymbol{\theta}, \boldsymbol{\sigma}_{\varepsilon}) = \prod_{i=1}^{n} \varphi \left[\frac{\hat{g}_{CSR_{SSEI,rep}} \left(\boldsymbol{\sigma}, \frac{\boldsymbol{S}_{A}}{\boldsymbol{PGA}}, \frac{\boldsymbol{h}}{\boldsymbol{B}}, \boldsymbol{\tau}_{b}, \boldsymbol{\tau}_{soil}, \boldsymbol{\theta} \right)}{\boldsymbol{\sigma}_{\varepsilon_{CSR_{SSEI,rep}}}} \right]$$
(5 - 14)

where $\varphi[.]$ is the standard normal probability density function. Note that the above is a function of the unknown parameters.

As part of maximum likelihood methodology, the coefficients which are estimated to maximize the likelihood functions given in Equation (5 - 8) are presented in Table 5.5-1 and the resulting form of Equation (5 - 8) takes the form:

$$CSR_{SSEI,rep} = \frac{f(\sigma) \times f\left(\frac{S_A}{PGA}\right) \times f\left(\frac{h}{B}\right) \cdot \tau_b + \tau_{soil}}{\sigma'_{SSI}}$$
(5 - 15)

where

$$f(\sigma) = 14.77 \times \exp(-0.01\sigma) \qquad (5-15a)$$

$$f\left(\frac{S_A}{PGA}\right) = 0.79 \times \exp\left(-0.68\frac{S_A}{PGA}\right)$$
(5 - 15b)

$$f\left(\frac{h}{B}\right) = \exp\left(-0.66\frac{h}{B}\right) \tag{5-15c}$$

Table 5.5-1. Structural-induced representative CSR_{SSEI} model coefficients

$\theta_{\sigma 1}$	θ_{σ^2}	θ_{S1}	θ_{S2}	$\theta_{\rm H1}$	σε
14.77	-0.01	0.79	-0.68	-0.66	0.06

Figure 5.5-2 shows the proposed $f(\sigma)$, $f\left(\frac{S_A}{PGA}\right)$ and $f\left(\frac{h}{B}\right)$ factors.



Figure 5.5-2. Proposed functions for $f(\sigma)$, $f\left(\frac{S_A}{PGA}\right)$ and $f\left(\frac{h}{B}\right)$

A comparison of the recommended representative cyclic stress ratio value and the calculated value is presented in Figure 5.5-3 with using the fixed base natural period of the structural system. This figure represents a good correlation between the values calculated as a result of numerical analyses and proposed formulation. The solid line is the diagonal line (1:1) and the dashed lines show the border of 2:1 and 1:2. Being in this limited area is a satisfaction from probabilistic and geotechnical points of view.

Figure 5.5-3 shows a relatively satisfactory correlation between the modeled and the measured $CSR_{SSEI,rep}$ values (with a correlation coefficient of $R^2 = 0.75$). In this prediction for the estimation of the spectral acceleration values (S_A) and base shear

(τ_b), fixed base natural period of the structural system was adopted. However, it is a controversial issue in soil-structure interaction problems to decide which period parameter to use: i) fixed base natural period or ii) lengthened flexible period of the system as mentioned earlier. This issue was discussed in detail in Section 5.5.1.1. In probabilistic assessments, three different period values have been utilized. An average lengthened period equal to $1.1 \times T_{str}$ is also analyzed in addition to the two period alternatives. Results by using fixed-base natural period of the structure were presented in Figure 5.5-3. Figure 5.5-4 and Figure 5.5-5 present the calculated values for a typical lengthened period of $1.1 \times T_{str}$ and the actual lengthened period respectively.



Figure 5.5-3. Comparison of CSR_{SSEI, rep} values calculated as a result of numerical analyses and proposed formulation



Figure 5.5-4. Comparison of $CSR_{SSEI, rep}$ values for $T_{str} = 1.1T_{str}$



Figure 5.5-5. Comparison of CSR $_{\rm SSEI, \, rep}$ values for lengthened structural period

The correlations in Figure 5.5-4 and Figure 5.5-5 are concluded to be equally well. Thus use of the easiest to calculate, i.e.: fixed base period (period before seismic excitation and soil-structure interaction) of the structure is preferred.

5.5.2 Calculation of CSR_{SSEI,max} by the Proposed Simplified Procedure

Estimation of the CSR_{SSEI,max} value for a soil-structure earthquake interacting system is performed by two alternative methods: i) estimating the maximum value from the representative value defined in Section 5.5.1, ii) defining an expression similar to the one used for estimating the representative value. Probabilistic assessment framework of these alternative methodologies will be discussed briefly in the following paragraphs, with limited details as they were discussed in the preceding part. The first equation below has a functional from similar to the representative CSR model. The second alternative is based on the idea of estimating the maximum value by using the representative.

$$g_{CSR_{SSEI,max}}\left(\sigma, \frac{S_{A}}{PGA}, \frac{h}{B}, \tau_{b}, \tau_{soil}, \Theta\right) = \ln(CSR_{SSEI,max}) - \left[\left(\frac{\left(\theta_{m-\sigma1} \times \exp(\theta_{m-\sigma2} \cdot (\sigma))\right) \times \left(\exp(\theta_{m-h1} \cdot \left(\frac{h}{B}\right)\right)\right) \times \tau_{b} + \tau_{soil}}{\sigma_{SSI}}\right] \pm \varepsilon_{CSR_{max}}$$
(5 - 16)
$$g_{CSR_{SSEI,max}}\left(CSR_{rep}, \sigma, \alpha_{max}, \Theta\right) = \ln(CSR_{SSEI,max}) - \ln\left(\left(\theta_{m-1} \cdot CSR_{rep}\right) \times \exp(\theta_{m-2} \cdot \sigma) \times \exp(\theta_{m-3} \cdot \alpha_{max})\right) \pm \varepsilon_{CSR_{max}}$$
(5 - 17)

Then the likelihood functions can be written respectively as:

$$L_{CSR_{SSEI,max}}(\boldsymbol{\theta}, \boldsymbol{\sigma}_{\varepsilon}) = \prod_{i=1}^{n} \varphi \left[\frac{\hat{g}_{CSR_{SSEI,max}}\left(\boldsymbol{\sigma}, \frac{\boldsymbol{S}_{A}}{\boldsymbol{PGA}}, \frac{\boldsymbol{h}}{\boldsymbol{B}}, \boldsymbol{\tau}_{b}, \boldsymbol{\tau}_{soil}, \boldsymbol{\theta}\right)}{\boldsymbol{\sigma}_{\varepsilon_{CSR_{SSEI,max}}}} \right]$$
(5 - 18)

and

$$L_{CSR_{SSEI,max}}(\theta,\sigma_{\varepsilon}) = \prod_{i=1}^{n} \varphi \left[\frac{\hat{g}_{CSR_{SSEI,max}}(CSR_{rep},\sigma,\alpha_{max},\theta)}{\sigma_{\varepsilon_{CSR_{SSEI,max}}}} \right]$$
(5 - 19)

As part of maximum likelihood methodology, the coefficients which are estimated to maximize the likelihood functions given in Equations (5 - 18) and (5 - 19) are presented in Table 5.5-2 and Table 5.5-3. Final equations for CSR_{SSEI,max} are presented in Equations (5 - 20) and (5 - 21) determined from all variables and determined from CSR_{SSEI, rep} respectively.

$$CSR_{SSEI,max} = \frac{1.92 \cdot \exp\left(\frac{S_A}{PGA}^{-0.37}\right) \times \exp\left(\frac{h}{B}^{-0.122}\right) \cdot \tau_b + \tau_{soil}}{\sigma_{SSI}}$$
(5 - 20)

$$CSR_{SSEI,\max} = 1.09 \cdot CSR_{SSEI,rep} \times \exp(-0.017 \times \sigma) \times \alpha_{\max}^{0.004}$$
(5 - 21)

Table 5.5-2. Structural-induced maximum CSR_{SSEI} model coefficients

$\theta_{M-\sigma 1}$	$\theta_{M-\sigma 2}$	θ_{M-S1}	θ_{M-S2}	θ_{M-H1}	σ_{ϵ}
2.88	-0.014	1.92	-0.37	-0.122	0.1

Table 5.5-3. Structural-induced maximum CSR_{SSEI} model coefficients obtained from CSR_{SSEI,rep}

θ _{M-1}	θ_{M-2}	θ_{M-3}	σ_{ϵ}
1.09	-0.017	0.004	0.07

A comparison of the recommended and calculated maximum cyclic stress ratio values is presented in Figure 5.5-6 and Figure 5.5-7 with using the fixed base natural period of the structural system. These figures express a reasonably well relationship between the modeled and calculated values. The solid line is the 1:1 line and the dashed lines show the border of 2:1 and 1:2 predictions as before. It should be noted that predicting CSR_{SSEI,max} by using CSR_{SSEI,rep} values produce better predictions.



Figure 5.5-6. Comparison of CSR_{SSEI, max} values calculated as a result of numerical analyses and proposed formulation



Figure 5.5-7. Comparison of CSR_{SSEI, max} values (from CSR_{SSEI,rep})

5.6 CONCLUDING REMARKS

In this chapter, a simplified procedure for the assessment of soil, structure and earthquake interaction induced shear stresses is proposed for potential use in liquefaction triggering evaluation of foundation soils As part of this scope, two complementary definitions of CSR values are introduced: one representing the average (CSR_{SSEI,rep}) and one for expressing the maximum (CSR_{SSEI,max}) foundation soil response. It is concluded that that these values are highly interdependent and can be reliably expressed as a function of SSEI parameters, of ' σ ' (Veletsos and Meek, 1974), 'S_A/PGA' (Rollins and Seed, 1990) and 'h/B' (aspect ratio).

Simplified methodologies for the estimation of these representative and maximum CSR_{SSEI} values were developed within a probabilistic framework, more specifically by using maximum likelihood methodology. Resulting formulations for representative and maximum CSR values are presented in Equations (5 – 15), (5 –

20) and (5 - 21) respectively. Verification of these calculated CSR_{SSEI} values with the well documented case histories after 1999 Turkey earthquakes, centrifuge and shaking table test results will be presented in the next chapter.

CHAPTER 6

VALIDATION OF THE PROPOSED SIMPLIFIED PROCEDURE

6.1 INTRODUCTION

Although a satisfactory fit was achieved among the CSR estimations by numerical seismic response analysis results and the proposed simplified procedure, validation of the proposed procedure further with available laboratory shaking table and centrifuge tests and well-documented field case histories is preferred. For this purpose, centrifuge and shaking table test results of soil and structure models were studied. Fifteen centrifuge test results were concluded to be suitable for validation purposes. A summary of available laboratory tests and field case history data that was used for the purpose is presented in Table 6.1-1 and Table 6.2-1.

Table 6.1-1. Summary of the available laboratory tests

Type of Test	Name of the Excitation	Name of the Structure	Reference
Centrifuge Test	EQ1	BG-01	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ2	BG-01	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ3	BG-01	Ghosh and Madabhushi (2003)

Type of Test	Name of the Excitation	Name of the Structure	Reference
Centrifuge Test	EQ4	BG-01	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ3	BG-03	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ4	BG-03	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ5	BG-03	Ghosh and Madabhushi (2003)
Centrifuge Test	EQ1	BM1 (SDOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ2	BM1 (SDOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ3	BM1 (SDOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ4	BM1 (SDOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ1	BM2 (2DOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ2	BM2 (2DOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ3	BM2 (2DOF)	Mitrani and Madabhushi (2006)
Centrifuge Test	EQ4	BM2 (2DOF)	Mitrani and Madabhushi (2006)
Shaking Table	-	-	Rollins and Seed (1990)

Table 6.1-1. Summary of the available laboratory tests (cont'd)

Centrifuge models include either a rigid or a single/two degree of freedom structural element located on loose saturated sands shaken by acceleration amplitudes in the range of 0.08 to 0.32 g. The performances of the residential structures from Adapazari and Duzce, which were well documented and studied after 1999 Turkey earthquakes, were used to assess the reliability of the proposed procedure. These case history field performance data cover a wide range of soil and structure conditions as shown in Table 6.2-1.

6.2 CENTRIFUGE TESTS

For the purpose of verifying the proposed model, centrifuge tests aiming to assess seismic soil-structure interaction problems were carefully studied. Table 6.2-2 presents a summary of the centrifuge test models studied. Among these models, centrifuge tests performed at University of Cambridge, Engineering Department on structural systems resting on potentially liquefiable saturated soils with stress and pore pressure transducers located at points of interest are extremely valuable. These tests were performed with different types of structures (rigid, single degree of
	Nome of the	Turno of			Structu	Observed			
SPT TEST	Building	Soil	EQ*	В	Ĺ	Н	Period	Sa	Settlement
	_			(m)	(m)	(m)	(sec)	(g)	(cm)
SPT-A1	A1	ML-CH	EQ1	9.90	11.9	14.0	0.36	0.86	10.0
SPT-A3	A2	CH-ML	EQ1	13.70	17.0	14.0	0.36	0.75	50.0
SPT-B1	B1	SP-SM	EQ 1	5.10	20.0	14.0	0.36	0.86	20.0
SPT-B2	B2	SP-SM	EQ 1	6.00	23.4	14.0	0.36	0.86	0.0
SPT-C1	C3	ML-SP	EQ 1	19.50	20.0	14.0	0.36	0.75	17.0
SPT-C2	C1	CH-MH	EQ 1	19.50	20.0	14.0	0.36	0.75	17.0
SPT-C4	C2	CL-ML	EQ 1	19.50	20.0	14.0	0.36	0.86	20.0
SPT-D1	D1	SW	EQ 1	9.8	11.0	14.0	0.36	0.75	60.0
SPT-E1	E1	SP-SM	EQ 1	12.0	17.0	14.0	0.36	0.75	20.0
SPT-F1	F1	ML-CL	EQ 1	7.5	13.0	11.2	0.31	0.90	50.0
SPT-H1	H1	CH-CL	EQ 1	10.5	14.5	11.2	0.31	0.79	18.0
SPT-H1	H1	CH-CL	EQ 1	9.0	18.0	11.2	0.31	0.90	2.0
SPT-I1	l1	ML-SP	EQ 1	9.0	18.3	11.2	0.31	0.90	10.0
SPT-I1	12	ML-SP	EQ 1	13.3	14.9	16.8	0.41	0.91	12.5
SPT-I1	13	ML-SP	EQ 1	14.9	14.9	16.8	0.41	0.91	15.0
SPT-J1	J3	SM-ML	EQ 1	22.2	24.6	14.0	0.36	0.75	20.0
SPT-K1	K2	ML-SP	EQ 1	12.6	35.6	14.0	0.36	0.75	20.0
SPT-K1	K2	ML-SP	EQ 1	12.6	35.6	14.0	0.36	0.86	35.0
SPT-L1	L1	ML-SM	EQ 1	19.1	22.1	14.0	0.36	0.86	20.0
CSK-3	CASE-3	ML-MH	EQ 2	11.0	18.0	11.2	0.31	0.90	16.0
CSK-3	CASE-4	ML-MH	EQ 2	9.4	18.0	16.8	0.41	0.79	60.0

 Table 6.2-1. Summary of the available case histories after 1999 Turkey earthquakes

SPT TEST	Name of the Building	Type of Soil	EQ*		Structu	iral Pro	perties		Observed Settlement (cm)
CPT-TIGCI-3	TIGCILAR DIST.	ML	EQ 2	12.0	14.0	11.2	0.31	0.90	42.0
TSK-2	CASE-1	ML	EQ 2	8.9	24.6	15.4	0.39	0.91	21.0
SPT-SH4	SAPANCA HOT.	SW-SM	EQ 2	30.0	55.0	9.0	0.26	0.83	15.0
SPT-C5	C2	CL-ML	EQ 2	19.5	20.0	14.0	0.36	0.75	17.0
SPT-C6	C2	ML	EQ 2	19.5	20.0	14.0	0.36	0.75	17.0
SPT-C1	C4	ML-SP	EQ 2	23.4	24.0	14.0	0.36	0.75	17.0
SPT-D2	D1	ML-SW	EQ 2	9.8	11.0	14.0	0.36	0.86	40.0
SPT-E1	E3	SP-SM	EQ 2	15.0	21.6	14.0	0.36	0.75	25.0
CPT-A5	A2	SM-ML	EQ 2	13.7	17.0	14.0	0.36	0.86	50.0
CPT-C5	C3	SM	EQ 2	19.5	20.0	14.0	0.36	0.86	17.0
CPT-C5	C4	SM	EQ 2	23.4	24.0	14.0	0.36	0.86	17.0
CPT-E3	E1	SM-SW	EQ 2	12.0	17.0	14.0	0.36	0.86	20.0
CPT-E3	E3	SM-SW	EQ 2	15.0	21.6	14.0	0.36	0.86	25.0
CPT-CUM3	ORNEK APT	SM-ML	EQ 2	18.0	29.0	16.8	0.41	0.79	57.0
CPT-CUM3	CASE-4	SM-ML	EQ 2	10.2	17.5	11.2	0.31	0.79	0.0
CPT-CUM3	CASE-5	SM-ML	EQ 2	9.5	17.5	11.2	0.31	0.79	2.0
CPT-TIGCI-1A	CASE-3	SM	EQ 2	7.5	10.9	11.4	0.31	0.79	20.0
CPT-TIGCI-1A	CASE-4	SM	EQ 2	9.3	10.9	11.4	0.31	0.79	24.0
CPT-TIGCI-1A	CASE-5	SM	EQ 2	11.8	12.5	8.4	0.25	0.70	15.0
CSK-3	KAYINOGLU A.	ML	EQ 2	11.0	19.4	14.0	0.36	0.75	15.0
SK-02	CASE-I	ML	EQ 2	17.6	25.3	11.2	0.31	0.79	40.0

 Table 6.2-1. Summary of the available case histories after 1999 Turkey earthquakes (cont'd)

* EQ1 indicates 1999 Kocaeli earthquake and EQ2 indicates 1999 Duzce earthquake.

Table 6.2-2. Summary of the centrifuge test models

	BM1	BM2	BG-01	BG-02	BG-03
Layer thickness (m)	14.3	14.5	8.0	8.0	8.0
(dry (kN/m3)	14.9	14.9	-	-	-
(sat (kN/m3)	19.1	19.1	-	-	-
DR (%)	59	58	54	55	54
Type of structure	SDOF	2DOF	Rigid	Rigid	Rigid
Period of structure (s)	0.08	0.16	~0	~0	~0

(a) Test scheme and structural properties

(b) Earthquake properties

	Frequency (Hz)	Dı	uration	(s)	Max.	Base Ac	cc. (g)	No. of Cycles					
		BM1	BM2	BM3	BM1	BM2	BM3	BM1	BM2	BM3			
EQ1	14.3	27	27	27 27		0.09	0.06	17	17	17			
EQ2	14.9	28	27	27	0.13	0.14	0.14	22	22	22			
EQ3	19.1	29	27	30	0.20	0.19	0.22	28	27	30			
EQ4	59	29 27		35	0.27	0.22	0.32	28	27	35			

freedom structures, (SDOF), and two degree of freedom structures, (2DOF)) founded on different types of soils (layered or homogeneous) with different levels of excitations (either harmonic or not). Details of the tests can be found in Ghosh and Madabhushi (2003) and Mitrani and Madabhushi (2006). Pore pressure readings were reported at three different depths and at different locations beneath the structures. Verification procedure is based on the comparisons of the measured pore pressure response vs. the predicted pore pressures with the simplified procedure. A detailed description of the procedure followed will be presented in the following sections. Before then, a review of the performed centrifuge tests will be presented including the calculation of pore pressure ratios.

6.2.1 A Review on the Centrifuge Test Results

A series of centrifuge tests were performed at University of Cambridge, Department of Engineering, Civil, Structural, Environmental and Sustainable Development, Geotechnical Group. Four different sets of test models were used in this study: i) nearly rigid structures founded on homogeneous soil profile, ii) nearly rigid structures founded on layered soil profiles, iii) single degree of freedom structures founded on homogeneous soils and iv) two degree of freedom system founded on homogeneous soils. These set-ups have been shaken by different earthquake records. A schematic view of the centrifuge model set-up is presented in Figure 6.2-1.



Figure 6.2-1. Instrumentation and layout of BG-01 (Ghosh and Madabhushi, 2003)

Table 6.2-2 (a) summarizes some of the important characteristics of the test setup as well as monitoring details. On the basis of the test results, it was concluded by the researchers that pore pressures never reach the free field values underneath the structures. This is reported to be due to static shear stresses which lead to the

formation of a dilation zone underneath the foundation, which in turn inhibits the rise of excess pore pressures to the free field values. In one or two degree freedom systems, it was concluded that structural form has a significant effect upon seismic response. If material strengths are relatively small and natural modes of vibration dominate the response, elastic response spectra concepts are able to predict peak structural response to a reasonable degree of accuracy.

6.2.2 Calculating Pore Pressure Ratios in Centrifuge Tests

For comparison purposes, pore pressure ratios obtained from centrifuge tests have been calculated. The measured and reported pore pressures are divided by the effective stress at the point of interest as described in Equation (6 - 1).

$$r_u = \frac{u - u_{hydrostatic}}{\sigma_v'} \tag{6-1}$$

where

r_u : pore pressure ratio

u : pore pressure obtained during the test at the point of consideration
 u_{hydrostatic} : hydrostatic pore pressure at the point of consideration

 σ_{v}' : vertical effective stress at the point of consideration

The pore pressure ratio values (r_u) are calculated by using Equation (6 – 1). $r_u = 0$ means no excess pore water pressure generation and $r_u = 1$ means full liquefaction. A recorded excess pore pressure field is shown in Figure 6.2-2. X-axis in this figure represents the location of the pore pressure transducer, where "x" is the distance from the center of the foundation and "B" is the width of the foundation, i.e. x/B = 0 defines the geometric centerline of the structure, $x/B = \pm 0.5$ are the corners. On the y-axis, r_u values as estimated by Equation (6 – 1) are shown. The squares in this graph show the locations of the pore pressure transducers. As the figure implies two

of these measurements were available just under the two corners of the structure $(x/B = \pm 0.5)$ and the last one was taken at a distance x/B = 2.0 away from the centerline.



Figure 6.2-2. Variation of r_u beneath the foundation

6.2.3 Calculating Pore Pressure Ratios from Predicted CSR_{SSEI}

After calculating the pore pressure ratios based on centrifuge test pore pressure measurements, the next step involves the prediction of these pore pressure ratios by using proposed simplified procedure. For this purpose, $CSR_{SSEI,rep}$ values were estimated as defined in Chapter 5. These CSR_{SSEI} values were then corrected for the duration (number of cycles) of shaking and K_{α} and K_{σ} effects. These corrected values were used to estimate pore pressure response. Calculation steps are summarized in Figure 6.2-3.

Step 1 in Figure 6.2-3 is the hearth of this study and has been defined in detail in Chapter 5. The fourth and the last step is the prediction of pore pressure ratio. In the literature, the generation of pore pressure was attempted to be assessed by 3 fundamentally different models: i) stress based models (e.g. Lee and Albaisa (1974), Seed et al. (1975) and Booker et al. (1976)); ii) strain based models (e.g. Martin et al.

(1975) and Byrne (1991)) and iii) energy based models (Berrill and Davis (1985) and Green et al. (2000)). In this study due to benefiting from widely used CSR definitions, stress-based models were preferred. Among the stress-based models, Booker et al. (1976)'s model was used for its simplicity. It is a two-parameter effective stress based model and reported to be one of the first attempts to model effective stresses in cohesionless soils under cyclic loading. In stress based models, it is assumed that two kinds of pore pressures are generated in soils during shaking: i) transient pore pressure and ii) residual pore pressure. Transient means pore



Figure 6.2-3. Steps in pore pressure ratio (r_u) calculations

pressures are equal to the changes in the applied mean normal stress and have little influence on the average effective stress changes in the soil while residual pore pressure can exert major influences on strength and stiffness of the sand. Booker et al. (1976) have proposed the equation given below for predicting pore pressure ratio, r_u .

$$r_{u} = \frac{2}{\pi} \cdot \sin^{-1} \left[\left(\frac{N_{cyc}}{N_{liq}} \right)^{\frac{1}{2}\theta} \right]$$
(6-2)

where N_{cyc} is the number of cycles that the sample is subjected to, N_{liq} and θ are two calibration parameters. θ is an empirical constant depending on soil type and testing conditions. Here we need to calculate N_{liq} , which is the number of cycles required to cause liquefaction and depends on relative density and confining stress of the soil. For this purpose, prediction of the equivalent moment magnitude corresponding to the number of cycles of loading applied during the centrifuge test is necessary. Inspired by the cyclic resistance ratio equation in Cetin et al. (2004), magnitude corresponding to a certain CSR value can be calculated as presented in Equation (6 – 3).

$$M_{w} = \exp\left[\frac{\binom{N_{1,60} \cdot (1 + 0.004 \cdot FC) - 3.70 \cdot \ln(\sigma_{v}') + 0.05 \cdot FC}{-13.32 \ln(CSR) + 44.97}}{29.53}\right]$$
(6-3)

 $N_{1,60}$ is the corrected SPT-N blow-count, FC is the fines content in percent, M_w is the moment magnitude of the earthquake, σ_v ' is the effective vertical stress. CSR_{SSEI} value is substituted into this equation as CSR term and M_w can easily be calculated for known parameters. As a next step, equivalent number of cycles corresponding to this magnitude should be calculated so that pore pressure ratio can be predicted. In this study, methodology proposed by Liu et al. (2001) has been utilized. In their study, seismic demand of potentially liquefiable soils was approximated by a series of uniform stress cycles by means of empirical regression equations as a function of magnitude, site source distance, site conditions and near fault directivity effects. The number of cycles of an earthquake magnitude "m" is calculated as given below:

$$\ln(N_{cyc}) = \ln\left[\frac{\left(\frac{\exp(b_1 + b_2(M_w - m^*))}{10^{1.5M_w + 16.05}}\right)^{-\frac{1}{3}}}{4.9 \cdot 10^6 \beta} + S[c_1 + c_3(r - r_c)] + c_2(r - r_c)\right] + \varepsilon (6 - 4)$$

where

N_{cyc} : the number of equivalent harmonic cycles,

 β : the shear wave velocity at the source in km/s and taken as 3.2 km/s,

 M_w : moment magnitude of the earthquake,

- *S* : a constant equal to 0 for rock sites and 1 for soil sites,
- *r* : the distance from source in kilometers,
- r_c : cutoff distance in kilometers to be determined by regression,
- ϵ : normally distributed residuals with mean zero and standard deviation σ ,

 b_1 , b_2 , c_1 , c_2 and m^* : coefficients determined by regression and estimated as presented in Table 6.2-3.

Separate regressions were performed with and without c_3 term and due to negligible changes in error, this term was dropped permanently. r_c value was taken as zero for calculations.

Coefficient	Regression Estimate
b ₁	1.53 ± 0.15
b ₂	1.51 ± 0.12
c ₁	0.75 ± 0.42
c ₂	0.095 ± 0.014
m*	5.8

Table 6.2-3. Average Regression Estimates of Coefficients (Liu et al., 2001)

6.2.4 Comparison of Pore Pressure Values

After reviewing pore pressure generation methods in the literature, the stress-based model proposed by Booker et al. (1976), i.e. Equation (6 - 2) was decided to be used to verify the proposed model. The number of cycles which the soil body is exposed to was reported as part of the centrifuge test reports. Number of cycles causing liquefaction has been calculated as recommended by Cetin et al (2004) by using the moment magnitude vs. number of cycles relationships of Liu et al. (2001). The constant θ is assumed as 0.7, which is the average value proposed by Booker et al. (1976).

A summary of the calculation steps is presented in Figure 6.2-4. Steps presented on the left hand side of the flowchart summarize the steps followed for estimating pore pressure ratio from centrifuge test results and the right hand side shows the pore pressure calculation steps of the proposed model. Having estimated all the input parameters and variables, pore pressure ratios are calculated by using Equation (6 – 2). Comparisons of the pore pressure ratios are shown in a graphical form in Appendix A. As can be seen from these graphs, trends in pore pressure ratio generations showed very good agreement between the predicted and the measured values.



Figure 6.2-4. Steps in comparing pore pressure ratios (r_u)

A representative plot showing the comparison between predicted and measured pore pressure ratios can be seen in Figure 6.2-5 respectively for depths 2.00, 3.50 and 6.50 m for the case BQ01 – EQ01 whose model set-up was presented earlier in Figure 6.2-1. Figure 6.2-5 shows the goodness of the match.

At shallow depths, foundations soils are concluded to be not liquefied (i.e. $r_u < 1.0$) and r_u values were reported to be less than 0.5. There is a significant difference in



Figure 6.2-5. Comparison of r_u between centrifuge tests and the one found from prediction (BG01 – EQ1, d/B = 0.67, 1.17 and 2.17 m respectively)

the magnitude of recorded and estimated pore pressures values at these shallow depths. This may be due to i) uncontrollable increased permeability of the centrifuge model just beneath the foundation level, ii) difficulties in the saturation of surfacial pore pressure transducers, iii) slip of the foundation mat on shallow foundation soils, reducing structural-induced shear stresses. At d/B = 1.17, pore pressure ratio increases to 0.5 and converges to a value of 1.0 at and beyond the depth of d/B = 2.17. Consistent pore pressure field trends at shallow depths (i.e.: $d/B \le 0.67$), and the almost perfect match in both magnitudes and trends at depths deeper than d/B > 0.67 are concluded to be promising and mutually supportive.

6.3 SHAKING TABLE TEST

Many investigators have performed shaking table tests to simulate the behavior of foundation soils during cyclic loading. An example to this can be seen in Figure 6.3-1. A shaking table box of 65 cm in width and 30 cm in height was shaken by an acceleration of 0.1 g. then pore pressure ratios were recorded as illustrated in Figure 6.3-1. These pore pressure ratios, similar to the others in the literature, show that building has a positive influence on the liquefaction potential of the soil: the soils under the structure generate a pore pressure of 20% whereas the pore pressure ratio increases up to 100% at "free field" at equivalent depth.

For validation of the proposed methodology with these test results, it is attempted to calculate the pore pressure ratios similar to the centrifuge tests presented in the preceding section. However, this was not possible due to missing data such as the magnitude of earthquake. For this reason, CSR values have been calculated and compared at certain depths and presented in Figure 6.3-2. In this figure x/B = 0.5 is the corner of the structure similar to the previous graphics.



Figure 6.3-1. Measured excess pore pressure ratio development (a) Early Stage and (b) Later Stage (Yoshimi and Tokimatsu 1978, from Rollins and Seed 1990)



Figure 6.3-2. Variation of CSR at d/B = 0.6 and d/B = 1.0

Figure 6.3-2 implies that CSR values are lower beneath the foundation and they increase with distance away from the structure, which means that less pore pressure will develop beneath the foundation. This is consistent with the test result available. Additionally, although there is a small difference, with depth, CSR increases under

the structure and decrease at free field which also shows a similar trend with the shaking table test results.

6.4 FIELD OBSERVATIONS FROM PAST EARTHQUAKES

Based on post earthquake reconnaissance, especially after 1999 Kocaeli and Düzce and 2000 Chi-Chi earthquakes, following observations were made by various researchers: i) sand boils were usually observed at the edges of some structures where as no sand boils were observed at free field soil sites with similar soil profiles, (Figure 6.4-1), ii) structures located at the end of closely spaced residential building series are more vulnerable to liquefaction-induced bearing capacity loss and corollary tilting (Figure 6.4-2). Thus for the purpose of validation, even though it may be perceived as quite weak, it is intended to use the proposed simplified model to check if above summarized observations could have been predicted a priori.



Figure 6.4-1. Sand boils are usually observed at the edges of structures (photos from nieese.berkeley.edu)



Figure 6.4-2. Structures located at the corners are more vulnerable (photos from peer.berkeley.edu)

6.4.1 Effects of Adjacent Structures

In urban areas, closely located structures may significantly affect the seismic response. Weight and base shear of the adjacent structures are the major interaction reasons from static loading point of view. On the other hand, rocking and sliding response of the adjoining structures can be listed as the other modes of interaction. However, until now, numerical analyses were performed for single structures. To assess the effects of the adjacent structures, 3-D numerical analyses were performed for a case of three adjacent structures which were 4.0 m apart. The structures modeled are assumed to be identical. Buildings were chosen to be 4-storey height having a width of 12 m. Figure 6.4-3 shows a schematic view of the model. 3-D model was shaken by 1999 Kocaeli earthquake, Sakarya Station (SKR) record.



Figure 6.4-3. Schematic model for the finite difference model for adjoining structures

Figure 6.4-4 shows the calculated and the predicted results for depths of 1.50, 2.50, 3.50, 5.50, 7.50, 10.50 and 15.00 m respectively. The shaded areas in the figures show the locations of the buildings. The solid lines are the FLAC-3D results and the dashed lines are the prediction of the proposed methodology. As expected CSR values have their highest values in the exterior corners of the structures. Existence of the exterior buildings has a positive effect on the liquefaction potential of the building in the middle. However, the same is not true for the exterior ones as differential settlements are expected to occur which may result in collapse of the structures. Predicted and the calculated values show a similar trend and their numerical values are nearly the same.



Figure 6.4-4. Comparison between FLAC-3D results and prediction for three adjacent structures

The same equations and the framework originally developed for the single structure model was used with no modifications on the proposed simplified model. The differences lie not in the framework but in stress fields. Close agreement among i) field observation, ii) 3-D numerical simulations and iii) the simplified procedure predictions are concluded to be strongly supportive and promising.

6.5 VALIDATION THROUGH FOUNDATION PERFORMANCE CASE HISTORIES

Well-documented foundation performance case histories of residential structures founded on liquefiable soils after 1999 Kocaeli earthquake were used to assess liquefaction potential of foundation soils. The foundation soil profiles of these case histories generally consist of silty soils, sand-silt mixtures and silt-clay mixtures. Overburden and procedure corrected SPT-N values vary in the range of 2 to 5 blows/30 cm in the upper 5 meters and gradually increases up to a maximum value of 25 blows/30 cm beyond depths of 5 to 8 m's. Overlying structures are mainly 3 to 4 storey, residential buildings with no basements. The structures were composed of frame elements of beams and columns. Foundation systems were either documented or assumed to be mats. Calculation steps followed for the liquefaction triggering assessment of foundation soils will be discussed next.

6.5.1 Site A

One of the cases studied after 1999 Kocaeli Earthquake was called Site A, located in Cumhuriyet District, Adapazari, between Telli and Yakin Streets. The geographical coordinates of this site is N40.78 E30.39. A general overview for this case is presented in Figure 6.5-1. As can be seen from this figure, relatively closely located, two buildings rest on this site: Buildings A1 and A2. They are both 5 storey residential buildings. The magnitudes of the settlements observed at the edges of the structures are shown in Figure 6.5-1. The black dots shown in Figure 6.5-1 indicate the locations of the field tests performed including four standard penetration (SPT) and six cone penetration tests (CPT). For illustration purposes, SPT – A1 and CPT – A1 are shown in Figure 6.5-2 and Figure 6.5-3, respectively. Figure 6.5-4 and Figure 6.5-5 show after earthquake appearance of the buildings.



Figure 6.5-1. A general view of Site A

	UCB-BYU-UCLA Project Name: Ground Failure and Building Performance in Adapazari, Turkey ZETAS-SaLI-METU Location: Sile A - Tul and Yakin Streets,Cumhurlyet District, Adapazari Joint Research Date: June 23, 2000 Joint Research Date: June 23, 2000 Sponsored by: Doparator: ZETAS (Zernin Teknolojisi, A. S.) NSP, Catexas Drilling Method: Rotary wash with 9 cm-diameter tricone bit Wafer Table Elevation: CeVL-0.90m 06/26/4000. 0.77m 06/26/00, 0.77m 06/26/00 Notes: Solid flight auger used to a depth of 1.5 m													iey Test ID: SPT-A1 GPS Coordinate: 40.77922*N 30.39487*E Elevation: -0 cm with respect to CPT-A3 Drilling Equipment: Custom made, equivalent to Creatius XC90H Responsible Engineers: J. D. Bray and R. B. Sanco, U. C. Berkeley SPT System: Rope, pulley and cathead method. AWJ rods. Hammer Type: Safety Hammer (per Kovacs et al. 1983)								
Depth Scale (m)	Lithology	nscs	Sample Type and No.	Recorrery/ Length (cm)	SPT Blows/15 cm	Casing Depth (m)	Rod Length (m)	Energy Ratio (%)	Description	9u PocketPen (kPa)	su Torrane (kPa)	Moisture Content (%)	Liquid Limit	Plasticity Index	% fines < 75 µm	< 5 µm (%)	< 2 µm (%)	D50 (mm)	D10 (mm)	Remarks		
Ē									ASPH: Boring performed through asphalt and subgrade of Tul street	•												
-1 	77	ML	S-A1-1	18/45	1-3-3		4.27	37	FILL: Materials transition from a brown to gray gravely sand to red silty clay of hard			38	41	13	90							
-2 2	\sim	MH/CH	S-A1-2	40/45	3-2-5		5.80	46	CH: Brown moist sticky biob		28	39	53	23	94							
-3	$\mathcal{L}\mathcal{L}$	сн	S-A1-3	31/45	2-3-4		5.80	42	plasticity sity clay without visible sand particles. S-A1-4 shows darker tones and some fine to medium sand content	140	50	39	65	35	100	61	36	0.0035	<2µm			
F	2	CL	S-A1-4	36/45	1-2-2		7.32	57		80	22	37	46	23	87							
4		ML/ML-	S-A1-5	40/45	2-2-2		7.32	53	ML: Gray slit with sand. Field description: ML	70	23	29	29	6	74	16	>10%	0.045	0.003			
-5	77	сн	S-A1-6	45/45	1-2-1		8.84	55	ML: Brown, low plasticity slit with fine sand and some red clay points	80	25	44	55	28	92							
-6	<u></u> ⊒ ∕ ∕	CL/ML	S-A1-7	39/45	1-1-2		8.84	50	CH: High plasticity gray clay with low sand content (traces). At 5.3 m a thin fine sand seam was identified. Sample A1-7 exhibits some sand seams	75	26	39	47	20	97	31	18	0.012	<2µm			
-7		ML	S-A1-8	37/45	6-6-9		10.37	65	ML: Gray sandy slit. Increasing sand content with depth	450		27	30		70	15	10	0.057	0.002			
8		ML	S-A1-9	41/45	6-9-10		11.89	75		275		27	29		58							
9		SP	S-A1-10	41/45	11-20-23		11.89	64	SP: Medium to fine poorly graded gray sand	300		24			5			0.29	0.12			

Figure 6.5-2. SPT Borelog (SPT-A1)



Figure 6.5-3. CPT Log (CPT-A1)



Figure 6.5-4. Appearance of Building A1 after earthquake (from point A1, photos from peer.berkeley.edu)



Figure 6.5-5. Appearance of Building A2 after earthquake (from point A6, photos from peer.berkeley.edu)

As the figures imply, an extreme tilting is observed in Building A1 and it has demolished after the earthquake. On the other hand, Building A2 settled relatively uniformly in the order of 40 to 60 cm's.

The assessment procedure including the estimation of simplified model input parameters will be discussed next, by illustrating the calculation details.

6.5.1.1 Model input parameters

The proposed simplified procedure, as the name implies, needs easy to estimate, yet powerful enough parameters to capture the observed response. As discussed earlier, these parameters are grouped as: i) structural, ii) geotechnical and iii) ground motion related. Since they were discussed in detail in Chapter 5, they won't be repeated herein. For the purpose of evaluating some earthquake related parameters, one dimensional equivalent linear seismic response analyses were performed. For every case history site, elastic response spectrum corresponding to 5% damping was determined as also presented in Figure 6.5-6 for Site A, 1999 Kocaeli Earthquake.



Figure 6.5-6. Response Spectrum for Site A after 1999 Kocaeli Earthquake

i) Geometric properties for Building A1:

Width, B	: 10 m
Length, L	: 12 m
Height, H	: 5 storeys, ~14 m
Period of the structure	: ~0.5 sec

ii) Properties for Site A (using SPT – A1)

Overburden corrected SPT-N blow-counts have been presented in Figure 6.5-2.

iii) Properties for the 1999 Kocaeli earthquake:

Moment magnitude, Mw	: 7.2
Peak ground acceleration, PGA	: 0.40 g
Spectral acceleration, SA	: 1.22 g (corresponding to structural
period of 0.5 seconds)	

6.5.1.2 Calculation of CSR_{SSEI,rep} and CSR_{SSEI,max}

The variables other than the ones listed above can be calculated using the simple parameters stated in the preceding section. Some of these values such as the vertical stresses and shear stresses vary depending on the depth, however the functions of $\frac{S_A}{PGA}$, $\frac{h}{B}$ and σ is same for a specific site and/or building. These constant values are listed as a table in Table 6.5-1 for representative and maximum cases respectively. Afterwards, Table 6.5-2 summarizes the steps for calculating CSR_{SSEI,rep} and CSR_{SSEI,max}.

 K_{α} values in Table 6.5-2 have been calculated using the chart proposed by NCEER 1997 (Figure 3 – 3.3). K_{σ} values were calculated after Boulanger and Idriss (2004) as previously defined in Equation (3 – 26).

	For CSR _{SSEI,rep}	For CSR _{SSEI,max}
$\frac{S_A}{PGA} = 3.05$	$f\left(\frac{S_A}{PGA}\right) = 0.10 *$	$f\left(\frac{S_A}{PGA}\right) = 0.62 *$
$\frac{h}{B} = 1.00$	$f\left(\frac{h}{B}\right) = 0.51^{**}$	$f\left(\frac{h}{B}\right) = 0.89 **$
$\sigma = 12.90$	$f(\sigma) = 12.76 * * *$	$f(\sigma) = 2.45^{***}$

Table 6.5-1. Constants for Building A1 of Case A

*
$$f\left(\frac{S_A}{PGA}\right) = 0.79 \times \exp\left(-0.68\frac{S_A}{PGA}\right)$$
 and $f\left(\frac{S_A}{PGA}\right) = 1.92 \times \exp\left(-0.37\frac{S_A}{PGA}\right)$ for

CSR_{SSEI,rep} and CSR_{SSEI,max} respectively.

** $f(\sigma) = 14.77 \times \exp(-0.01\sigma)$ and $f(\sigma) = 2.88 \times \exp(-0.014\sigma)$ for CSR_{SSEI,rep} and CSR_{SSEI,max} respectively.

$$f\left(\frac{h}{B}\right) = \exp\left(-0.66\frac{h}{B}\right)$$
 and $f\left(\frac{h}{B}\right) = \exp\left(-0.122\frac{h}{B}\right)$ for CSR_{SSEI,rep} and

 $CSR_{SSEI,max}$ respectively.

Depth (m)	N1,60	σ _v ' (Pa)	🤃 σ _b ' (kPa)	r _d (ii)	≘i τ _b (kPa)	<u>Ξ</u> τ _{soil} (kPa)	σ' _{v,SSI} (kPa) (v)	<u>(s</u>) CSR _{SSEl,rep}	(i) CSR _{SSEI,max}	α _{rep} (vii)	α _{max} (viii)	$K_{\alpha,rep}$	$K_{\alpha,max}$	Kσ	$\frac{\text{CSR}_{\text{SSEI,rep}}}{\overline{x}} \alpha = 0, \ \sigma = 100$	$\begin{array}{c} \text{CSR}_{\text{SSEI,max}}\\ \widehat{\mathbf{x}}\\ \alpha=0, \ \sigma=100 \end{array}$	(x) CSR _{FF}
0.78	10	6	75	0.99	37	3	70	0.62	0.94	0.15	0.50	0.82	0.65	1.67	0.45	0.87	0.33
1.93	11	15	75	0.99	30	8	71	0.56	0.81	0.12	0.33	0.86	0.65	1.70	0.38	0.74	0.32
2.70	9	22	75	0.98	26	11	73	0.53	0.74	0.09	0.25	0.89	0.70	1.64	0.36	0.64	0.32
3.50	6	28	75	0.97	23	14	75	0.51	0.69	0.07	0.20	0.87	0.64	1.58	0.37	0.69	0.32
4.33	5	35	75	0.97	20	17	78	0.49	0.64	0.06	0.15	0.90	0.73	1.55	0.35	0.57	0.31
5.23	4	42	75	0.96	17	21	82	0.48	0.61	0.04	0.11	0.92	0.80	1.51	0.34	0.50	0.31
6.20	3	50	75	0.95	15	24	86	0.47	0.58	0.03	0.08	0.94	0.86	1.49	0.33	0.45	0.30
7.20	20	58	75	0.94	13	28	91	0.46	0.55	0.03	0.05	1.00	1.00	1.94	0.24	0.28	0.30
8.20	28	66	75	0.94	12	31	96	0.46	0.53	0.03	0.04	1.00	1.00	2.27	0.20	0.23	0.29
14.0	42	112	75	0.89	7	51	132	0.44	0.47	0.01	0.00	1.00	1.00	2.99	0.15	0.16	0.27

 Table 6.5-2. Calculation Steps for Building A1 of Case A

(i) $\sigma_b = 15 \times Number \ of \ storeys = 15 \times 5 = 75$

(ii) For d < 20 m

$$r_{d} = \left[\frac{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (-d + 0.0785 \cdot V_{s,12m}^{*} + 7.586)}}}{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (0.0785 \cdot V_{s,12m}^{*} + 7.586)}}}\right]$$

For $d \ge 20 \text{ m}$

$$r_{d} = \left[\frac{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (-d + 0.0785 \cdot V_{s,12m}^{*} + 7.586)}}}{1 + \frac{-23.013 - 2.949a_{\max} + 0.999M_{w} + 0.0525V_{s,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \cdot (0.0785 \cdot V_{s,12m}^{*} + 7.586)}}}\right] - 0.0046(d - 20)$$

(iii)
$$\tau_b = 0.65 \frac{0.80 \cdot S_A \cdot \sigma_b \cdot (B \cdot L)}{(B+1.6 \cdot z) \cdot (L+1.6 \cdot z)}$$

(iv)
$$\tau_{soil} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \gamma \cdot z \cdot r_d$$

(v)
$$\sigma_{SSI}'(z) = \sigma'_{soil}(z) + \frac{(\sigma_{str} \times B \times L)}{(B + 0.9z) \cdot (L + 0.9z)}$$

(vi)
$$CSR_{SSEI} = \frac{f(\sigma) \cdot f\left(\frac{S_A}{PGA}\right) \cdot f\left(\frac{h}{B}\right) \cdot \tau_b + \tau_{soil}}{\sigma_{SSI}}$$

(vii)
$$\alpha_{rep} = \exp\left(\frac{z - 1.48}{-4.36}\right) \times \frac{T_{str}^{-0.12}}{10.03} \times N_{1,60}^{0.04}$$

(viii)
$$\alpha_{\text{max}} = \exp\left(\frac{z-5.87}{-2.63}\right) \times \frac{T_{str}^{0.28}}{11.08} \times N_{1,60}^{-0.01}$$

(ix)
$$CSR_{\alpha=0,\sigma=100kPa} = \frac{CSR_{\alpha,\sigma}}{K_{\alpha} \times K_{\sigma}}$$

(x)
$$CSR_{FF} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d$$

6.5.2 Interpretation of the Results for Case A

Table 6.5-2 summarized the results for a case history after 1999 Kocaeli earthquake. As can be seen from this table, representative values of CSR_{SSEI} are higher than the maximum values of CSR_{SSEI} which points out that the edges of the structure is more critical than the middle portions. This difference in cyclic stress values indicates that the structures have a tilting potential which is valid for most of the cases in Adapazari after the earthquake.

Similarly, free field cyclic stress ratio values are less than the representative ones which indicates that liquefaction potential beneath the buildings is higher than the free field liquefaction potential. This situation is also valid for the cases in Adapazari, i.e. although the free field did not liquefy after the earthquake; the foundation soils liquefied and caused structural failures.

6.5.3 Deformation Analysis for Case Histories

Table 6.2-1 summarized the observed settlements for the case histories after 1999 Kocaeli and Duzce earthquakes. In this section, a summary of the calculations of the deformations at the foundation soils documented after 1999 Kocaeli and Duzce earthquakes using these structural-induced cyclic stress ratio values will be presented briefly. The cases include more than 15 sites and over 40 different buildings from Adapazari and Duzce.

The deformations under the structures were calculated by using the procedure presented in Cetin et al. (2009). They have described a maximum likelihood

framework for probabilistic assessment of post-cyclic straining of saturated clean sands and compiled a large number of data from literature as well as a series of stress controlled cyclic triaxial and simple shear tests performed on laboratory constituted saturated clean sand specimens have been utilized. According to this procedure, the deviatoric and volumetric components of strain are calculated separately, multiplied with the corresponding layer thickness and then added up to find the total deformation in the soil profile. Definitions of the shear strain and volumetric strain are given in Equations (6 - 5) and (6 - 6) respectively.

$$\ln(\gamma_{\max}) = \ln\left[\frac{-0.025 \cdot N_{1,60,CS} + \ln(CSR_{SS,20,1-D,1atm}) + 2.613}{0.001 \cdot N_{1,60,CS} + 0.001}\right]$$
(6-5)

$$\ln(\varepsilon_{v}) = \ln\left[1.879 \cdot \ln\left[\frac{780.416 \cdot \ln(CSR_{SS,20,1-D,1\,atm}) - N_{1,60,CS} + 2442.465}{636.613 \cdot N_{1,60,CS} + 306.732}\right] + 5.583\right]$$
(6 - 6)

In these equations, γ_{max} and ε_v represent the maximum double amplitude shear strain and post-cyclic volumetric strain respectively both of which are in percent, N_{1,60,CS} is the overburden and energy corrected SPT-N value for clean sands, CSR_{SS,20,1-D,1atm} is the CSR value corresponds to 1 dimensional, 20 uniform loading cycles, under a confining pressure of 100 kPa (=1 atm). Details of the calculation of these corrected CSR_{SS,20,1-D,1atm} and N_{1,60,CS} values can be found in Cetin et al. (2009). In this study, an extra correction factor for depth is introduced to this formulation. Strain values from the upper layers of soil profile are added with a higher weighting factor and the effect of strains diminish with depth. The formulation for this weighting factor is presented in Equation (6 – 7) and (6 – 8) for volumetric and deviatoric strains respectively.

$$WF_{vol} = 1 - \theta_{vol} \cdot \left(\frac{d}{B}\right) \tag{6-7}$$

$$WF_{dev} = 1 - \theta_{dev} \cdot \left(\frac{d}{B}\right)$$
 (6-8)

Where θ_{vol} and θ_{dev} are found to be 0.0 and 0.65 respectively as result of regression analyses. In the regression analyses, the cases with a settlement higher than 40 cm are not taken into account. In this formulation d is the depth from the ground surface and B is the width of the structure. The values of weighting factors, θ_{vol} and θ_{dev} , present that the effect of deviatoric strains on the total settlement diminishes at z/B < 2. However, volumetric strains effect continues beyond this depth. Table 6.5-3 presents a summary of the calculation procedure for Site AA1 and Building A1.

Total volumetric settlement is calculated by multiplying NF (Column 27) with the sum of Column 23. Total deviatoric settlement is the sum of Column 24. Then the two components are added up with the weighing factors 0.77 and 0.01 for volumetric and deviatoric components respectively.

Volumetric Component = $0.15 \times 0.73 = 0.1095m = 10.95$ *cm*

Deviatoric Component = 1.22m = 122 cm

 $Settlement = 0.77 \times Volumetric + 0.01 \times Deviatoric = 0.77 \times 10.95 + 0.01 \times 122 = 9.7 cm$

Figure 6.5-7 shows the comparison of the displacements calculated using formula mentioned above and the observed settlements. The solid line in this figure is 45° line (1:1) and the dashed lines are 1:2 and 2:1 lines. This graph shows that the estimated settlements match with the observed settlements in the range which can be counted as a good match from geotechnical earthquake engineering point of view. Red points in this figure show the cases which are kept outside the regression analyses.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27
								Rep.	CSR _s	SEI							,rep			rep						
SPT-N	Depth (m)	N _{1.60}	N _{1.60, CS}	a	S _A /PGA	H/B	f(σ)	f(S _A /PGA)	f(H/B)	$\tau_{\rm b}$	τ_{soil}	CSRrep	К _о	κ _α	£v,rep	WF _{vol}	Weighted ϵ_v	γ rep	WF _{dev}	Weighted γ	$\mathbf{AF}_{\mathrm{D,i}}$	S _{w,i} rep	$DEV_{w,i\;rep}$	Σt_i	$\Sigma t_{i}.AF_{i}$	NF
6	1.2	10	18	12.9	2.3	1.0	12.8	0.16	0.51	90	3	0.45	0.92	0.80	2.07	1.00	2.07	18.84	0.95	17.87	0.98	0.03	0.28	19.20	14.08	0.73
7	1.9	11	20	12.9	2.3	1.0	12.8	0.16	0.51	85	8	0.38	0.93	0.85	1.72	1.00	1.72	14.13	0.87	12.32	0.95	0.01	0.09			
7	2.7	9	18	12.9	2.3	1.0	12.8	0.16	0.51	82	11	0.36	0.94	0.88	1.91	1.00	1.91	16.15	0.82	13.25	0.93	0.01	0.11			
4	3.5	6	13	12.9	2.3	1.0	12.8	0.16	0.51	78	14	0.37	0.95	0.85	2.50	1.00	2.50	24.68	0.77	18.93	0.90	0.02	0.15			
4	4.3	5	11	12.9	2.3	1.0	12.8	0.16	0.51	75	18	0.35	0.96	0.88	2.80	1.00	2.80	29.73	0.71	21.18	0.88	0.02	0.18			
3	5.2	4	10	12.9	2.3	1.0	12.8	0.16	0.51	72	21	0.34	0.97	0.91	2.91	1.00	2.91	31.84	0.65	20.78	0.85	0.02	0.20			
3	6.2	3	9	12.9	2.3	1.0	12.8	0.16	0.51	69	25	0.33	0.97	0.93	2.98	1.00	2.98	33.11	0.59	19.46	0.83	0.02	0.19			
15	7.2	20	29	12.9	2.3	1.0	12.8	0.16	0.51	65	29	0.24	0.93	1.00	0.55	1.00	0.55	3.84	0.52	2.00	0.80	0.00	0.02			
19	8.2	28	37	12.9	2.3	1.0	12.8	0.16	0.51	63	34	0.20	0.90	1.00	0.00	1.00	0.00	0.53	0.45	0.24	0.77	0.00	0.00			
43	9.2	42	50	12.9	2.3	1.0	12.8	0.16	0.51	49	57	0.15	0.86	1.00	0.00	1.00	0.00	0.00	0.07	0.00	0.61	0.00	0.00			
																					SUM=	0.15	1.22			

 Table 6.5-3. Summary of the calculation procedure (performed for Site AA1 and Building A1)



Figure 6.5-7. Comparison of calculated and observed settlements for case histories after 1999 Turkey earthquakes

6.6 INTERPRETATION OF THE RESULTS

The sensitivity of the proposed methodology with respect to some parameters such as period of the structure, distance to the fault, magnitude of the earthquake has been checked in terms of structural-induced cyclic stress ratio and free field cyclic stress ratio (CSR_{SSEI}/CSR_{FF}). The sensitivity analyses have been performed for a range of structural periods ($T_{str} = 0.2$, 0.5 and 1 s) magnitude of earthquakes ($M_w = 5.5$, 6.5 and 7.5), soil stiffnesses, ($V_s = 80$ and 150 m/s), and finally closest distance to the fault rupture (d = 0, 50, 100 and 150 km). For this purpose, attenuation relationship by Abrahamson and Silva (1997) was used along with Boore et al. (1997) relationship for comparison purposes. The following results were obtained.

• Period of the structure significantly affects the liquefaction triggering response of foundation soils. As Figure 6.6-1 presents, for a structural period

of 0.2 seconds, foundation soils are concluded to be more vulnerable to liquefaction compared to the free field soil sites: i.e. $CSR_{SSEI}/CSR_{FF} \cong 2$. If structural period increases to 0.5 seconds, as indicated in the second line of the mentioned figure, the ratio of structural induces CSR to free field CSR decreases to about 1.1, which means that foundation soils are still more vulnerable to liquefaction compared to the free field soils, but not as much as the foundation soils of structures with a period of 0.2 seconds. However, for structures with a period of 1 second, the free field sites become more critical than the foundation soils, i.e. $CSR_{SSEI}/CSR_{FF} < 1$.



Figure 6.6-1. Sensitivity of the results with respect to T_{str} , M_w and distance to the fault rupture with Abrahamson and Silva (1997) and Boore et al., (1997)

- As also shown in Figure 6.6-1, neither the magnitude of the earthquake nor the distance to the fault rupture significantly changes the CSR_{SSEI}/CSR_{FF} value. This could be due to the fact that both the foundation and free field soils are affected from the magnitude and the distance in a similar manner.
- Figure 6.6-2 shows the variation in CSR_{SSEI}/CSR_{FF} for different soils sites. Even though CSR_{SSEI}/CSR_{FF} slightly increases with increase in V_s, stiffness of the site is concluded to be a relatively insignificant parameter.



Figure 6.6-2. Sensitivity of the results with respect to stiffness of the soil site

• As should be anticipated, the structural-induced and free field cyclic stress ratios should approach to each other with depth. Figure 6.6-3 clearly shows this trend and it can be concluded that the effect of the structure diminishes at depths beyond z/B > 1. For a structure with a period of 0.2 second, the

value of CSR_{SSEI}/CSR_{FF} decreases from about 2.0 at depth z/B = 0.0 to 1.0 at a depth z/B = 0.5. For the case of $T_{str} = 1.0$ second, CSR_{SSEI}/CSR_{FF} increases from about 0.5 at z/B = 0 to 0.92 at a depth z/B = 2.0.



Figure 6.6-3. Sensitivity of the results with respect to depth

• Figure 6.6-4 shows the variation of CSR_{SSEI}/CSR_{FF} with the uncertainty in the attenuation models. If median, $+\sigma$ and $-\sigma$ predictions of the attenuation relationship by Abrahamson and Silva (1997) are used, then no significant changes in the ratios are observed for $T_{str} = 0.5$ s. As the figure implies, uncertainty in attenuation relationships is not a significant issue as long as it consistently affects both S_A and PGA.


Figure 6.6-4. Variation of the results considering the error function of attenuation relationship

• Figure 6.6-5 shows the variation in the CSR_{SSEI}/CSR_{FF} due to the variations in soil stiffness. (V_s = 80 m/s and V_s = 150 m/s). This figure implies that although the stiffness of soils increases twice, the ratio of structural-induced cyclic stress ratio to free field stress ratio increases slightly.



Figure 6.6-5. Sensitivity of the results with respect to soil stiffness

6.7 CONCLUDING REMARKS

Although a satisfactory fit was achieved among the CSR estimations by numerical seismic response analysis results and the proposed simplified procedure, validation

of the proposed procedure further with available laboratory shaking table and centrifuge tests and well-documented field case histories is preferred. For this purpose, centrifuge and shaking table test results of soil and structure models were studied. As presented in this chapter, the proposed simplified procedure successfully captures almost all of the behavioral trends and most of the amplitudes.

CHAPTER 7

SUMMARY and CONCLUSION

7.1 SUMMARY

The purpose of this study is defined as the development of a simplified procedure for the assessment of seismic liquefaction triggering potential of foundation soils. Within this scope, three dimensional, numerical, soil-structure interaction analyses were performed to simulate both static and seismic stress state and performance. Parallel to these studies, i) static effects of the buildings in terms of base shear stresses ii) static shear stress ratios, α , iii) effective stress effects beneath the foundations are intended to be resolved.

Current practice in the calculation of cyclic stress ratios for liquefaction triggering potential is largely dominated by Seed and Idriss (1971)'s simplified procedure. Unfortunately, the simplified procedure is originally proposed for free field level site conditions, where vertical and horizontal directions are the major and minor principal stress directions, and seismically-induced shear stresses oscillates along the horizontal plane. However, these assumptions are not satisfied for soils beneath structure foundations due to i) presence of foundation loads complicating the static stress state, ii) kinematic and inertial interaction of the superstructure with the

foundation soils and seismic excitation. Addressing the effects of the different static stress state in liquefaction initiation response, series of corrections, formerly known as K_{α} and K_{σ} , were proposed later to the original procedure. In the literature, there exist contradicting arguments regarding if and how the presence of an overlying structure and foundation element affects liquefaction triggering potential and how these corrections should be applied. Thus, within the confines of this thesis, it is also intended to resolve this controversial, yet important issue.

Since the very early days of geotechnical earthquake engineering profession, the seismic response of soil and structure interacting sites are acknowledged to be different than that of free field soils sites. However, due to complexities in the treatment of these differences, foundation soils are usually treated as if they were free field level soil sites with a major assumption that this treatment is conservatively biased (Watanabe, 1966, Ishihara et al., 1980 and Rollins, 1987). Although, its critical importance has been recognized for years, there are very limited number of studies tackling the effects of SSEI from both structural and geotechnical points of views. Veletsos and Meek (1974) studied if soil structure interaction is critical, and Rollins and Seed (1990) investigated how this interaction may affect the liquefaction potential of foundation soils. Veletsos and Meek (1974) introduced a relative stiffness term, σ , which represents the ratio of structure-to-soil stiffnesses. It is concluded that when σ values vary in the range of 3 to 20, soilstructure interaction becomes more critical. Later, Rollins and Seed (1990) proposed a simple and easy to estimate parameter, S_A/PGA, of which produces a critical liquefaction response if it is larger than 2.40.

For the purpose of assessing the effects of the presence of a structure on liquefaction triggering potential of foundation soils, three-dimensional, finite difference-based total stress analyses were performed for generic soil, structure and earthquake combinations. 2, 3, 4 and 5 storey typical residential structures with first mode periods varying in the range of 0.2 to 0.50 seconds were modeled on a mat foundation. Foundation soil profiles were selected as composed of cohesionless soils

with shear wave velocities varying in the range of 100 - 300 km/s. Four different types of earthquake excitations, namely 1999 Kocaeli, 1989 Loma Prieta, 1995 Kobe and 1979 Imperial Valley earthquakes, were used to assess the seismic interaction of the soil and structure system.

In addition to dynamic analyses, three dimensional, finite difference based static and dynamic soil-structure interaction analyses were also performed. Based on these static analyses results, corrections for overburden and static shear stresses were applied leading to K_{σ} and K_{α} corrected structural – induced cyclic stress ratio, CSR_{SSEI} values. These CSR_{SSEI} values showed a significant variability beneath the foundation. Representative (average) and a maximum CSR_{SSEI} values were calculated at various locations.

A simplified procedure was proposed which would produce unbiased estimates of these representative and maximum CSR_{SSEI} values eliminating the need to perform 3-D dynamic response assessment of soil and structure systems for conventional projects. Consistent with the available literature, the descriptive (input) parameters were selected as σ , S_A/PGA, aspect ratio. The model coefficients were estimated through maximum likelihood methodology which is used to produce an unbiased match with the predictions of 3-D analyses and proposed simplified procedure.

Although a satisfactory fit was achieved among the CSR estimations by numerical seismic response analysis results and the proposed simplified procedure, validation of the proposed simplified procedure further with available laboratory shaking table and centrifuge tests and well-documented field case histories was preferred. For this purpose, centrifuge and shaking table test results of soil and structure models were studied. Fifteen centrifuge test results were concluded to be suitable for validation purposes. As presented in detail in Chapter 6, the proposed simplified procedure successfully captures almost all of the behavioral trends and most of the amplitudes.

7.2 CONCLUSIONS

Based on the results of both 3-D soil structure and earthquake interaction models and forward interpretation of the proposed simplified procedure to assess SSEIinduced liquefaction triggering for foundation soils, following conclusions were drawn:

Contrary to general conclusions of Rollins and Seed (1990), and consistent with the observations of Finn and Yodengrakumar (1987), Liu and Dobry (1997) and Mylonakis and Gazetas, (2000), soil-structure interaction does not always beneficially affect the liquefaction triggering potential of foundation soils. In other words, use of Seed and Idriss (1971) simplified procedure, which was originally developed for the liquefaction assessment of free field soil sites, for foundation soils under the influence of an overlying structural system, even with proper K_{α} and K_{σ} corrections does not always produce conservative estimates of the liquefaction triggering response. More specifically,

Valid for structures with mat foundations, under static conditions, shear stresses due to the presence of the structure scatter to a wider area than vertical stresses. Hence, α field dissipates much faster than the vertical foundation stresses, $\Delta\sigma$.

High static shear stresses, contrary to relatively lower vertical effective stresses were observed at the edges of the structures, leading to a high α value. Thus, "loose" soil zones extending B/6 distance from the edges are concluded to be more vulnerable to liquefaction triggering than the foundation soils in the vicinity of the symmetry line.

For perfectly rigid structures founded on potentially liquefiable soils, compared to free field soil sites, presence of the structure reduces seismic demand expressed in terms of CSR. Thus, use of simplified procedure produces conservative conclusions.

For all other cases, the interactions among soil, structure and earthquake, mostly defined by i) σ , ii) S_A/PGA, iii) aspect ratio, h/B, along with iv) static stress field and v) state parameters, determine if the presence of the structure positively or

negatively affects the liquefaction triggering response. For foundation soils of structures with long periods, such as high rise buildings, bridge piers, dams etc., presence of an overlying structure may reduce liquefaction triggering potential with the exception of deep soil sites (resonance effects).

The proposed simplified procedure effectively estimates the seismic demand, CSR beneath foundations considering the essentials of the SSEI problem. Easy to estimate parameters such as shear wave velocity, thickness of the soil profile, period of the structure, peak ground acceleration of the earthquake excitation, spectral acceleration corresponding to the first modal period of the structure was selected to model this interaction. The proposed procedure conveniently corrects for K_{σ} and K_{α} effects.

 CSR_{SSEI} rapidly approaches to $CSR_{free field}$ values with depth or outside the influence zone of the structure. Beyond a depth of 1B, or 1B distance away from the edges of the structure, the effects of the overlying structure on CSR is significantly reduced, and almost disappears beyond 2 B.

7.3 RECOMMENDATIONS FOR FUTURE RESEARCH

These studies have identified various important aspects of seismic soil liquefaction potential of foundation soils within a probabilistic framework, which warrant additional research. These include:

 Spatial variability in structural, soil and excitation characteristics. More cases can be added to refine the studies. Structures with different periods resting on different soil profiles (either homogenous or layered) can be added as new cases. Different earthquake excitations can be used to shake the combined structure – soil system.

- 2. The foundation system can be enhanced. Instead of mat foundations, piled foundations or single footings can be used. Effect of foundation systems can be added to see how it changes the liquefaction potential.
- Liquefaction potential of foundation soils on cohesive soils can be investigated instead of cohesionless soils although the liquefaction potential of cohesive soils is still a controversial issue.
- 4. The exact locations of maximum CSR value can be tried to be determined. It is known from this study that it appears near the edges of the structures at the surface. However, with depth the location of this maximum CSR value changes (either conically or randomly).
- 5. Finally, having solved the "triggering" part of the problem, efforts can be focused on developing probabilistically-based methodologies beginning from the estimation of seismic soil liquefaction-induced ground deformations and continue with other steps of liquefaction engineering.

REFERENCES

- Abrahamson, N.A. and Silva, W.J., (1997), "Empirical response spectral attenuation relations for shallow crustal earthquakes.", Seism. Research Letters, 68(1), 94-127.
- Allen R., Oppenheim I., Parker A., Bielak J. (1986), "On the dynamic response of rigid body assemblies", Earthquake Engineering and Structural Dynamics, 14(6), 861 876.
- Allen R., Duan X. (1995), "Effects of linearizing on rocking block", Journal of Structural Engineering, 12(7), 1146 1149.
- Andrews D.C., Martin G.R. (2000), "Criteria for liquefaction of silty sands", Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand
- Aslam M., Godden W., Scalise D. (1980), "Earthquake rocking response on rigid bodies", Journal of the Structural Division of the ASCE, 106(ST2), 377 – 392.
- Bardet, J.P., Mace, N., Tobita, T., and Hu, J. (1999). "Large-scale modeling of liquefactioninduced ground deformation part II: MLR model applications and probabilistic model.", Proceedings of the Seventh U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, Technical Report No. MCEER-99-0019, Mid-America Center for Earthquake Engineering Research, Buffalo, N.Y.
- Bartlett, S. F. and Youd, T. L. (1992), "Empirical analysis of horizontal ground displacement generated by liquefaction-induced lateral spreads", Technical Report No. NCEER-92-0021, National Center for Earthquake Engineering Research, State University of New York, Buffalo, pp. 5-14-15.
- Bartlett, S.F. and Youd, T.L. (1995), "Empirical prediction of liquefaction-induced lateral spread." Journal of Geotechnical Engineering, Vol. 121, No. 4, pp. 316-329.
- Berrill, J.B., Davis, R.O. (1985), "Energy Dissipation and Seismic Liquefaction of Sands: Revised Model." JSSMFE Soils and Foundations 25(2): 106-118.

- Bielak, J. (1975), "Dynamic behavior of structures with embedded foundations.", J. *Earthquake Engrg. Struct. Dyn.*, 3(3), 259–274.
- Bolton, M.D. (1986), "The strength and dilatancy of sands.", Geotechnique 36, No. 1,65-78.
- Booker, J.R., Rahman, M.S., Seed, H.B. (1976), "GADFLEA A Computer Program for the Analysis of Pore Pressure Generation and Dissipation During Cyclic or Earthquake Loading", EERC 76-24. University of California, Berkeley.
- Boore D.M., Joyner W.B. and Fumal T.E. (1997), "Equations for estimating horizontal response spectra and peak acceleration from western north American earthquakes: a summary of recent work.", Seismological Research Letters, Vol. 68, No. 1, 128-153.
- Boulanger, R.W., Seed, R.B. (1995), "Liquefaction of sand under bi-directional monotonic and cyclic loading", Journal of Geotechnical Engineering, ASCE, 121(12), 870-878.
- Boulanger R.W. (2003a), "Relating K_{α} to Relative State Parameter Index", Technical note, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 129(8), 770-773.
- Boulanger R.W. (2003b), "High Overburden Stress Effects in Liquefaction Analyses", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 129(12), 1071-1082.
- Boulanger, R.W., Idriss, I.M. (2004), "State normalization of penetration resistances and the effect of overburden stress on liquefaction resistance", Proc., 11th International Conference on Soil Dynamics and Earthquake Engineering, and 3rd International Conference on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Vol. 2, 484-491.
- Boulanger, R.W., Idriss, I.M. (2006), "Liquefaction susceptibility criteria for silts and clays", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 132(11), 1413-1426.
- Boussinesq, J. (1885), "Applications Des Potentiels a l'etude de l'equilibre et du Mouvement Des Solids Elastiques.", Gather-Villars.

- Bray, J. D., Sancio, R. B., Durgunoglu, H. T., Onalp, A., Seed, R. B., Stewart, J. P., Youd, T. L.,
 Baturay, M. L., Cetin, K. O., Christensen, C., Karadayilar, T., and Emrem, C., (2001),
 "Ground failure in Adapazari, Turkey.", Proceedings of Earthquake Geotechnical
 Engineering Satellite Conference of the XVth International Conference on Soil Mechanics &
 Geotechnical Engineering, Istanbul, Turkey, August 24-25.
- Bryne P.M. (1991), "A cyclic shear-volume coupling and pore pressure model for sand", Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri: 47-55.
- Castro G. (1975), "Liquefaction and cyclic mobility of saturated sands", ASCE, Geotechnical Engineering Division, 101(6), 551–569.
- Castro G., Poulos S.J. (1977), "Factors affecting liquefaction and cyclic mobility", ASCE, Geotechnical Engineering Division, 103(6), 501–516.
- Cetin K.O., Seed R.B., Der Kiureghian A., Tokimatsu K., Harder L.F. Jr, Kayen R.E., Moss R.E.S. (2004), "SPT-based probabilistic and deterministic assessment of seismic soil liquefaction potential", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 130(12), 1314-1340.
- Cetin K.O., Bilge H.T., Wu J., Kammerer A. and Seed R.B., (2009). Probabilistic models for cyclic straining of saturated clean sands. J. Geotech. and Geoenvironmental Engrg, ASCE, in press.
- Chen S.S., Shi J.Y. (2006), "Simplified model for vertical vibrations of surface foundations", Journal of Geotechnical and Geoenvironmental Engineering, 132(5), 651-655.
- Davis, A.P., Poulos, S.J. and Castro, G. (1988), "Strenghts backfigured from liquefaction case histories.", Second International Conference on Case Histories in Geotechnical Engineering, St. Louis. 3: 1693-1701

- Ehlers G. (1942), "The effect of soil flexibility on vibration systems", Beton und Eisen, 41(21/22), 197–203.
- Eurocode 8 (2003), "Design of structures for earthquake resistance International Building Code".
- Fadum, R.E. (1948), "Influence values for estimating stresses in elastic foundations", Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Volume 3, pp. 77-84.
- FLAC3D (2005), "User's Manual-Fast Lagrangian Analysis of Continua in 3 Dimensions", Itasca Consulting Group, Minnesota.
- Finn, W. D. L., Ledbetter, R. H., and Beratan, L. L. (1986), "Seismic Soil-Structure Interaction: Analysis and Centrifuge Model Studies.", Nuclear Engineering and Design, Vol. 94, No. 1, pp. 53-66.
- Finn W.D.L., Yodengrakumar M. (1987), "Analysis of pore water pressure in seismic centrifuge tests"., Soil Dynamics and Liquefaction, Elsevier, Amsterdam.
- France, J.W., Adams, T., Wilson, J., and Gillette, D., (2000), "Soil dynamics and liquefaction", Geotechnical SpecialPublication No. 107, ASCE.
- Ghosh B., Madabhushi S.P.G. (2003), "Behaviour of rigid foundation on homogeneous loose soil.", Data report on tests BG-01 to BG-03, Engineering Department, University of Cambridge, CUED/D-SOIL/TR. 330.
- Golesorkhi, R. (1989), "Factors influencing the computational determination of earthquakeinduced shear stresses in sandy soils.", PhD dissertation, University of California at Berkeley.
- Green R. A., Mitchell, J.K., Polito C.P., (2000), "An energy-based excess pore pressure generation model for cohesionless soils", Proceedings of the John Booker Memorial

Symposium, Sydney, New South Wales, Australia, November 16-17, 2000, A.A. Balkema Publishers, Rotterdam. Netherlands.

- Hamada, M., Yasuda, S., Isoyama, R., and Emoto, K. (1986)."Study On Liquefaction Induced Permanent Ground Displacements", Report of the Association for the Development of Earthquake Prediction in Japan, Tokyo Japan
- Hogan S. (1990), "The many steady state responses of a rigid block under harmonic forcing", Earthquake Engineering and Structural Dynamics, 19(7), 1057 1071.
- Housner GW. (1963), "The behaviour of inverted pendulum structures during earthquakes", Bulletin of the Seismological Society of America, Vol. 53, 403 417.
- Kandilli Observatory and Earthquake Research Center, Boğaziçi University, http://www.koeri.boun.edu.tr/earthqk/earthqk.html, last visited on September 2008.
- Hu L., Pu J. (2004), "Testing and modeling of soil-structure interface." Journal of Geotechnical and Geoenvironmental Engineering, 130(8), 851 860.
- Hwang J. et al. (1994), "Seismic analysis of liquefaction considering soil-structure interaction". 10th ECCE, Vol. I, Vienna, 1994.
- IBC (2000), International Building Code, International Code Council, USA.
- Idriss, I. M.; Sun, J. I., (1993), "User's manual for SHAKE91: a computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits", Center for Geotechnical Modeling, Dept. of Civil and Environmental Engineering, University of California, Davis Davis, Calif.
- Idriss, I.M., (1999), "An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential", Proc., TRB Workshop on New Approaches to Liquefaction, January, Publication No.FHWA-RD-99-165, Federal Highway Administration, 1999.

- Idriss, I.M., Boulanger, R.W. (2007), "SPT- and CPT-based relationships for the residual shear strength of liquefied soils", Earthquake Geotechnical Engineering, 4th International Conference on Earthquake Geotechnical Engineering – Invited Lectures, K. D. Pitilakis, ed., Springer, The Netherlands, 1 – 22.
- Ishihara, K., Yoshimine, M. (1992), "Evaluation of settlements in sand deposits following liquefaction during earthquakes", Soils and Foundations, 32(1), 173 188.
- Ishihara, K. (1993), "Liquefaction and flow failure during earthquakes", Geotechnique, 43(3), 351 415.
- Ishihara, K., (1996), "Soil Behaviour in Earthquake Geotechnics", Oxford Engineering Science Series, Clarendon Press, Oxford.
- Ishihara, K., Kawase, Y., and Nakajima, M. (1980), "Liquefaction characteristics of sand deposits at an oil tank site during the 1978 Miyagiken-Oki earthquake.", Soils and Found., Tokyo, Japan, 20(2), 97-111.
- Ishiyama Y. (1982), "Motions of rigid bodies and criteria for overturning by earthquake excitations", Earthquake Engineering and Structural Dynamics, 10(5), 635 650.
- Jong H.L., Seed R.B. (1988), "A Critical Investigation of Factors Affecting Seismic Pore Presure Generation and Post-Liquefaction Flow Behavior of Saturated Soils" Geotechnical Research Report No. SU/GT/88-01, Stanford University, April 1988, 407 pp.
- Kikuchi, M. (1995), "Teleseismic analysis of the Southern Hyogo (Kobe), Japan, earthquake of January 17, 1995", Yokohama City University Seismological Note #38.
- Kim Y-S., Miura K., Miura S., Nishimura M. (2001), "Vibration characteristics of rigid body placed on sand ground", Soil Dynamics and Earthquake Engineering, 21(1), 19 37.
- Koh A-S., Spanos P.D., Roesset J. (1986), "Harmonic rocking of rigid body block on flexible foundation", Journal of Engineering Mechanics, ASCE, 112(11), 1165 1180.

- Koh A-S., Spanos P.D. (1986), "Analysis of block random rocking", Soil Dynamics and Earthquake Engineering, 5(3), 178 183.
- Kuhlemeyer R.L. and Lysmer J., (1973), "Finite element method accuracy for wave propagation problems.", Journal of Soil Mechanics and Foundations, Div., ASCE Vol. 99, pp. 421–427 (SM5).
- Lamb, H., (1904), "On the propagation of tremors over the surface of an elastic solid.", Phil. Trans. Roy. Soc. (London) A, 203, 1-42.
- Lee K.L., Albaisa A. (1974), "Earthquake-induced settlement in saturated sands", Journal of Soil Mechanics and Foundations Division, ASCE, 100, 387–406.
- Liao S.S.C., Whitmann R.V. (1986)' "Overburden correction factors for SPT in sand", Journal of Geotechnical Engineering, 112(3), 373 – 377.
- Liu A.H., Stewart J.P., Abrahamson N.A and Moriwaki Y. (2001), "Equivalent number of uniform stress cycles for soil liquefaction analysis", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(12), 1017–1026.
- Liu L., Dobry R. (1997), "Seismic response of shallow foundation on liqueifable sand" Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 123(6), 557-567.
- Love, A.E.H. (1929), "The stress produced in a semi infinite solid by pressure on part of the boundary", Philosophical Transactions of the Royal Society of London, Series A, Volume 228, pp. 377-420. (Q1.L84).
- Lysmer, J., Kuhlemeyer, R.L., (1969). "A finite dynamic model for infinite media", Journal of Engineering Mechanics Division, 95, 859 877.
- Makris N., Zhang J. (2001), "Rocking response of anchored blocks under pulsetype motions", Journal of Engineering Mechanics, ASCE, 127(5), 484 – 493.

- Martin, G.R., Finn, W.D.L., Seed, H.B. (1975), "Fundamentals of liquefaction under cyclic loading", Journal of the Geotechnical Engineering Division, ASCE, 101(GT5), 423 438.
- Meek J.W., Wolf J.P. (1992a), "Cone models for homogeneous soil I", Journal of the Geotechnical Engineering Division, ASCE, 118(5), 667 685.
- Meek J.W., Wolf J.P. (1992b), "Cone models for soil layer on rigid rock II.", Journal of the Geotechnical Engineering Division, ASCE, 118(5), 686 703.
- Mitrani H., Madabhushi S.P.G. (2006), "Frame structures founded on liquefiable soil Data report on centrifuge tests BM1 BM2 and BM3.", Engineering Department, University of Cambridge, CUED/D-SOIL/TR. 342.
- Mogami, H., Kubo, T. (1953), "The behavior of soil during vibration", Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 152-153.
- Morris D.V. (1981), "Dynamic soil-structure interaction modelled experimentally on a geotechnical centrifuge", Canadian Geotechnical Journal, 18, 40-51.
- Mostaghel N., Hejazi M., Tanbakuchi J. (1983), "Response of sliding structures to harmonic support motion", Earthquake Engineering and Structural Dynamics, 11(3), 355 366.
- Mylonakis G., Gazetas, G. (2000), "Seismic soil-structure interaction: beneficial or detrimental?", Journal of Earthquake Engineering, Vol. 4, No. 3, 277 301.
- NEHRP (2003), "The 2003 NEHRP Recommended Provisions For New Buildings And Other Structures Part 2: Commentary (FEMA 450).
- Newmark, N.M. (1942), "Influence charts for computation of stresses in elastic foundations", University of Illinois, Engineering Experimental Station, Bulletin Series no. 338, p. 28.
- Newmark, N. M. (1965), "Effects of earthquakes on dams and embankments." *Geotechnique*, 15(2), 139–160.

- NCEER (1997), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", Edited by Youd, T. L., Idriss, I. M., Technical Report No. NCEER-97-0022, December 31, 1997.
- Olson, S.M., Stark, T.D. (2002), "Liquefied strength ratio from liquefaction case histories" Canadian Geotechnical Journal, 39, 629 – 647.
- Olson, S.M., Stark, T.D. (2003), "Yield strength ratio and liquefaction analysis of slopes and embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 129(8), 727 737.
- Pillai, V.S., Muhunthan, B. (2001). "A review of the influence of initial static shear (K_{α}) and confining stress (K_{σ}) on failure mechanisms and earthquake liquefaction of soils", Paper No. 1.51, Proc. 4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA.
- Popescu R., Prevost J.H. (1993), "Centrifuge validation of numerical model for dynamic soil liquefaction", Soil Dynamics and Earthquake Engineering, 12(2), 73-90.
- Poulos S.J., Castro G., France J.W. (1985), "Liquefaction evaluation procedure", Journal of Geotechnical Engineering Division, ASCE, 111(6), 772–792.
- Psycharis I., Jennings P. (1983), "Rocking of slender rigid bodies allowed to uplift", Earthquake Engineering and Structural Dynamics, 11, 57 76.
- Psycharis I. (1990), "Dynamic behaviour of rocking two-block assemblies", Earthquake Engineering and Structural Dynamics, 19, 555 575.
- Rainer J.H. (1975), "Simplified analysis of dynamic structure-ground interaction", Canadian Journal of Civil Engineering, 2, 345 – 356.
- Robertson, P.K. et al., (2000), "The CANLEX Project: summary and conclusions", Canadian Geotechnical Journal, 37, 563–591.

- Robertson, P.K., Woeller, D.J., Finn, W.D.L. (1992), "Seismic cone penetration test for evaluating liquefaction potential under cyclic loading", Canadian Geotechnical Journal, 29, 686-695.
- Robertson, P.K., and Wride, C.E. (1997). "Cyclic Liquefaction Potential and Its Evaluation Based on the SPT and CPT," Proceedings, Workshop on Evaluation of Liquefaction Resistance, NCEER-97-0022, Buffalo, NY, 41-87.
- Rollins, K. M. (1987). "The influence of buildings on potential liquefaction damage.", Dissertation presented to the University of California, at Berkeley, Calif., in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
- Rollins K.M., Seed H.B. (1990), "Influence of buildings on potential liquefaction damage." Journal of Geotechnical Engineering, ASCE, 116(2), 165 – 185.
- Schnabel, P. B. (1973), "Effects of Local Geology and Distance from Source on Earthquake Ground Motion", Ph.D. Thesis, University of California, Berkeley, California.
- Schnabel P.B., Lysmer J., Seed H.B. (1973), "SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites", Report No. EERC/72-12, Earthquake Engineering Research Center, University of California at Berkeley, 88p.
- Seed, H.B., Idriss, I.M. (1970), "Soil moduli and damping factors for dynamic response analysis", Report No. UCB/EERC-70/10, Earthquake Engineering Research Center, University of California, Berkeley, December, 48p.
- Seed, H.B., Idriss, I.M. (1971), "Simplified procedure for evaluating soil liquefaction potential" Journal of the Soil Mechanics and Foundation Division, ASCE, 97(SM9), Proc. Paper 8371, September 1971, 1249 – 1273.
- Seed, H.B., Idriss, I.M., Makdisi, F., and Banerjee, N. (1975). "Representation of Irregular Stress Time Histories by Uniform Equivalent Series in Liquefaction Analysis," Report No. EERC-75-29, University of California, Berkeley, 40 pp.

- Seed, H.B., Idriss, I.M. (1982), "Ground motions and soil liquefaction during earthquakes", Earthquake Engineering Research Institute Monograph Series.
- Seed H.B. (1983), "Earthquake resistant design of earth dams" Proceedings of Seismic Design of Embankments and Caverns, Philadelphia, ASCE, New York, 46–64.
- Seed, H.B., Idriss, I.M., Arango, I. (1983), "Evaluation of liquefaction potential using field performance data", Journal of Geotechnical Engineering, ASCE, 109(3), 458 – 482.
- Seed, H.B., Tokimatsu, K., Harder, L. F., Chung, R. M. (1984a), "The influence of SPT procedures in soil liquefaction resistance evaluations", Earthquake Engineering Research Center Report No. UCB/EERC-84/15, University of California at Berkeley, October, 1984.
- Seed, H.B., Wong, R.T., Idriss, I.M., Tokimatsu, K. (1984b), "Moduli and damping factor for dynamic analyses of cohesionless soils", Geotechnical Engineering Report No. UCB/EERC-84/14, University of California at Berkeley, September 1984.
- Seed, H.B. (1987), "Design problems in soil liquefaction", Journal of Geotechnical Engineering, ASCE, 113(8), 827 – 845.
- Seed, R.B., Harder, L.F. Jr., (1990), "SPT-based analysis of cyclic pore pressure generation and undrained residual strength", Proceedings of the H. B. Seed Memorial Symposium, May 1990.
- Seed, R. B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Wu, J., Pestana, J. M., and Riemer, M. F. (2001), "Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation", International Conference and Symposium on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA.
- Seed, R.B. et al. (2003), "Recent advances in soil liquefaction engineering: a unified and consistent framework", 26th Annual ASCE Los Angeles Geotechnical Spring Seminar.
- Shenton III H.W. (1996), "Criteria of initiation of slide, rock, and slide-rock rigid-body modes", Journal of Engineering Mechanics, ASCE, 122(7), 690 – 693.

- Shamoto, Y., Zhang, J., Tokimatsu, K. (1998). "New charts for predicting large residual postliquefaction ground deformations." Soil Dynamics and Earthquake Engineering, 17(7-8), Elsevier, New York, 427–438.
- Shibata, T. and Teparaksa, W. (1988), "Evaluation of liquefaction potential of soils using cone penetration tests", Soils and Foundations, Vol.28, No.2,pp.49-60.
- Sinopoli A. (1987), "Dynamics and impact in a system with unilateral constraints. The relevance of dry friction", Journal of the Italian Association of Theoretical and Applied Mechanics, 22(4), 210 – 215.
- Smith I.. (1969), A finite element approach to elastic soil-structure interaction, Canadian Geotechnical Journal, 7(2) MAY 1970 NUMBER 2, pp 95-105.
- Smith-Bardo J.P., Bobet A. (2007), "Behavior of rigid footings on gravel under axial load and moment", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 133(10), 1203-1215.
- Spanos P.D., Koh A.S. (1984), "Rocking of rigid blocks due to harmonic shaking", Journal of Engineering Mechanics, 110(11), 1627 – 1642.
- Spanos P.D., Roussis P.C., Politis N.P.A. (2001), "Dynamic analysis of stacked rigid blocks", Soil Dynamics and Earthquake Engineering, 21(7), 559 – 578.
- Stark, T.D., Mesri G. (1992), "Undrained Shear Strength of Liquefied Sands for Stability Analysis" Journal of Geotechnical Engineering Division, ASCE, 118(11), 1727-1747.
- Stewart J.P., Fenves G.L., Seed R.B. (1999), "Seismic soil-structure interaction in buildings. I: Analytical methods", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 125(1), 26-37.
- Stewart J.P. (2000), "Variations between foundation-level and free-field earthquake ground motions", Earthquake Spectra, 16 (2), 511-532.

- Szerdy,F. (1985), "Flow slide failure associated with low level vibrations.", Dissertation Presented to the University of California at Berkeley, Calif., In partial fulfillment of the requirements for the degree of Doctor of Philosophy.
- Tatsuoka, F., Muramatsu, M., Sasaki, T. (1982). "Cyclic undrained stress-strain behavior of dense sands by torsional simple shear test", Soils and Foundations, 22(2), 55-70.
- Terzaghi, K., Peck, R.B. (1948), "Soil Mechanics in Engineering Practice", John Wiley and Sons, Inc., 2nd Edition.
- Tileylioglu, S., Nigbor, R.L. and Stewart J.P. (2008), "Determination of soil-structure interaction effects for a model test structure using parametric system identification procedures", Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics IV GSP 181, Sacramento, CA.
- Tokimatsu, K., Yoshimi, H. (1981), "Field correlation of soil liquefaction with SPT and grain size", Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics.
- Tokimatsu, K., Seed, H.B. (1984), "Simplified procedures for the evaluation of settlements in sands due to earthquake shaking", Report No. UCB/EERC-84/16, Earthquake Engineering Research Center, University of California, Berkeley, CA.
- Towhata, I., Sasaki, Y., Tokida, K.I., Matsumoto, H., Tamari, Y., Yamada, K. (1992), "Mechanism of permanent displacement of ground caused by seismic liquefaction", Soils and Foundations, 32(3), 79–96.
- Travasarou T., Bray J.D., Sancio R.B. (2006), "Soil-structure interaction analyses of building responses during the 1999 Kocaeli earthquake", Proc. 8th U.S. National Conference on Earthquake Eng., Paper No.1877.
- Tso W., Wong C. (1989), "Steady state rocking response of rigid blocks. Part 1: analysis", Earthquake Engineering and Structural Dynamics, 18(1). 89 106.

- Turkish Earthquake Code (2007), Bayındırlık ve İskân Bakanlığı, Afet Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik, Ankara (in Turkish)..
- Vaid, Y.P., Thomas, J. (1995), "Liquefaction and postliquefaction behavior of sand", Journal of Geotechnical Engineering, ASCE, 121(2), 163–173.
- Vaid, Y.P., Chern, J.C. (1983), "Effect of static shear on resistance to liquefaction", Soils and Foundations, 23(1): 47–60.
- Vaid, Y.P., Finn, W.D.L. (1979), "Static shear and liquefaction potential", Journal of the Geotechnical Engineering Division, ASCE, 105(GT10), 1233–1246.
- Vaid, Y.P., Chern, J.C. (1985), "Cyclic and monotonic undrained response of sands", Proceedings of the ASCE Convention: Advances in the Art of Testing Soils Under Cyclic Loading Conditions, Detroit, Mich. Edited by V. Khosla, 171–176.
- Vaid, Y.P., Chern, J.C., and Tumi, H. (1985), "Confining pressure, grain angularity and liquefaction.", Journal of Geotechnical Engineering, ASCE, 111(10), 1229–1235.
- Vaid Y.P., Thomas J. (1994), "Post-liquefaction behaviour of sand", Proc. 13th International Conference on Soil Mechanics and Foundation Engineering, New Delhi, Vol. 3, 1305–1310.
- Vaid, Y.P., Sivathayalan, S. (1996), "Static and cyclic liquefaction potential of Fraser Delta sand in simple shear and triaxial tests", Canadian Geotechnical Journal, 33(2), 281–289.
- Veletsos, A. S., Nair V. V. (1975), "Seismic interaction of structures on hysteretic foundations" Journal of the ASCE Structural Division, 101(ST1), 109-129.
- Veletsos A.S., Meek J.W. (1974), "Dynamic behavior of building-foundation systems", Earthquake Eng. and Structural Dynamics, 3, 121-138.
- Watanabe, T. (1966), "Damage to oil refinery plants and a building on compacted ground by the Niigata earthquake and their restoration.", *Soils and Found.*, Tokyo, Japan, 6(2), 86-99.

- Wride, C.E., Mcroberts, E.C., and Robertson, P.K. (1999) "Reconsideration of case histories for estimating undrained shear strength in sandy soils," Can. Geotech. Jr, Vol. 36(5), 907-933.
- Wu, J., Seed, R.B., Pestana, J.M. (2003), "Liquefaction Triggering and Post Liquefaction Deformations of Monterey 0/30 Sand under Uni-Directional Cyclic Simple Shear Loading", Geotechnical Engineering Research Report No. UCB/GE-2003/01, UC Berkeley.
- Wu, J., Seed R. B. (2004), "Estimation of liquefaction-induced ground settlement (case studies)", Proceedings, 5th International Conf. on Case Histories in Geotech. Eng., NY.
- Yang Y.B., Lee T.Y., Tsai I.C. (1990), "Response of multi-degree-of-freedom structures with sliding supports", Earthquake Engineering and Structural Dynamic. Vol. 19(5),739–752.
- Yegian M.K., Ghahraman V.G. and Harutinunyan R.N. (1994), "Liquefaction and embankment failure case histories, 1988 Armenia earthquake.", J. Geotech. Eng. 120, pp. 581–596.
- Yim C.S., Lin H. (1991), "Nonlinear impact and chaotic response of slender rocking objects", Journal of Engineering Mechanics, ASCE, 117(9), 2079 – 2100.
- Yoshimi Y., H. Oh-Oka H. (1975), "Influence of degree of shear stress reversal on the liquefaction potential of saturated sands", Soils and Foundations, 26(1), 25–35.
- Yoshimi Y.K., Tokimatsu K. (1977), "Settlement of buildings on saturated sand during earthquakes", Soils and Foundations.
- Yoshiaki Y. et al., (1997), "Settlement of building on saturated sand during earthquake" Soils and Foundations, 37(1).
- Youd, T. L., Hansen, C. M., and Bartlett, S. F. (2002), "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, pp. 1007-1017.
- Zhang J., Makris N. (2001), "Rocking response of free-standing blocks under cycloidal pulses", Journal of Engineering Mechanics, ASCE, 127(5), 473 – 483.

APPENDIX A

CENTRIFUGE AND SHAKING TABLE TEST RESULTS



Figure A. 1. Comparison of centrifuge test results with the proposed formulations, BG01-EQ-1



Figure A. 2. Comparison of centrifuge test results with the proposed formulations, BG02-EQ-2



Figure A. 3. Comparison of centrifuge test results with the proposed formulations, BG02-EQ-3



Figure A. 4. Comparison of centrifuge test results with the proposed formulations, BG02-EQ-4



Figure A. 5. Comparison of centrifuge test results with the proposed formulations, BG03-EQ-3



Figure A. 6. Comparison of centrifuge test results with the proposed formulations, BG03-EQ-4



Figure A. 7. Comparison of centrifuge test results with the proposed formulations, BG03-EQ-5



Figure A. 8. Comparison of centrifuge test results with the proposed formulations, BM01-SDOF-EQ-1



Figure A. 9. Comparison of centrifuge test results with the proposed formulations, BM01-SDOF-EQ-2



Figure A. 10. Comparison of centrifuge test results with the proposed formulations, BM01-SDOF-EQ-3



Figure A. 11. Comparison of centrifuge test results with the proposed formulations, BM01-SDOF-EQ-4



Figure A. 12. Comparison of centrifuge test results with the proposed formulations, BM02-2DOF-EQ-1



Figure A. 13. Comparison of centrifuge test results with the proposed formulations, BM02-2DOF-EQ-2


Figure A. 14. Comparison of centrifuge test results with the proposed formulations, BM02-2DOF-EQ-3



Figure A. 15. Comparison of centrifuge test results with the proposed formulations, BM02-2DOF-EQ-4



Figure A. 16. Comparison of shaking table test results with the proposed formulation

APPENDIX B

COMPARISON OF 1-D AND 3-D ANALYSES RESULTS



Figure B. 1. Comparison of Response Spectra for $V_s = 100$ m/s and 1999 Kocaeli Earthquake



Figure B. 2. Comparison of Response Spectra for $V_s = 150$ m/s and 1999 Kocaeli Earthquake



Figure B. 3. Comparison of Response Spectra for $V_s = 200$ m/s and 1999 Kocaeli Earthquake



Figure B. 4. Comparison of Response Spectra for $V_s = 100$ m/s and 1979 Imperial Valley Earthquake



Figure B. 5. Comparison of Response Spectra for $V_s = 150$ m/s and 1979 Imperial Valley Earthquake



Figure B. 6. Comparison of Response Spectra for $V_s = 200$ m/s and 1979 Imperial Valley Earthquake

APPENDIX C

SHAKE AND FLAC INPUT FILES

SHAKE91 INPUT FILE

Option 1 - Dynamic Soil Properties Set No. 1 1 10 20 Soil PI=0 G/Gmax - Soil with PI=0, OCR=1-15 (Vucetic & Dobry, JG 0.0003 0.0005 0.0007 0.001 0.002 0.003 0.004 0.007 0.1 0.01 0.02 0.04 0.05 0.08 0.2 0.3 0.4 0.5 0.8 1. 0.99 0.965 0.92 0.875 0.85 1. 0.98 0.76 0.705 0.425 0.385 0.29 0.25 0.16 0.575 0.11 0.09 0.07 0.04 0.04 20 Soil PI=0 Damping - Soil with PI=0, OCR=1-8 (Vucetic & Dobry, JG 0.005 0.006 0.003 0.004 0.008 0.01 0.02 0.03 0.04 0.05 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1. 2.9 4.2 4.8 5.4 7.9 3.3 3.7 9.7 11. 12.1 15.2 18.4 20. 21. 21.9 22.4 22.9 23.2 23.6 23.8 19 Soil PI=15 G/Gmax - Soil with PI=15, OCR=1-15 (Vucetic & Dobry, J 0.0007 0.0009 0.001 0.002 0.003 0.004 0.006 0.008 0.01 0.02 0.03 0.04 0.08 0.1 0.2 0.3 0.4 0.6 1. 1. 0.995 0.99 0.97 0.95 0.925 0.875 0.85 0.455 0.815 0.72 0.65 0.6 0.405 0.29 0.22 0.19 0.14 0.95 Soil PI=15 Damping - Soil with PI=15, OCR=1-8 (Vucetic & Dobry, J 19 0.003 0.004 0.005 0.006 0.008 0.01 0.02 0.03 0.04 0.05 0.07 0.1 0.2 0.3 0.4 0.5 0.6 0.7 1. 2.5 4.5 2.8 3.2 3.5 4.1 6.4 7.6 8.4 9.2 10.3 11.5 15.9 17. 14.3 17.6 18.3 18.8 19.9 20 Soil PI=30 G/Gmax - Soil with PI=30, OCR=1-15 (Vucetic & Dobry, J 0.006 0.008 0.001 0.002 0.003 0.004 0.005 0.009 0.01 0.02 0.03 0.04 0.07 0.1 0.2 0.3 1. 0.4 0.6 0.8

0.985 0.97 0.96 0.95 1. 0.995 0.925 0.91 0.9 0.82 0.745 0.7 0.6 0.53 0.42 0.35 0.305 0.24 0.205 0.165 20 Soil PI=30 Damping - Soil with PI=30, OCR=1-8 (Vucetic & Dobry, J 0.002 0.003 0.004 0.005 0.006 0.008 0.01 0.02 0.03 0.04 0.05 0.06 0.08 0.1 0.2 0.3 0.4 0.5 0.7 1. 1.7 2.1 2.5 2.6 2.9 3.3 3.7 5.05 6.4 5.7 10.8 6.9 7.3 8.1 8.7 12.3 16.9 13.3 14.1 15.6 Soil PI=50 G/Gmax - Soil with PI=50, OCR=1-15 (Vucetic & Dobry, J 20 0.006 0.007 0.008 0.009 0.003 0.004 0.005 0.01 0.06 0.02 0.03 0.04 0.08 0.1 0.2 0.3 1. 0.5 0.6 0.8 1. 0.99 0.985 0.98 0.97 0.965 0.96 0.955 0.815 0.75 0.71 0.565 0.48 0.905 0.85 0.67 0.385 0.35 0.3 0.25 20 Soil PI=50 Damping - Soil with PI=50, OCR=1-8 (Vucetic & Dobry, J 0.002 0.003 0.004 0.005 0.006 0.008 0.01 0.02 0.03 0.04 0.05 0.06 0.08 0.1 0.2 0.3 0.4 0.5 0.7 1. 1.6 1.8 2.1 2.3 2.4 2.7 3. 3.7 5.7 4.2 4.6 5. 5.2 6.1 8. 9.2 10.1 10.9 12.2 13.5 20 Soil PI=100 G/Gmax - Soil with PI=100, OCR=1-15 (Vucetic & Dobry, 0.005 0.006 0.007 0.009 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08 0.09 0.1 0.2 0.3 0.5 0.6 0.9 1. 0.985 0.995 0.99 1. 0.96 0.935 0.915 1. 0.9 0.815 0.715 0.635 0.88 0.865 0.85 0.83 0.53 0.49 0.405 0.375 Soil PI=100 Damping - Soil with PI=100, OCR=1-8 (Vucetic & Dobry, 20 0.003 0.005 0.007 0.009 0.01 0.02 0.001 0.03 0.04 0.05 0.06 0.08 0.1 0.2 0.3 0.4 0.5 0.6 0.7 1. 1.2 1.7 2. 2.05 2.5 2.9 1.5 1.9 3.1 3.3 3.5 3.8 4. 5.2 6.1 6.8 7.4 8. 8.6 9.7 Sand S1 G/Gmax - S1 (SAND CP<1.0 KSC) 3/11 1988 9 0.00316 0.01 0.0001 0.000316 0.001 0.0316 0.1 0.316 1. 0.934 0.838 0.672 0.463 0.253 1. 0.978 0.14 0.057 Sand upper Damping for SAND, Upper Bound (Seed & Idriss 1970) 9 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 0.3 1. 0.7 1.2 2.7 5.5 9.9 14.8 21. 25.5 27.9 Sand S2 G/Gmax - S2 (SAND CP=1-3 KSC) 3/11 1988 10

0.0001 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316 1. 10. 0.985 0.952 0.724 0.532 0.332 0.2 0.114 1. 0.873 0.114 11 Sand Avg. Damping for SAND, Average (Seed & Idriss 1970) 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 0.3 1. 3. 10. 0.00162 0.421 1.27 2.6 5.19 8.95 14.4 19.6 24.3 27.3 29.3 9 Sand S3 G/Gmax - S3 (SAND CP>3.0 KSC) 3/11 1988 0.0001 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316 1. 0.782 0.969 0.908 1. 0.991 0.602 0.393 0.266 0.183 9 Sand lower Damping for SAND, Lower Bound (Seed & Idriss 1970) 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 0.3 1. 2.7 0.3 0.4 0.7 1.4 5. 9.8 15.5 20.7 G/Gmax - ROCK (Schnabel 1973) 8 Rock 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 1. 0.99 1. 1. 0.95 0.9 0.81 0.725 0.55 5 Rock Damping for ROCK (Schnabel 1973) 0.01 0.0001 0.001 0.1 1. 0.4 0.8 1.5 3. 4.6 9 Gravel Avg. G/Gmax - GRAVEL, Average (Seed et al. 1986) 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 0.3 1. 1. 0.97 0.87 0.73 0.55 0.37 0.2 0.1 0.05 Damping for Gravelly Soils (Seed et al 1988) 9 Gravel 0.0001 0.0003 0.001 0.003 0.01 0.03 0.1 0.3 1. 0.5 1. 1.75 3. 5.5 9.5 15.5 21. 21. 3 4 5 6 7 8 9 10 10 1 2 Option 2 - Soil Profile Set No. 1 2 10 Soil Profile Set No. 1 1 7 1 0.05 0.112 328.0 0.5 2 7 3.28 0.05 0.112 328.0 3 7 3.28 0.05 0.112 328.0 4 7 6.56 0.05 0.112 328.0 5 7 6.56 0.05 0.112 328.0 6.56 6 7 0.05 0.112 328.0 7 7 26.25 0.05 0.112 328.0 8 7 26.25 0.05 0.112 328.0 9 7 19.68 0.05 0.112 328.0 10 9 2500.0 0.05 0.13 Option 3 - 1995 Kobe CHY090 Mw=6.9 18 3 5400 8192 0.01 sample\chy090.at2 (5F15.11) 4 5 15 1 Option 4 - Assignment of Object Motion to a Specific Sublayer Set No. 1 4

10 1

Option 5 - Number of Iterations & Strain Ratio Set No. 1 0.65 Option 6 - Computation of Acceleration at Specified Sublayers Set No. 1 4 5 6 7 8 9 10 0 0 0 0 0 0 0 0 Option 7 - Computation of Shear Stress or Strain Time History Set No. 1 Stress History Layer No. 1 Strain History Layer No. 1 Option 9 - Response Spectrum Set No. 1 1 0 32.2 0.05 Option 9 - Response Spectrum Set No. 4 5 1 32.2 0.05 Execution will stop when program encounters 0

FLAC-3D INPUT FILE

```
;***** Free Field Static Analysis*******
*****
new
config dyn
set dyn off
gen zone brick size 30 20 30 p0 (0,0,0) &
p1 (30,0,0) &
p2 (0,20,0) &
p3 (0,0,30)
model mohr
;*************DRAINED PARAMETERS****************
prop shear=72e6 bulk=156e6 dens=1800 fric=30 cohes=5e3 dil=0 tens=10e3
set grav 0,0,-9.81
ini xdisp=0 ydisp=0 zdisp=0
fix x y z range z - 0.1 0.1
fix x y range y -0.1 0.1
fix x y range y 19.9 20.1
fix x y range x - 0.1 0.1
fix x y range x 29.9 30.1
ini szz -5.4e5 grad 0 0 18000
ini sxx -2.7e5 grad 0 0 9000
ini syy -2.7e5 grad 0 0 9000
save Vs200ini.sav
solve
save Vs200ini.sav
restore Vs200ini.sav
model elas
;
hist reset
hist unbal
```

```
; for sands drained parameters remain, for clays, silts undrained parameters are used
prop shear=72e6 bulk=3576e6 dens=1800 ;fric=28 cohes=5e3 dil=0 tens=10e3
set dyn on
;set large
ini xvel 0 yvel 0 zvel 0
ini xdisp 0 ydisp 0 zdisp 0
:******* DYNAMIC BOUNDARY CONDITIONS*****
free x y z
apply ff
fix z range y=-0.1 0.1
fix z range y=19.9 20.1
fix z range x=-0.1 \ 0.1
fix z range x=29.9 30.1
fix z range z=-0.1 0.1
ini damp hyst sig3 1.0315 -0.6575 -1.4124
set dyn time 0.00
ini xvel 0 yvel 0 zvel 0
ini xdisp 0 ydisp 0 zdisp 0
:
table 1 read berna2.dat
apply xacc 9.81 hist table 1 range z -.1 .1
;
def _locgp
      p_gp0 = gp_near(15, 10, 0)
      p_gp1 = gp_near(15, 10, 1)
      p_gp2 = gp_near(15, 10, 7)
      p_gp3 = gp_near(15, 10, 8)
      p_gp4 = gp_near(15, 10, 15)
      p_gp5 = gp_near(15, 10, 16)
      p_gp6 = gp_near(15, 10, 22)
      p_gp7 = gp_near(15, 10, 23)
      p_gp8 = gp_near(15, 10, 28)
      p_gp9 = gp_near(15, 10, 29)
end
_locgp
;
```

```
def strain1
        strain1 = gp_xdisp(p_gp1) - gp_xdisp(p_gp0)
        strain2 = gp_xdisp(p_gp3) - gp_xdisp(p_gp2)
        strain3 = gp_xdisp(p_gp5) - gp_xdisp(p_gp4)
       strain4 = gp_xdisp(p_gp7) - gp_xdisp(p_gp6)
        strain5 = gp_xdisp(p_gp9) - gp_xdisp(p_gp8)
end
;
;****Hist Corner of the Building, Point A ****
hist gp xacc 11 6 30 ;hist 2
hist gp xacc 11 6 29
hist gp xacc 11 6 28
. . . . .
. . . . . .
. . . . . .
hist gp xacc 3 3 14
hist gp xacc 3 3 6
;
;*****Shear Stress XZ time Histories under the Building
;****Hist Corner of the Building, Point A ****
hist zone sxz 11 6 30 ;hist 42
hist zone sxz 11 6 29
hist zone sxz 11 6 28
. . . . .
. . . . . .
. . . . . .
hist zone sxz 3 2 22
hist zone sxz 3 2 14
hist zone sxz 3 2 6
;***** Shear Stress YZ Histories at Horizontal Directions*****
; ***** Middle of the Building******
;**** W/B=0.0625*****
hist zone syz 15 10 30 ;hist 378
hist zone syz 15 10 29
hist zone syz 15 10 28
. . . . .
. . . . . .
. . . . . .
hist zone syz 3 2 14
hist zone syz 3 2 6
```

```
;*************Shear Stresses in Short Directions********
```

```
;****** XZ Shear Stressses *********
; **** In the middle, X=14.5 ****
;**** W/L=0.0625*****
hist zone sxz 15 11 30 ;634
hist zone sxz 15 11 29
hist zone sxz 15 11 28
; Displacement history
;****Hist Corner of the Building, Point A ****
hist gp xdis 11 6 30 ;hist 1155
hist gp xdis 11 6 29
hist gp xdis 11 6 28
. . . . .
. . . . . .
. . . . . .
hist gp xdis 15 6 1
hist gp xdis 15 6 2
;
his zone sxz 15 10 0 ;1207
his strain1
;
his zone sxz 15 10 7
his strain2
his zone sxz 15 10 15
his strain3
his zone sxz 15 10 22
his strain4
his zone sxz 15 10 29
his strain5
hist nstep=50
set dyn multi on
plot his 1155 1161 1195
set logfile StruNodePos.log
set log on
```

print sel node pos range x 14.5 15.5 y 9.5 10.5 z 30 42 print sel node pos range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node pos range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node pos range x 14.5 15.5 y 5.5 6.5 z 30 42 set log off set logfile dispHist.log set log on print sel node disp range x 14.5 15.5 y 9.5 10.5 z 30 42 print sel node disp range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node disp range x 14.5 15.5 y 5.5 6.5 z 30 42 solve age 0.1 print sel node disp range x 14.5 15.5 y 9.5 10.5 z 30 42 print sel node disp range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node disp range x 14.5 15.5 y 5.5 6.5 z 30 42 solve age 0.2 solve age 20.8 print sel node disp range x 14.5 15.5 y 9.5 10.5 z 30 42 print sel node disp range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node disp range x 14.5 15.5 y 5.5 6.5 z 30 42 solve age 20.9 z 30 42 print sel node disp range x 14.5 15.5 y 9.5 10.5 print sel node disp range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node disp range x 14.5 15.5 y 5.5 6.5 z 30 42 z 30 42 print sel node pos range x 14.5 15.5 y 9.5 10.5 print sel node pos range x 10.5 11.5 y 5.5 6.5 z 30 42 print sel node pos range x 10.5 11.5 y 9.5 10.5 z 30 42 print sel node pos range x 14.5 15.5 y 5.5 6.5 z 30 42 set log off save Vs200ELAS_FF_SKREQ.sav

;

```
;***** Building Dynamic Analysis******
new
restore Vs200ini.sav
model elas
; for sands drained parameters remain, for clays, silts undrained parameters are used
prop shear=72e6 bulk=3576e6 dens=1800; fric=28 cohes=5e3 dil=0 tens=10e3
;BASE BEAMS
sel beam id 1
          b (11,6,30) e (19,6,30)
                             n=8
sel beam id 1
          b (11,10,30) e (19,10,30)
                             n=8
sel beam id 1
          b (11,14,30) e (19,14,30)
                             n=8
sel beam id 1
          b (11,6,30) e (11,14,30)
                             n=8
sel beam id 1
          b (15,6,30) e (15,14,30)
                             n=8
sel beam id 1
          b (19,6, 30) e (19,14,30)
                             n=8
;STOREY #1
          BEAMS
sel beam id 1
          b (11,6,33) e (19,6,33)
                             n=8
sel beam id 1
          b (11,10,33) e (19,10,33)
                             n=8
sel beam id 1
          b (11,14,33) e (19,14,33)
                             n=8
sel beam id 1
          b (11,6,33) e (11,14,33)
                             n=8
          b (15,6,33) e (15,14,33)
sel beam id 1
                             n=8
sel beam id 1
          b (19,6,33) e (19,14,33)
                             n=8
:STOREY #2 BEAMS
sel beam id 1
          b (11,6,36) e (19,6,36)
                             n=8
sel beam id 1
          b (11,10,36) e (19,10,36)
                             n=8
sel beam id 1
          b (11,14,36) e (19,14,36)
                             n=8
sel beam id 1
          b (11,6,36) e (11,14,36)
                             n=8
          b (15,6,36) e (15,14,36)
sel beam id 1
                             n=8
sel beam id 1
          b (19,6,,36) e (19,14,36)
                             n=8
:STOREY # 3
          BEAMS
sel beam id 1
          b (11,6,39) e (19,6,39)
                             n=8
sel beam id 1
          b (11,10,39) e (19,10,39)
                             n=8
sel beam id 1
          b (11,14,39) e (19,14,39)
                             n=8
sel beam id 1
          b (11,6,39) e (11,14,39)
                             n=8
sel beam id 1
          b (15,6,39) e (15,14,39)
                             n=8
```

```
sel beam id 1 b (19,6,,39) e (19,14,39)
                                      n=8
:STOREY #4 BEAMS
sel beam id 1 b (11,6,42) e (19,6,42)
                                      n=8
sel beam id 1
            b (11,10,42) e (19,10,42)
                                      n=8
sel beam id 1 b (11,14,42) e (19,14,42)
                                      n=8
                                      n=8
sel beam id 1
            b (11,6,42) e (11,14,42)
sel beam id 1
             b (15,6,42) e (15,14,42)
                                      n=8
sel beam id 1
             b (19,6,42) e (19,14,42)
                                      n=8
sel beam id 2 b (11,6,30) e (11,6,42)
                                      n=12
sel beam id 2 b (15,6,30) e (15,6,42)
                                      n=12
sel beam id 2 b (19,6,30) e (19,6,42)
                                      n=12
sel beam id 2 b (11,10,30) e (11,10,42)
                                      n=12
sel beam id 2 b (15,10,30) e (15,10,42)
                                      n=12
sel beam id 2 b (19,10,30) e (19,10,42)
                                      n=12
sel beam id 2 b (11,14,30) e (11,14,42)
                                      n=12
sel beam id 2 b (15,14,30) e (15,14,42)
                                      n=12
sel beam id 2 b (19,14,30) e (19,14,42)
                                      n=12
sel shell id 1 range x 11 19 y 6 14 z 29.9 30.1
sel beam id 1 prop density 5000 emod 10250000000 nu 0.35 xcarea 0.25 &
xciy 5.2E-3 xciz 5.2E-3 xcj 0.01 ; BEAMS (0.50x0.50)
sel beam id 2 prop density 2400 emod 10250000000 nu 0.35 xcarea 0.64 &
xciy 0.034 xciz 0.034 xcj 0.068 ; COLUMNS (0.80x0.80)
sel shell id 1 prop density 5000 iso=(10.25e9, 0.25) thick = 0.30
his unbal
plot his 1
;****Hist Corner of the Building, Point A ****
hist gp xacc 11 6 30; hist 2
hist gp xacc 11 6 29
. . . .
. . . . . .
. . . . . .
hist gp xacc 3 3 14
hist gp xacc 3 3 6
;*****Shear Stress XZ time Histories under the Building
;****Hist Corner of the Building, Point A ****
hist zone sxz 11 6 30 ;hist 42
```

```
hist zone sxz 11 6 29
hist zone sxz 11 6 28
hist zone sxz 11 6 26
. . . .
. . . . . .
. . . . . .
hist zone sxz 3 2 14
hist zone sxz 3 2 6
;***** Shear Stress YZ Histories at Horizontal Directions*****
; ***** Middle of the Building*****
;**** W/B=0.0625*****
hist zone syz 15 10 30 ;hist 378
hist zone syz 15 10 29
hist zone syz 15 10 28
. . . .
. . . . . .
. . . . . .
hist zone syz 3 2 14
hist zone syz 3 2 6
hist gp xacc 11 6 0 ;hist 1146
hist gp xacc 11 6 1
hist gp xacc 11 6 2
:
hist gp xacc 15 10 1
hist gp xacc 15 10 2
hist gp xacc 11 10 1
hist gp xacc 11 10 2
hist gp xacc 15 6 1
hist gp xacc 15 6 2
; Displacement history
;****Hist Corner of the Building, Point A ****
hist gp xdis 11 6 30 ;hist 1155
hist gp xdis 11 6 29
hist gp xdis 11 6 28
hist gp xdis 11 6 26
hist gp xdis 11 6 24
hist gp xdis 11 6 22
hist gp xdis 11 6 14
hist gp xdis 1166
```

;****Hist Center of the Building,Point B **** hist gp xdis 15 10 30 ;hist 1163 hist gp xdis 15 10 29 hist gp xdis 15 10 28 hist gp xdis 15 10 26 hist gp xdis 15 10 24 hist gp xdis 15 10 22 hist gp xdis 15 10 14 hist gp xdis 15 10 6 ;****Hist Center of the Building,Point C **** hist gp xdis 11 10 30 ;hist 1171 hist gp xdis 11 10 29 hist gp xdis 11 10 28 hist gp xdis 11 10 26 hist gp xdis 11 10 24 hist gp xdis 11 10 22 hist gp xdis 11 10 14 hist gp xdis 11 10 6 ;****Hist Center of the Building,Point D **** hist gp xdis 15 6 30 ;hist 1179 hist gp xdis 15 6 29 hist gp xdis 15 6 28 hist gp xdis 15 6 26 hist gp xdis 15 6 24 hist gp xdis 15 6 22 hist gp xdis 15 6 14 hist gp xdis 15 6 6 ;****Hist Free Field,Point F **** hist gp xdis 1 1 30 ;hist 1187 hist gp xdis 1 1 29 hist gp xdis 1 1 28 hist gp xdis 1 1 26 hist gp xdis 1 1 24 hist gp xdis 1 1 22 hist gp xdis 1 1 14 hist gp xdis 116 ;********** Disp at the bottom******** hist gp xdis 11 6 0 ;hist 1195 hist gp xdis 11 6 1 hist gp xdis 11 6 2

```
hist gp xdis 15 10 0 ;hist 1198
hist gp xdis 15 10 1
hist gp xdis 15 10 2
hist gp xdis 11 10 0 ;hist 1201
hist gp xdis 11 10 1
hist gp xdis 11 10 2
hist gp xdis 15 6 0 ;hist 1204
hist gp xdis 15 6 1
hist gp xdis 15 6 2
hist nstep=50
;
step 2000
save Vs200BLDG.sav
;
new
restore Vs200BLDG.sav
hist reset
hist unbal
;
set dyn on
;set large
ini xvel 0 yvel 0 zvel 0
ini xdisp 0 ydisp 0 zdisp 0
:
;******* DYNAMIC BOUNDARY CONDITIONS*****
free x y z
apply ff
fix z range y=-0.1 0.1
fix z range y=19.9 20.1
fix z range x=-0.1 \ 0.1
fix z range x=29.9 30.1
fix z range z=-0.1 0.1
ini damp hyst sig3 1.0315 -0.6575 -1.4124
set dyn time 0.00
ini xvel 0 yvel 0 zvel 0
ini xdisp 0 ydisp 0 zdisp 0
```

```
table 1 read berna2.dat
apply xacc 9.81 hist table 1 range z -.1 .1
def _locgp
       p_gp0 = gp_near(15, 10, 0)
       p_gp1 = gp_near(15, 10, 1)
       p_gp2 = gp_near(15, 10, 7)
       p_gp3 = gp_near(15, 10, 8)
       p_gp4 = gp_near(15, 10, 15)
       p_gp5 = gp_near(15, 10, 16)
       p_gp6 = gp_near(15, 10, 22)
       p_gp7 = gp_near(15, 10, 23)
       p_gp8 = gp_near(15, 10, 28)
       p_gp9 = gp_near(15, 10, 29)
end
_locgp
def strain1
       strain1 = gp_xdisp(p_gp1) - gp_xdisp(p_gp0)
       strain2 = gp_xdisp(p_gp3) - gp_xdisp(p_gp2)
       strain3 = gp_xdisp(p_gp5) - gp_xdisp(p_gp4)
       strain4 = gp_xdisp(p_gp7) - gp_xdisp(p_gp6)
       strain5 = gp_xdisp(p_gp9) - gp_xdisp(p_gp8)
end
;
;****Hist Corner of the Building, Point A ****
hist gp xacc 11 6 30; hist 2
hist gp xacc 11 6 29
hist gp xacc 11 6 28
. . . .
. . . . . .
. . . . . .
hist gp xacc 3 3 14
hist gp xacc 3 3 6
;
;*****Shear Stress XZ time Histories under the Building
;****Hist Corner of the Building, Point A ****
hist zone sxz 11 6 30 ;hist 42
hist zone sxz 11 6 29
hist zone sxz 11 6 28
. . . .
. . . . . .
. . . . . .
```

```
hist zone sxz 3 2 14
hist zone sxz 3 2 6
;***** Shear Stress YZ Histories at Horizontal Directions*****
; ***** Middle of the Building*****
;**** W/B=0.0625*****
hist zone syz 15 10 30 ;hist 378
hist zone syz 15 10 29
hist zone syz 15 10 28
. . . .
. . . . . .
. . . . . .
hist zone syz 3 2 14
hist zone syz 3 2 6
hist gp xacc 11 6 0 ;hist 1146
hist gp xacc 11 6 1
hist gp xacc 11 6 2
hist gp xacc 15 10 1
hist gp xacc 15 10 2
hist gp xacc 11 10 1
hist gp xacc 11 10 2
:
hist gp xacc 15 6 1
hist gp xacc 15 6 2
; Displacement history
;****Hist Corner of the Building, Point A ****
hist gp xdis 11 6 30 ;hist 1155
hist gp xdis 11 6 29
hist gp xdis 11 6 28
hist gp xdis 11 6 26
hist gp xdis 11 6 24
hist gp xdis 11 6 22
hist gp xdis 11 6 14
hist gp xdis 1166
;****Hist Center of the Building,Point B ****
hist gp xdis 15 10 30 ;hist 1163
hist gp xdis 15 10 29
hist gp xdis 15 10 28
hist gp xdis 15 10 26
```

```
hist gp xdis 15 10 24
hist gp xdis 15 10 22
hist gp xdis 15 10 14
hist gp xdis 15 10 6
;
. . . .
. . . . . .
. . . . . .
hist gp xdis 15 6 2
his zone sxz 15 10 0 ;1207
his strain1
his zone sxz 15 10 7
his strain2
his zone sxz 15 10 15
his strain3
his zone sxz 15 10 22
his strain4
his zone sxz 15 10 29
his strain5
hist nstep=50
:
set dyn multi on
plot his 1155 1161 1195
;
set logfile StruNodePos.log
set log on
print sel node pos range x 14.5 15.5 y 9.5 10.5 z 30 42
print sel node pos range x 10.5 11.5 y 5.5 6.5
                                                    z 30 42
print sel node pos range x 10.5 11.5 y 9.5 10.5 z 30 42
print sel node pos range x 14.5 15.5 y 5.5 6.5
                                                    z 30 42
set log off
;
plot create testview
plot current testview
pl add sel geo node off
plot add sel disp max auto
plot set rotation 17 0 31 mag 3
```

```
plot set cen 20 12 34 dist 159
set movie avi step 50 file SKR_BLDG.avi
movie start
:
set logfile dispHist.log
set log on
print sel node disp range x 14.5 15.5 y 9.5 10.5 z 30 42
print sel node disp range x 10.5 11.5 y 5.5 6.5 z 30 42
print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42
print sel node disp range x 14.5 15.5 y 5.5 6.5
                                                      z 30 42
solve age 0.1
. . . .
. . . . . .
. . . . . .
solve age 20.8
print sel node disp range x 14.5 15.5 y 9.5 10.5
                                                  z 30 42
print sel node disp range x 10.5 11.5 y 5.5 6.5
                                                        z 30 42
print sel node disp range x 10.5 11.5 y 9.5 10.5 z 30 42
print sel node disp range x 14.5 15.5 y 5.5 6.5
                                                       z 30 42
solve age 20.9
print sel node disp range x 14.5 15.5 y 9.5 10.5
                                                  z 30 42
print sel node disp range x 10.5 11.5 y 5.5 6.5
                                                        z 30 42
print sel node disp range x 10.5 11.5 y 9.5 10.5
                                                  z 30 42
print sel node disp range x 14.5 15.5 y 5.5 6.5
                                                        z 30 42
print sel node pos range x 14.5 15.5 y 9.5 10.5
                                                  z 30 42
print sel node pos range x 10.5 11.5 y 5.5 6.5
                                                        z 30 42
print sel node pos range x 10.5 11.5 y 9.5 10.5 z 30 42
print sel node pos range x 14.5 15.5 y 5.5 6.5
                                                       z 30 42
set log off
movie finish
save Vs200ELAS_BLDG_SKREQ.sav
;
```

CURRICULUM VITAE

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EDUCATION

- 2000 2003 METU Civil Engineering, Geotechnical Engineering, MS
- 1995 2000 METU Civil Engineering, BS
- 1994 1995 Bolu Atatürk High School

WORK EXPERIENCE

- 2001 2008 METU Department of Civil Engineering Research/Teaching Assistant
- 2000 2001 KANALET Construction Electricity Generation Industry and Commerce Inc., Project Engineer

FOREIGN LANGUAGES

Advanced English

PUBLICATIONS

Cetin K. O., Bakir B. S., Yilmaz R., Gulkan P., Soyoz S., Yilmaz M. T., Unutmaz B., Ozan C. 'Soil investigations and characterization and documentation of surface soils.' Rehabilitation of Turkish Disaster Management Project (TUR/94/006), Turkish Disaster Management Office, 200 pages, April 2002 (in Turkish).

Cetin K. O. Unutmaz B. 'Probabilistic models for the assessment of post cyclic soil deformations.' 9th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability, Albequerque, New Mexico, 2004.

Cetin K. O., Isik N., Unutmaz B. 'Seismically-induced landslide at Degirmendere Nose, Izmir Bay after 1999 Kocaeli (Izmit) – Turkey Earthquake.' J. Soil Dynamics and Earthquake Eng. 24(3): 189-197, 2004.

Cetin K. O., Unutmaz B. 'Seismic performance assessment of earthfill dams: State of the art.' Proceedings of the 1st National Symposium on Dams and Hydropower Plants: 435-445, 2004 (in Turkish).

Unutmaz, B., Cetin, K. O. 'Seismic response and seismic soil liquefaction in embankment dams.' 1st National Dam Safety Symposium and Exhibition, Ankara, 2007. (in Turkish)

Unutmaz, B., Cetin, K. O. 'Effects of soil-structure-earthquake-interaction on seismic soil liquefaction triggering.' 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, 25-28 June 2007.

Cetin, K. O., Horoz, A., Erol, O., Unutmaz B. 'Seismic performance of existing Bakacak landslide after 1999 Düzce (Turkey) Earthquake.' XIV. European Conference on Soil Mechanics and Geotechnical Engineering, Madrid 2007. Bilge, H. T., Yunatcı, A. A., Unutmaz, B., Yunatcı, I., Çetin, K. O. 'Liquefaction Triggering Response under Wave-Induced Cyclic Loading.' 6th National Conference on Marine Engineering, Izmir, Turkey, 2007.

Unutmaz, B., Cetin, K. O. 'Effects of soil-structure-earthquake-interaction on seismic soil liquefaction triggering.' Sixth National Conference of on Earthquake Engineering, Istanbul, Turkey16-20 October 2007 (in Turkish).

Unutmaz, B., Cetin, K. O. 'Effects of soil-structure-earthquake-interaction on seismic soil liquefaction triggering.' Conference on Soil-Structure Interaction in Theory and Practice, Istanbul, Turkey 8-9 November 2007 (in Turkish).

Unutmaz, B., Cetin, K. O. 'Assessment of Structure-Induced Liquefaction Triggering.', Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics IV, Sacramento, CA., July 2008.

Unutmaz, B., Cetin, K. O., Gokyay M.C, 'Simplified frame analysis in seismic soilstructure interaction of buried structures.' 12th Soil Mechanics and Foundation Engineering Conference, Konya, Turkey, 16-17 October 2008 (in Turkish).