İSTANBUL TECHNICAL UNIVERSITY ✦ INSTITUTE OF SCIENCE AND TECHNOLOGY

COMPARATIVE LIQUEFACTION ANALYSES ON ADAPAZARI SOIL

M. Sc. Thesis by

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Department : **Civil Engineering**

Programme: **Soil Mechanics & Geotechnical Engineering**

JUNE 2006

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PREFACE

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NOTATIONS

SPT	: Standard Penetration Test
USCS	: Unified Soil Classification System
SCPTu	: Seismic Piezocone Test
CPT	: Cone Penetration Test
DMT	: Dilatometer Test
PMT	: Pressuremeter Test
DHT	: Downhole Test
SASW	: Spectral Analyses of Surface Waves
SRER	: Seismic Refraction
VST	: Vane Shear Test
BPT	: Becker Penetration Test
CSR	: Cyclic Stress Ratio
CRR	: Cyclic Resistance Ratio
ER	: Energy Ratio
ASTM	: American Society of Testing Materials
FC	: Fine Content
PI	: Plasticity Index

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LIST OF SYMBOLS

u	**:** Pore water pressure
$\mathbf{D_r}$	**:** Relative density
Ip	: Plasticity index
$\mathbf{D_{50}}$	**:** Mean grain diameter (mm)
$\mathbf{D_{10}}$	: 10% grain diameter (mm)
$\mathbf{F_L}$	: Resistance ratio against liquefaction
R	: Dynamic shear strength ratio
L	: Shear stress ratio during an earthquake
$\mathbf{r_w}$	: Modification factor based on earthquake motion properties
$\mathbf{R_L}$	: Cyclic triaxial strength ratio
$\mathbf{r_d}$	: Reduction coefficient in the depth direction of the shear stress ratio during an earthquake
$\mathbf{k_{hc}}$	: Design lateral force coefficient used with the ductility design method
σ_v	: Total overburden pressure
σ_v'	: Effective overburden pressure
x	: Depth from the ground surface (m)
γ_{t1}	: Unit weight (tf/m^3) of the soil shallower than the ground water level
γ_{t2}	: Unit weigh (tf/m^3) of the soil deeper than the ground water level
γ'_{t2}	: Effective unit weight (tf/m^3) of the soil deeper than the ground water level
$\mathbf{h_w}$	: Depth of the ground water level(m)
τ_f	: Shear strength
ϕ'	: Effective angle of internal friction
A, a	: Constants, normally having values of 1,000 and 0.5, respectively
$\mathbf{K_2}$	: A function of the index properties of the soil and is an inverse function of the shear strain amplitude.
G	: Shear modulus
$\mathbf{S_u}$	: Undrained shear strength
σ_m'	: Mean effective stress
$\mathbf{K_o}$	: Lateral earth pressure coefficient
$\mathbf{(N_1)_{60}}$	: Standardized SPT blowcount
$\mathbf{CSR_{eq}}$	: Cyclic stress ratio generated by the anticipated earthquake ground motions at the site
$\mathbf{CSR_{liq}}$	: Cyclic stress ratio required to generate liquefaction
$\mathbf{q_c}$	: Tip resistance
τ_e	: Seismic shear strength
σ_{vo}	: Total overburden pressure
S	: Soil factor
α	: Ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g

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ÖZET

ADAPAZARI ZEMİNİ ÜZERİNDE KARŞILAŞTIRMALI SIVILAŞMA ANALİZİ

Emre SERDAR

Depremler sırasında, yapılar üzerinde meydana gelen en dramatik hasarlardan biri de kaba daneli zeminlerin sebep olduğu sıvılaşmadır. 17 Ağustos 1999, Adapazarı depreminin yapılar üzerinde meydana gelen hasarlardaki yerel zemin koşulları etkisini incelememiz açısından istisnai bir fırsat tanımıştır.

Bu tez çalışmasında, dört farklı yönetmelik, California Sismik Tasarım Yönetmeliği, Eurocode 8, T.C Bayıdırlık ve İskan Bakanlığı Afet Bölgelerinde Yapılacak Yapılarla İlgili Yönetmelik (1998) ve Japon Karayolu Yönetmeliği, için sıvılaşma analizleri yapılmıştır. Tezin asıl amacı, sıvılaşma riski olan bölgelerde yapılacak yapılar için yönetmeliklerde bulunan sıvılaşma kriterlerine gösterilmesi gereken önemi vurgulamak ve kısa dönemde, Afet Bölgelerinde Yapılacak Yapılarla İlgili Yönetmelik' te mevcut olmayan sıvılaşma ile ilgili kısımların belirtilmesi gerekliliğini göstermek ve orta - uzun dönemde ise Türkiye'nin Avrupa Birliğine girmeye çalıştığı bugünlerde Eurocode 8' in Türkçe'ye çevrilerek uygulanmasında gördüğüm yararı belirtmek isterim.

Anahtar Kelimeler: Sıvılaşma, Yönetmelik, Adapazarı.

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ABSTRACT

COMPARATIVE LIQUEFACTION ANALYSES ON ADAPAZARI SOIL

Emre SERDAR

One of the most dramatic causes of damage of structures during earthquakes is the development of liquefaction in saturated cohesionless deposits. In 17 August 1999, Adapazarı earthquake provides an exceptional opportunity to investigate the effects of local soil conditions on damage patterns under strong shaking conditions in areas that experienced ground failure.

In this thesis; the primary goal is, to develop a comparative liquefaction analysis according to Turkish Specification for Structures to be Built in Disaster Areas, Eurocode 8, Japanese and Californian Seismic Codes, at sites undergoing ground failure to clarify that, a careful consideration should be given while we are analyzing and designing of the structures in that region, to suggest that we have to complete the missing parts about liquefaction in Turkish Specification for Structures to be Built in Disaster Areas and to suggest that a translation of Eurocode 8 should be made and should be in force.

Keywords: Liquefaction, Specification, and Adapazarı.

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1. INTRODUCTION

One of the most dramatic causes of damage of structures during earthquakes is the development of liquefaction in saturated cohesionless deposits. These deposits tend to densify when subjected to earthquake loading. However, when saturated, the tendency to densify causes the excess pore water pressure to increase. Consequently, the effective stress of soil decreases. The cohesionless deposit will suffer a great deal of loss of strength until the excess pore water pressure has a chance to dissipate. The phenomenon of pore pressure build, following with the loss of soil strength is known as liquefaction.

The study of liquefaction has become extensive since the Niigata and the Alaska earthquakes occurred in 1964. The study that has been considered as a major breakthrough on the subject of liquefaction is the one conducted by Seed and Idriss. They proposed a procedure to evaluate the liquefaction resistance of soils based on Standard Penetration Test (SPT) blow counts. The procedure is known as the "Simplified Procedure". This procedure has become a worldwide standard practice. The procedure has evolved over the years as considerable efforts have been devoted to the study of liquefaction. Many efforts have been done to develop this procedure, especially in relation to in situ tests (Youd, et al., 2001).

The primary importance of the site conditions or the effects of the subsurface layers on the ground motion characteristics during earthquakes has been realized for a long period of time. In the art or rather the science of microzonation, this aspect of earthquake engineering has been studied in certain detail. Even though, there are numerous examples pointing out clearly the predominant influence of the local site conditions on the structural damage observed during past earthquakes, there appears to be some controversy among the researchers and engineers in assessing the magnitude of this effect. Some of the basic reasons for diversity of the approaches proposed in this field may be attributed to the interdisciplinary nature of earthquake engineering. The seismologists and geologists due to their scientific formation are more interested and involved with tectonic phenomena causing earthquakes and the source characteristics. On the other hand, engineers are faced with the problem of analyzing and designing of the structures that need to be earthquake resistant. In this respect the question imposed in relation to the magnitude, duration and frequency content of the acceleration on the ground surface whether the local site conditions or the source characteristics are main controlling factors. At the present, in the majority of the answers given to such a question, though the local site conditions are considered as the primary factor (Ansal 1999).

In 17 August 1999, Adapazarı earthquake (M_w = 7.4) provides an exceptional opportunity to investigate the effects of local soil conditions on damage patterns under strong shaking conditions in areas that experienced ground failure (http://peer.berkeley.edu 2000).

The primary goal of this study is to develop a comparative liquefaction analysis for 30 field logs investigated at Adapazarı region, according to Turkish Specification for Structures to be Built in Disaster Areas, Eurocode 8, Japanese and Californian Seismic Codes. At sites undergoing ground failure, we have to clarify that, a careful consideration should be given while we are analyzing and designing the structures in that region.

2. LIQUEFACTION

2.1 Definition of Liquefaction

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for tremendous amounts of damage in historical earthquakes around the world.

Liquefaction occurs in saturated soils, that is, soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure is relatively low. However, earthquake shaking can cause the water pressure to increase to the point where the soil particles can readily move with respect to each other (www.ce.washington.edu 2000).

The strength that a saturated soil can mobilize to resist shearing along a given plane depends on the effective or intergranular pressure on the plane and the effective coefficient of friction. The shearing resistance or strength τ_f may be written:

$$\tau_f = \sigma'.\tan\phi' \tag{2.1}$$

In which σ' is the effective stress and ϕ' is the effective angle of internal friction. In saturated sand the intergranular normal stress σ' is defined as:

$$\sigma' = \sigma - u \tag{2.2}$$

Where:

σ : Total normal stress

u: Pore water pressure

Then,

$$\tau_f = (\sigma - u).\tan\phi' \tag{2.3}$$

If the pore water pressure, u, increases, while the total stress σ remains constant, the shear strength τ_f across any plane of failure decreases independent of the friction angle ϕ'. When $u = \sigma$, then $\tau_f = 0$, and the sand has lost all its shear strength and is said to have liquefied. The sand is sometimes considered to have liquefied when large strains occur under applied loads. In soil mechanics practice, the term "soil liquefaction" may be defined by two criteria. One defines liquefaction in terms of loss of strength and material transformation of a granular material into a fluid. An alternate definition is expressed in terms of the amount of strain or deformation that is unacceptable from a structural viewpoint.

2.2 Flow Liquefaction and Cyclic Mobility

The term liquefaction has actually been used to describe a number of related phenomena. Because the phenomena can have similar effects, it can be difficult to distinguish between them. The mechanisms causing them, however, are different. These phenomena can be divided into two main categories: flow liquefaction and cyclic mobility (www.ce.washington.edu 2000).

2.2.1 Flow Liquefaction

Flow liquefaction is a phenomenon in which the static equilibrium is destroyed by static or dynamic loads in a soil deposit with low residual strength. Residual strength is the strength of a liquefied soil. 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. After this relatively small disturbance, the static driving force caused by gravity, becomes greater than the frictional resisting force and causes acceleration. The path that brings an unstable state is analogous to the static or dynamic disturbance that triggers flow liquefaction - in both cases, a relatively small disturbance precedes an instability that allows gravity to take over and produce large, rapid movements (www.ce.washington.edu 2000).

2.2.2 Cyclic Mobility

Cyclic mobility is a liquefaction phenomenon, triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic mobility develop incrementally because of static and dynamic stresses that exist during an earthquake. Lateral spreading, a common result of cyclic mobility, can occur on gently sloping and on flat ground close to rivers and lakes (www.ce.washington.edu 2000).

2.3 Factors Affecting Liquefaction

The major factors associated with the liquefaction of saturated cohesionless soils are: initial relative density, cyclic shear stress level, initial (static) shear stress level, initial effective confining pressure, drainage conditions, and the number of cyclic shear stress applications, or duration of shaking. Of additional importance are fines content and soil grain characteristics such as particle size, shape, and gradation. Soil structure, the fabric as a result of previous history, is known to be a significant parameter, but it is difficult to define or sometimes even recognize and, hence, its effects are difficult to quantify.

The foregoing factors reflect the physical properties of the soil, the initial stress conditions, soil stratification, and the characteristics of the applied earthquake motions. Many of these items are difficult to control precisely in the laboratory and impossible to evaluate reliably in the field. A brief discussion follows on some of the more significant factors affecting liquefaction.

2.3.1 Dynamic Shear Stress Level

The fundamental concept of liquefaction is based upon the coupling of shear strain and volumetric strain exhibited by soils. The process of pore pressure buildup, leading to liquefaction under cyclic loading, is dependent upon the volumetric strain response under applied shear stresses. The residual increment of pore water pressure generated by an applied dynamic shear stress cycle is, under undrained conditions, related to the shear strain which is, in turn, related to the magnitude of that stress cycle. Actual earthquake motions may have components in all three principal directions. The most critical stresses from a liquefaction viewpoint arise from vertically propagating horizontal shear waves. Vertical stress components are not considered significant since these are of a dilatational nature and completely absorbed by the pore water.

2.3.2 Dynamics of Earthquake Shear Stress

Earthquake ground motions generally consist of a number of randomly distributed peak stress cycles of varying shapes and magnitudes. Difficulties involved in analyzing the various random earthquake ground motions have led to an attempt to express earthquake records in terms of an equivalent number of uniform stress cycles (Lee and Chan, 1972). The number of significant cycles in a particular earthquake record depends directly upon the frequency content and the duration of loading. These, in turn, are related to the magnitude of the earthquake, the distance to its epicenter, and the nature of the materials through which the stress waves must propagate.

It has been noted by Peacock and Seed (1968) and Yoshimi and Oh-Oka (1975) that the frequency of vibration, at least within 0.17 to 12 cps, which covers the range of earthquake motions, at least in overburden, is of secondary importance. The actual shape of the stress pulse used in laboratory test simulations has been found not to be critical; i.e., whether or not it is in the form of a sine wave, a saw tooth, or other form. It is common to present soil susceptibility to liquefaction in terms of number of uniform stress cycles causing liquefaction under a specified level of applied shear stress. The number of stress cycles a specimen can withstand increases almost exponentially with a decrease in shear stress level for any constant confining stress level and relative density.

There are some weaknesses in simulating random earthquake motions in terms of uniform cycles. For example Martin, Finn and Seed (1975) note that the tendency for dry sands to undergo volume changes is a direct function of dynamic shear strain level. But dynamic shear strain level is a function of soil modulus of rigidity G, which in turn depends upon the effective confining stress level and, hence, the pore water pressure generated. Since the pore pressure level existing at the time of application of a specific peak is very important, the relative position of any peak in a sequence of loading cycles is significant. Consideration of the effects of stress reversals also suggests that the peculiar characteristics of the loading history (i.e., the symmetry of the stress record, etc.) may be significant. Ishihara, Tatsuoka and

Yasuda (1975) note that ground motion inputs in which the maximum peak occurs early are less critical than input records for which the peaks are more uniformly distributed (i.e., vibratory as opposed to shock loadings).

2.3.3 Relative Density

The relative density of a soil is one of the major factors regarding liquefaction potential of cohesionless sands. Relative density is stressed here rather than absolute density since it is actually the pore volume of the soil compared to its minimum and maximum possible pore volumes that is of significance. The denser a soil, the lower is its tendency toward volume contraction during shearing; the lower is the pore pressure which will be generated; hence, the more unlikely to liquefy.

Relative density can be controlled in the laboratory using reconstructed samples; however, in typical field situations with complex stratification, relative density may lose its meaning. A factor such as relative density has meaning only in uniform soil conditions; actual experience shows that natural soil deposits are quite often very heterogeneous.

It is also conceivable that there is an upper limit of relative density, D_r, above which a soil under field behavior will either no longer tend to compress and generate pore pressures or will, immediately upon commencing yielding, undergo volume increases prohibit liquefaction. Soils are not likely to liquefy at relative densities above 75 percent. Although cyclic mobility (temporary loss of strength) can occur at relative densities up to 100 percent, it is thought that negligible distortions occur in this range at least prior to any drainage or pore water redistribution (Castro and Poulos, 1976). It is impossible to define an upper limit to D_r beyond which liquefaction will not occur; nevertheless, it appears it is less probable for a value of Dr above about 80 percent.

2.3.4 Initial Effective Confining Stress

The resistance of a soil to liquefaction under cyclic loading has been noted to be a function of the effective confining pressure, prior to application of shear. Field observations of liquefaction of level ground have generally been limited to relatively shallow depths, in few cases below 50 or 60 feet. This was noted by Kishida (1969) who observed in the 1964 Niigata earthquake that liquefaction did not occur where effective overburden stress exceeds 2 kg/cm^2 (27 psi). Although there is a trend toward reduced liquefaction potential at higher stresses, the observed field cases are very limited and cannot be expected to apply in all situations. Liquefaction evaluations must not omit regions simply because the effective pressure exceeds some empirical value.

Because it is difficult to estimate lateral stress levels in the field, the vertical effective stress is used to define the level of confinement, but much work is available (Seed and Idriss, 1971) to indicate that the ratio of lateral to vertical stress K_o and, hence, the true degree of confinement actually existing in the field are of major importance.

The shear stress level required to cause liquefaction in remolded sand specimens at a relative density less than 80 percent has been found to vary linearly with confining stress levels (Seed and Lee, 1966, and Peacock and Seed, 1968). Therefore it has been found convenient to normalize the effects of dynamic cyclic shear stress level with the value of initial effective confining stress. It is important to recognize that the use of this normalized ratio may not always be applicable to field conditions, particularly where strongly developed structure or cementation is present. Thus, this simplification in treatment of liquefaction potential may not be valid in all circumstances. Soils near the ground surface, under very small degrees of confinement could have resistance to liquefaction in excess of that suggested from test results acquired at higher confining stress levels. This might be associated with material fabric or structure, or, in effect, equivalent to a previous stress history or over-consolidation pressure. This exists for hydraulic fill sands and has been suggested by Meehan (1976).

2.3.5 Drainage Conditions

The rate at which pore water pressure is permitted to dissipate from within a soil body has a major influence upon whether or not liquefaction can occur, particularly under cyclic loading (Wong, Seed, and Chan, 1974). Since the rate of pore pressure dissipation is known to be a function of the square of the longest drainage path, the detailed geometry of the soil profile is important. A study of the interrelationships between different layer compressibilities and permeabilities on the occurrence of liquefaction has been presented by Yoshimi and Kuwabara (1973). This analytical study, based upon solutions to the Terzaghi one-dimensional consolidation problem, illustrates that liquefaction will propagate easily from a lower liquefied layer to an overlying permeability than the initially liquefied striation.

A useful tool for investigating the influence of drainage on potentially liquefiable soil strata is discussed by Seed, Martin and Lysmer (1975). Effective stress computer codes provide a numerical solution of the diffusion equation with a pore pressure generating term included to represent the earthquake-generated pore-pressure increases. It is possible to investigate the influence of length of drainage path, stratification, water table and saturation level variations, different permeabilities, compressibilities, densities, and other conditions.

2.3.6 Grain Size Characteristics

Limits on gradation curves can define bounds separating liquefiable and non-liquefiable soils. The lower boundary on particle size shows the influence of the fines in decreasing the tendency of the soils to density. Plastic fines make more difficult for the sand particles to come free of each other and seek denser arrangements, (NRC 1985). Fines content has been shown to be a factor in the occurrence of liquefaction and is delineated in field prediction relationships. The upper boundaries are significant because they are associated with the permeability of coarser material. Thus, increased drainage and dissipation of pore pressure can occur. Both the grain size and distribution can control the pore pressure buildup and dissipation.

2.3.7 Previous Stress History

The influence of previous stress history is of major interest in liquefaction studies. Finn, Bransby and Pickering (1970) present laboratory data showing that a sample, which has previously liquefied, is more susceptible to liquefaction. A specimen of sand at an initial relative density of 50 percent and an initial effective isotropic confining pressure of 200 kN/m^2 was subjected to cyclic loading with stress reversals. The specimen first underwent limited flow or cyclic mobility under the extensional portion of the 25th load cycle. This specimen then underwent several additional cycles wherein it reliquefied, flowed, and then restablilized. After a total of 29 load cycles, the specimen was permitted to drain, and was reconsolidated under an effective spherical pressure of 200 kN/m^2, which yielded a relative density of 60 percent. Upon resumption of cyclic loading the specimen was noted as reliquefying during the extensional segment of its first loading cycle, in spite of its increased relative density value over that of the initial test sequence. Based on such information, it is possible that the number of loading cycles required to cause liquefaction is substantially reduced by previous episodes of liquefaction. The conclusion is that judgment is necessary in interpreting liquefaction potential of sites which underwent previous liquefaction.

2.4 Parameters Indirectly Affecting Liquefaction

There is a family of soil parameters which, while not related to the liquefaction process directly, do influence the liquefaction potential. These are the response parameters which dictate how a soil will respond to applied stress. For example, since volumetric changes and, hence, liquefaction potential can be related to the distortional strain levels which a soil undergoes (Martin, Finn, and Seed, 1975), the shear stiffness or modulus of rigidity of a soil under a specific load level is of particular concern. Earthquake motions can be either amplified or attenuated, depending upon characteristics of the soil profile (and its interaction with the frequency content of the disturbing earthquake) which, in turn, depend upon the values of the stiffness and damping parameters involved.

Since many treatments of earthquake-induced liquefaction deal with vertically transmitted horizontal shear waves, one approach to analysis requires only a value for the shear modulus, G, together with a damping coefficient, to account for the energy absorption of the soil. Extensive experimental work dealing with these two parameters has been carried out by Seed and Idriss (1970), and Hardin and Drnevich (1970). These studies permit characterizing the shear response parameters of soil in terms of the basic soil index properties and the existing stress and strain states. For example, the shear modulus value for clean granular soils is related to void ratio, mean effective stress, maximum cyclic shear strain amplitude, and number of loading cycles (some soils have an additional dependency upon overconsolidation ratio, degree of saturation, and plasticity index). Soil damping, particularly in cohesionless soils, is at least partially due to relative movements between soil particles and, hence, is hysteric. The contribution by dry friction to the damping ratio should be substantially independent of strain rate. For analytical expediency damping is sometimes represented by an equivalent viscous damping. For soils, damping is generally specified as a percentage of critical damping, and measured in terms of specific damping capacity, related to the ratio of the area within a hysteric loop during a load cycle and the maximum stored energy during the cycle. Seed and Idriss (1970) have derived expressions for damping ratio as a function of strain level, number of cycles, frequency, mean effective stress, and the other index properties mentioned in reference to shear modulus.

The shear modulus is noted as increasing with density and confining pressure and decreasing with shear strain amplitude. Damping coefficients on the other hand increase with shear strain amplitude and appear to decrease with confining stress and increased density. Previous stress history is noted as increasing shear stiffness and decreasing damping. Shear modulus of granular materials is treated as:

$$G = A.K_2.(\sigma)^a \tag{2.4}$$

Where, A and a are constants, normally having values of 1,000 and 0.5, respectively, and K_2 is a function of the index properties of the soil and is an inverse function of the shear strain amplitude.

It has been found (Seed and Idriss, 1970; Hardin and Drnevich, 1970) that shear modulus values at any strain level may be normalized in terms of maximum shear modulus to permit a generalized relationship for many soil materials to be collapsed into a single relationship. Damping ratios, as mentioned, were found to vary as functions of soil index properties as well as the stress and strain states. Although cohesive materials have been treated in the same format as granular materials, their soil models have not been found quite as satisfactory in this context. It is more expedient to normalize the shear modulus of clays in terms of the undrained shear strength, S_u, in the form of G/S_u versus shear strain amplitude. It is again possible to collapse the various shear modulus relationships into a single curve by normalizing them by the maximum way, modulus values determined at very small strain levels, such as by measuring shear wave velocities in the field, can be used to predict the shear modulus under design loading conditions. Damping ratios for clays have been studied less extensively than for granular materials. Little data is available for materials other than sans and clays, but available information indicates that coarser grained materials such as gravels may be expected to behave as sands (Seed and Idriss, 1970; Hardin and Drnevich, 1970). Peats are generally treated in the same format as clays.

2.5 Potentially Liquefiable Soil Types

The quantitative liquefaction evaluation procedures in practice are based on the behavior of predominantly sandy soils. These methods have been validated with field studies over the last three decades, and a consensus has emerged regarding their application (Youd and Idriss 1997). Understanding the liquefaction behavior of silty and gravelly soils has, however, substantially lagged. Recommendations for these soils have been largely "rules of thumb" tempered by field observations made after earthquakes. For example, cohesive soils with a fine content greater than 30%, and whose fines either classify as "clays" based on the Unified Soil Classification System (USCS), or have a plasticity index (PI) of greater than 30%, are not generally considered potentially susceptible to soil liquefaction (Seed 1992; Youd and Idriss 1997).

The influence of fine-grained soil on the liquefaction resistance of predominantly sandy soils is a topic that has received considerable attention over the past decade (Ishihara 1993). Laboratory testing of silts has been performed, but to a very limited scale and with varying results. Recent examination of fine-grained soil behavior during earthquakes and the results of laboratory tests reveal that uniformly graded loose sandy soils that contain as much as 25% to 30% non-plastic to low plasticity fines may be highly liquefiable. Finn and others provide a review of the design and analysis of structures in potentially liquefiable silty soils.

In addition to sandy and silty soils, some gravelly soils and even rockfills are potentially vulnerable to liquefaction. A number of well-documented field case histories confirm that gravelly soils can liquefy. In recent years, the liquefaction behavior of gravelly soils has been investigated in the laboratory. Most coarse, gravelly soils are relatively free draining; if the voids are filled with finer particles, or the surrounded soils are less pervious, then drainage may be impeded and cyclic pore pressure generation or liquefaction becomes more likely. Similarly, when they are of considerable thickness and lateral extent, deposits of coarse gravelly soils may not be capable of dissipating pore pressures and may be vulnerable to potential liquefaction.

Field evidence has shown that most liquefied gravelly soils are sand-gravel composites. They present the results of cyclic triaxial tests on soils with increasing percentages of gravel content. They conclude that sand-gravel composites show an increase in cyclic strength with increased gravel content, even though the relative density of the composite is constant. This result raises questions about the relationship between laboratory test results and actual field behavior. Currently, the best techniques available for quantitative evaluation of the liquefaction resistance of coarse gravelly soils are those described by Harder, Seed, and several papers contained in Prakash and Dakoulas. These methods involve two primary evaluation procedures: (1) the use of very large-scale Becker Hammer penetration resistance correlations, or (2) corrections to penetration resistances obtained by the SPT. Application and support for the former method is provided by Harder, where case histories are provided to examine the application of the Becker penetration test for characterizing the liquefaction potential of gravelly soils.

3. EVALUATION OF LIQUEFACTION SUSCEPTIBILITY: IN-SITU AND LABORATORY PROCEDURES

3.1 Liquefaction Hazard Evaluation

The liquefaction of a loose, saturated granular soil occurs when the cyclic shear stresses and strains passing through the soil deposit induce a progressive increase in excess hydrostatic pore water pressure. During an earthquake, the cyclic shear waves that propagate upward from the underlying bedrock induce the tendency for the loose sand layer to decrease in volume. If undrained conditions are assumed, an increase in pore water pressure and an equal decrease in the effective confining stress are required to keep the loose sand at constant volume.

The degree of excess pore water pressure generation is largely a function of the initial density of the sand layer, and the intensity and duration of seismic shaking. In loose to medium dense sands, pore pressures can be generated which are equal in magnitude to the confining stress. In this state, no effective or intergranular stress exists between the sand grains and a complete loss of shear strength is temporarily experienced.

The following types of phenomena can result from soil liquefaction:

1. Catastrophic flow failures,
2. Lateral spreading and ground failures,
3. Excessive settlement,
4. Loss of bearing capacity,
5. Increase in active lateral earth pressures behind retaining walls, and
6. Loss of passive resistance in anchor systems.

Two phenomena commonly occur in soils when loading cyclically: liquefaction and cyclic mobility. Because both lead to a substantial rise in pore water pressures and large strains in the laboratory, they are often confused. Generally, liquefaction occurs only in specimens that are highly contractive, whereas cyclic mobility may occur in specimens from any initial state. The difference between these phenomena and the factors affecting them, as observed in the laboratory, are summarized by Castro and Poulos (1977). In an effort to clarify some of the terminology associated with liquefaction, some definitions are provided below (Seed 1979; Youd and Perkins 1987).

3.2 Triggering of Liquefaction or Initial Liquefaction

Denotes a condition where, during the course of cyclic stress applications, the residual pore water pressure on completion of any full stress cycle becomes equal to the applied confining pressure. The development of initial liquefaction has no implications concerning the magnitude of the soil deformations. However, it defines a condition that is a useful basis for assessing various possible forms of subsequent soil behavior.

3.3 Initial Liquefaction with Limited Strain Potential or Cyclic Mobility

Denotes a condition in which cyclic stress applications develop a condition of initial liquefaction. Subsequent stress applications cause limited strains to develop because of the remaining resistance of the soil to deformation or because the soil dilates; the pore pressure drops and the soil stabilizes under the applied loads. However, once the cyclic stress applications stop and if they return to the zero stress condition, there will be a residual pore water pressure in the soil equal to the overburden pressure, and this will lead to an upward flow of water in the soil which could have deleterious consequences for overlying layers.

3.4 Liquefaction with Large Strain Potential

Denotes a condition where a soil will undergo continued deformation at a constant low residual stress or with no residual resistance, due to the buildup of high pore water pressures that reduce the effective confining pressure to a very low value. The pore pressure buildup may be due to either static or cyclic stress applications.

In order to be susceptible to liquefaction, the soil must be fully saturated and subjected to a sudden or rapid loading such as that of an earthquake. The resistance of a soil to liquefaction is dependent on a combination of the soil properties, environmental factors, and characteristics of the earthquake. Soil properties such as the mineralogy, gradation or grain-size distribution, and particle shapes (e.g., angularity) all affect the soil's liquefaction resistance. The six principal environmental factors affecting a soil's intrinsic resistance to cyclic pore pressure generation or liquefaction during seismic loading are shown below (Seed 1992).

1. Relative Density: The resistance to cyclic pore pressure generation, as well as residual undrained strength, increase with the relative density of the soil. Relative density is the most important factor governing the liquefaction resistance of a cohesionless soil.

2. Geologic Age: The time under a sustained overburden can significantly increase the liquefaction resistance of some soils over time.

3. Prior Cyclic Load History: Prior seismic excitation can increase liquefaction resistance. This effect can also, however, be erased by more recent seismic excitation causing full or nearly full liquefaction.

4. Overconsolidation: Overconsolidation and the associated increased lateral effective confining stress can increase liquefaction resistance by increasing the coefficient of lateral earth pressure (K_o), which in turn increases the overall mean effective stress (σ_m').

5. Soil Fabric: The method of deposition and compaction can have a significant influence on liquefaction resistance.

6. Drainage Characteristics: The ability to rapidly dissipate excess pore pressures, which is a function of both the permeability of the soil and the drainage boundary conditions imposed by the surrounding soils, will affect the liquefaction resistance.

One additional factor with a potentially significant impact on liquefaction resistance is the effective confining stress. Resistance to cyclic pore pressure generation and/or liquefaction increases with increased effective confining stress. As a result, site conditions involving near-surface water tables or phreatic surfaces tend to represent an inherently more liquefaction-susceptible condition than those with a deeper water table.

The evaluation of liquefaction hazard is generally performed in several stages: (1) preliminary geological/geotechnical site evaluation, (2) quantitative evaluation of liquefaction potential and its potential consequences, and if necessary, (3) development of mitigation and foundation remediation programs. The scope of the investigation required is dependent not only on the nature and complexity of geologic site conditions, but also on the economics of a project and on the level of risk acceptable for the proposed structure or development.

3.5 Current Trends and Challenges in In-Situ Testing

In complement to conventional drilling and sampling operations for site exploration, direct measurements from in-situ tests are increasingly used to derive soil properties and parameters for geotechnical analysis and design. The interpretations of initial geostatic stress state and stress-strain-strength-flow characteristics are calibrated with laboratory test data obtained from high-quality samples, but at high costs. Considerable gains in efficiency, economy, and time are to be obtained by in-situ devices, including cone, dilatometer, pressure meter, and vane. Current interpretation procedures use a hybrid of empirical, analytical, experimental, and/or numerical methods, whereas a comprehensive integrated numerical simulation of all field tests is needed. Of particular interest, the seismic piezocone test with dissipation phases (SCPTu) offers an optimal collection of five separate readings (q_t, f_s, u_b, t_{50}, and V_s) of soil behavior within a single sounding, and therefore should be adopted for routine geotechnical investigations.

Soils are extremely complex four-dimensional (x, y, z, t) materials in their constituent behavior, having varied mineralogical and geological constituents, three-phase particulate components, and logarithmic size distributions. In addition, the aspects of initial stress state, nonlinear stiffness, strength, anisotropy, permeability, drainage characteristics, and geological behavior provide a formidable task for all those charged with conducting a meaningful site investigation. Yet, these geomaterials must be characterized adequately before any new foundation, embankment, roadway, earthen dam, tunnel, or excavation is constructed on or within the ground. A thorough investigation of a particular geologic formation should consider the initial anisotropic-preconsolidated geostatic stress state and nonlinear stress-strain strength behavior, drainage paths, and flow behavior under dry/saturated, drained/undrained, as well as partially-saturated conditions. Since Mother Nature has bequeathed such a wide diversity of particulates, mineralogies, fabrics, cementitious agents, and packing arrangements, a fully global numerical model which integrates all aspects of the ground may be difficult to formulate in the near future. At present, the best practice is to employ a combination of drilling, sampling, and in-situ field testing during geotechnical site exploration. Figure 3.1 shows the chart for the evaluation of the soil characteristics (Paul W. Mayne 2004).

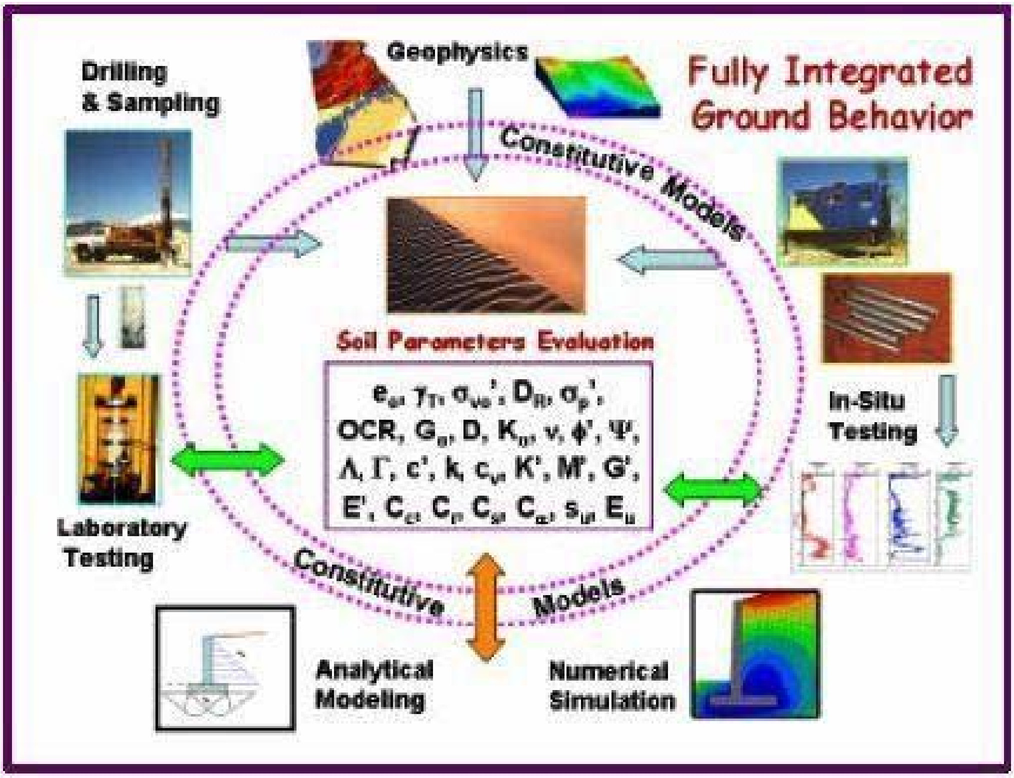

Figure 3.1 Evaluation of Soil Parameters in Nature

A good number of different in-situ tests are available for site investigation with the most common being the standard penetration test (SPT), cone penetration (CPT), piezocone (CPTu), flat plate dilatometer (DMT), pressuremeter (PMT), and vane shear test (VST). For measurements of mechanical waves, especially the shear wave, the geophysical methods include: crosshole (CHT), downhole (DHT), seismic reflection (SRFL), and spectral analysis of surface waves (SASW), as well as recent improvements in seismic refraction (SRFR). For most geotechnical projects, the full suite of drilling & sampling, laboratory and in-situ testing cannot be implemented because of time and costs. Depending upon the nature of geologic setting and level of the proposed construction, perhaps only a select number of lab tests (i.e., index, consolidation, direct shear, triaxial, permeability) and one or two of the basic in-situ tests (i.e., SPT, CPT, CPTu, DMT, PMT, VST) can be implemented. For these tests, the tasks of soil parameter interpretation can be handled by empirical, closed-form analytical, numerical, or experimental methods. In many cases, an assortment of these different methods is adopted in practical applications. Figure 3.2 shows some in-situ testing methods.

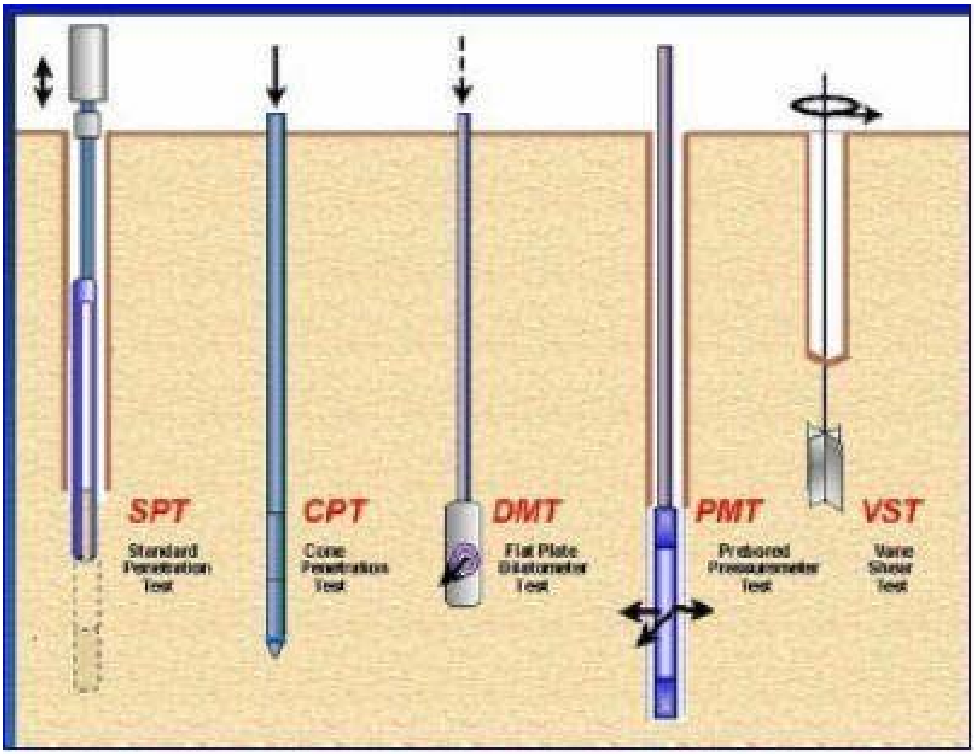

Figure 3.2 Some In-Situ Testing Methods

3.6 Selection of Empirical Method for Liquefaction Assessment

Methods for assessing liquefaction potential reviewed above are based on field performance of liquefiable soil deposits in past earthquakes. These empirical methods rely on some in situ measure of the liquefaction resistance of a soil. While any of a number of in situ soil tests could be used to evaluate liquefaction potential, most of the effort to develop suitable criteria has been based on one of four test measurements: Standard Penetration Test (SPT) blowcounts, Cone Penetration Test (CPT) tip resistance, Becker Penetration Test (BPT) blowcounts, and in situ shear wave velocity.

3.6.1 Approach Used To Develop Empirical Methods

Regardless of the in situ test employed, the approach for developing an empirical criterion for liquefaction assessment is basically the same. Sites subject to possible soil liquefaction in past earthquakes are studied and a database (or "catalog") is compiled of soil deposits that did or did not liquefy. For each case study:

1. Based on the field evidence, a judgment is made as to whether or not liquefaction occurred.

2. A representative measure of the in situ soil strength is determined.

3. The shear stresses induced in the soil by the earthquake are estimated.

A liquefaction assessment criteria is formulated by then attempting to separate conditions (represented by normalized shear stress and strength parameters) where a soil liquefied from those conditions where no liquefaction was observed.

In compiling data on sites subjected to possible soil liquefaction, a critical consideration is how to distinguish between soil layers that did and did not liquefy during an earthquake. For nearly all of the available assessment methods, this distinction is made on the basis of surface manifestations of soil liquefaction (Seed et al. 1985; Liao et al. 1988; Fear and McRoberts 1995; Stark and Olson 1995). Liquefaction is judged to have occurred if sand boils, ground cracking, lateral ground movements, settlement or translation of structures, bearing capacity failures, or uplifting of buried pipes and tanks is observed. If no such surface evidence is observed, the site is assumed to have not liquefied. Defining "liquefaction" in this manner is consistent with the definition adopted in this study. However, at some sites with no apparent liquefaction, deeper soils could have liquefied without producing surface evidence. Ishihara (1985) investigated the conditions where evidence of liquefaction in deeper layers is suppressed by the intact overburden soil. However, Youd and Garris (1994; 1995) have shown that Ishihara's findings are not valid for sites subject to lateral spreading.

For each case study, a representative index of the liquefaction resistance of the soil deposit is needed. The in situ penetration resistance can be used because the same factors that contribute to cyclic shear strength will increase the resistance to penetration. In addition, when surface evidence indicates liquefaction, the specific subsurface soil layer that liquefied must be identified before a representative index value can be defined. Seed and his co-workers (1985) appear to have identified the critical, liquefied deposit at each site as the soil layer with the lowest penetration resistance, and then compiled the average SPT blowcount measured in this critical layer. However, re-examinations of their data set indicate that the minimum blowcount in a boring was frequently compiled (Liao and Whitman 1986; Fear and McRoberts 1995). While the lowest observed blowcount may be an erroneous or spurious data point, this approach has merit because the sublayer with minimum penetration resistance will also have the least resistance to liquefaction. In addition to using the minimum blowcount, both Liao and Whitman (1986) and Fear and McRoberts (1995) considered each boring at a site as one case study to decrease the correlation among observations in their data catalogs. Because judgment is involved, different blowcount values are inevitably compiled for the same sites in the various liquefaction catalogs. To indicate the severity of the seismic loading imparted to a liquefiable soil, the cyclic shear stress generated by an earthquake is usually estimated from the peak horizontal accelerations at the ground surface. Most empirical liquefaction assessment methods are based on the cyclic stress ratio (CSR), which is calculated from the maximum horizontal surface acceleration (a_{max}) generated by an earthquake at a given site. Values of CSR in the available liquefaction catalogs are often based on fairly approximate values of a_{max} estimated from empirical attenuation equations (Liao and Whitman 1986; Ambraseys 1988).

Finally, the accumulation of excess pore pressures and shear strains is affected by static shear stresses in a slope that should be considered in a liquefaction assessment (Seed and Harder 1990). However, most liquefaction assessment models were developed from case studies of fairly level ground where the static shear stresses are very small. In using these methods to evaluate liquefaction potential at sites with gentle slopes, "the effects of the initial sustained shear stress on the triggering of liquefaction are considered negligible and ignored for all practical purposes" (Ishihara 1993). Hence, the static shear stress imparted by a mild surface slope in a lateral spread can be ignored in predicting soil liquefaction.

3.6.2 Methods based on the Standard Penetration Test

The most comprehensive liquefaction data catalogs are based on Standard Penetration Test (SPT) blowcounts (N_{SPT}). Starting in the 1970's, H. B. Seed and his colleagues worked to develop a reliable method for assessing liquefaction potential based on SPT data. Their framework for SPT-based assessments of liquefaction potential was developed in a series of papers that includes Seed and Idriss (1971), Seed et al. (1977), Seed (1979), Seed and Idriss (1982), and Seed et al. (1983). Significant contributions were also suggested in the work of Tokimatsu and Yoshimi (1983). This research culminated in the liquefaction criteria published by Seed et al. (1985).

The empirical chart published by Seed et al. (1985) is based on a standardized SPT blowcount, $(N_1)_{60}$, and the cyclic stress ratio (CSR). To get $(N_1)_{60}$, the measured N_{SPT} is corrected for the energy delivered by different hammer systems and normalized with respect to overburden stress. Boundary curves separating liquefied from unliquefied soils, in terms of CSR and $(N_1)_{60}$, were conservatively drawn to encompass nearly all observed cases of liquefaction in the data catalog. Three separate boundary curves were presented for clean to silty sands. To consider the effects of earthquake magnitude on the duration of strong shaking, magnitude scaling factors were specified. Over the last decade, the empirical method given by Seed et al. (1985), sometimes referred to as the “simplified procedure”, has been widely used for evaluating potential soil liquefaction in North America and around the world.

Recently, Fear and McRoberts (1995) carefully re-examined this liquefaction database and found that, while fines content affects the liquefaction resistance of a soil, this effect may be less pronounced than indicated by Seed et al. (1985). The analysis by Fear and McRoberts also suggests a transition zone, instead of a single boundary line, for separating conditions leading to severe liquefaction damage from those with no apparent liquefaction. Overall, Fear and McRoberts (1995) conclude that the liquefaction criteria established by Seed et al. (1985) will, as intended, conservatively predict liquefaction in some cases where no liquefaction damage would be observed.

In the empirical charts proposed by Seed and his colleagues, boundary lines between liquefied and unliquefied conditions were drawn subjectively. Several researchers have suggested using statistical analyses to construct these empirical lines more objectively. Liao et al. (1988) performed a statistical regression analysis to systematically develop a liquefaction criteria in terms of CSR and $(N_1)_{60}$. For a liquefaction probability of 0.18, the model proposed by Liao et al. (1988) agrees fairly well with the boundary line drawn by Seed et al. (1985). However, Liao and his colleagues found that their data does not support using fines content as a continuous variable; instead, they developed two models, one for clean sands and a second for silty sands.

Many of the empirical methods for liquefaction resistance rely on magnitude scaling factors published by Seed et al. (1983). These magnitude scaling factors were developed from cyclic laboratory test data and are based on a representative number of uniform load cycles in different magnitude earthquakes. Ambraseys (1988) points out that the idea of an equivalent, uniform stress cycle may oversimplify differences in ground motions from different magnitude earthquakes. Using essentially the same data as Seed et al. (1985), Ambraseys developed an empirical liquefaction criteria expressed directly in terms of CSR, $(N_1)_{60}$, and earthquake magnitude (M_w). Significantly, the analysis by Ambraseys indicates that Seed's magnitude scaling factors poorly represent the field data on liquefaction. Using a larger data catalog, Loertscher and Youd (1994) found that the magnitude scaling factors used by Seed et al. (1983) are significantly conservative for moderate-sized (M = 5 to 7) earthquakes.

Other researchers have developed SPT-based empirical methods that directly use the earthquake magnitude and source distance to represent the seismic energy imparted to the soil, instead of the cyclic stress ratio (based on a_{max}) and magnitude scaling factor. Two models of this type, which also use corrected $(N_1)_{60}$ values, are given by Liao et al. (1988) and Law et al. (1990). Because estimates of peak surface accelerations are not required, empirical correlations based on earthquake magnitude and distance are easier to use in a liquefaction assessment. However, because the attenuation of seismic energy varies in different geologic regimes, these methods are not necessarily valid for geographic regions other than those represented in the data used to develop the model. Liquefaction criteria based on site-specific estimates of surface accelerations are thus more easily applied in different geographic regions (Liao et al. 1988).

Finally, liquefaction assessments in Japan are often performed using the SPT-based, empirical method specified in the Japanese bridge design code. This method was developed in Japan from a large number of cyclic triaxial tests on soil samples with a known SPT penetration resistance (Ishihara 1985; 1993). Hence, this empirical method is based largely on laboratory tests, where sample disturbance is a potential issue, in contrast to the direct correlation with field behavior used in the other methods described here.

3.6.3 Methods Based On Other in -Situ Tests

The Cone Penetration Test (CPT) yields a continuous profile of penetration resistance and is thus well-equipped for detecting thin, liquefiable layers within a larger, stable soil deposit. Early CPT-based empirical methods for liquefaction evaluations were developed by converting SPT blowcounts in the available liquefaction case studies to equivalent CPT tip resistances. Models of this type include those proposed by Seed et al. (1983), Robertson and Campanella (1985), and Seed and De Alba (1986). In a different approach, Mitchell and Tseng (1990) used a model of cone penetration together with laboratory test data to suggest a liquefaction criteria using the CPT. However, these methods suffer from a lack of direct correlation between the measured CPT tip resistance and observed field performance of liquefiable soils in earthquakes. Using data from sites mostly in China, Shibata and Teparaksa developed a CPT-based liquefaction criterion that was based directly on field performance data. Using a more extensive database, Stark and Olson (1995) also developed an empirical method based on measured CPT tip resistances. Stark and Olson used a normalized tip resistance and drew bounding curves between liquefied and unliquefied states for clean sand, silty sand, and sandy silt.

Because large gravel particles interfere with the penetration of both the SPT sampler and the cone penetrometer, the SPT and CPT are not reliable for evaluating the liquefaction potential of gravelly soil deposits. To overcome this problem, the Becker Penetration Test (BPT) has been used for investigating the liquefaction potential of gravelly soils in North America (Harder 1996). The BPT involves driving a large diameter (168 mm recommended), closed-end casing into the ground using a double-acting diesel hammer. The number of hammer blows is typically recorded for every 30 cm of penetration. Using empirical correlations, BPT blowcounts can be converted to equivalent SPT blowcounts as discussed by Harder (1996). The equivalent-NSPT values can then be used to evaluate the potential for soil liquefaction using the SPT-based methods discussed in this chapter. Unfortunately, additional research and development is needed to further standardize the BPT and improve interpretations for liquefaction susceptibility.

Many of the same factors that contribute to the liquefaction resistance of a soil deposit (density, confinement, stress history, geologic age, etc.) also influence the velocity of traveling shear waves (Finn 1991; Robertson et al. 1992). Moreover, the shear wave velocity of a soil deposit can be measured economically with surface geophysics; this is particularly advantageous in evaluating gravelly soils that are difficult to penetrate or sample. Hence, several researchers have attempted to correlate liquefaction potential with in situ shear wave velocity. Both Robertson et al.

(1992) present correlations, in terms of a normalized shear wave velocity and cyclic stress ratio, which were developed directly from a limited number of field cases. Other methods for evaluating liquefaction potential based on shear wave velocities. However, additional data is needed to validate and improve the proposed correlations between shear wave velocity and liquefaction resistance.

3.7 In-Situ Liquefaction Resistance: The Cyclic Resistance Ratio of Soil

The cyclic resistance ratio (CRR) is defined as the ability of the soil to resist the shear stresses induced by the earthquake. The CRR can be determined through empirical relationships based largely on SPT and/or CPT resistance, or laboratory tests.

Once the equivalent uniform cyclic shear stress ratios resulting from the earthquake loading (CSR_{eq}) have been calculated at each point of interest, the next step is to evaluate the resistance of the in situ materials to cyclic pore pressure generation or accumulation of cyclic shear strain. This constitutes evaluation of the resistance to triggering of potential liquefaction failure; defined as sufficient pore pressure or strain accumulation to bring the material to a condition at which undrained residual (or .steady state.) strength will control behavior. The evaluation of in situ liquefaction resistance can be accomplished using either the SPT or CPT resistance data.

3.7.1 Cyclic Resistance Ratio Based on Standard Penetration Tests

The first step in evaluating the potential for soil liquefaction is to compute corrected values of $(N_1)_{60}$ from the measured SPT blowcounts. Here is the list for drilling and SPT procedures recommended:

1. Boring diameter of 66 to 115 mm (2.5 to 4.5 inch).
2. Borehole filled with drilling mud or cased to full depth.
3. Drilling method: wash boring with side discharge bit or rotary boring with side or upward discharge bit. Clean the bottom of the borehole (maximum allowable heave of 70 mm) before perform SPT. Hollow stem auger techniques are not recommended unless extreme care is taken to avoid heave and disturbance.

4. Standard sampler of 51 mm (2.00 inch) outside diameter, 35 mm (1.38 inch) inside diameter, and at least 457 mm (18 inch) long. If the sampler is made to hold a liner, a liner must be in place.
5. Record number of blows for each 150 mm (6 inch) of penetration. N_{SPT} is the number of blows for penetration from 150 to 450 mm (6 to 18 inch) from bottom of the borehole.

The most common technique for estimating the CRR is based on empirical relationships with the normalized SPT blowcount, $(N_1)_{60}$. The relationship is depicted by empirical curves plotted by Seed and others (1985), which divides sites that liquefied historically from those that did not on the basis of $(N_1)_{60}$. The relationship between CSR_{eq} and $(N_1)_{60}$ for M $_{7.5}$ earthquakes is illustrated in Figure 3.3. The points on the figure represent case studies where the cyclic stress ratios have been calculated following earthquakes. In practice it is common to use the chart to obtain the CRR of the sandy soil based on field SPT data. Given the $(N_1)_{60}$ values, the CRR (or τ_{av}/σ_{vo}' as indicated in Figure 3.3) can be determined using the appropriate curve. Alternatively, the practitioner can utilize the chart to determine the minimum SPT penetration resistance required for a given factor of safety against liquefaction. In this case the CSR_{eq} is used to enter the chart and the corresponding $(N_1)_{60}$ values for a factor of safety of one is determined. The latter approach is common when developing specifications for remedial soil improvement. Note that the ratio τ_{av}/σ_{vo}' provided in Figure 3.3 can refer to either CRR or CSR_{eq}, depending on the approach employed.

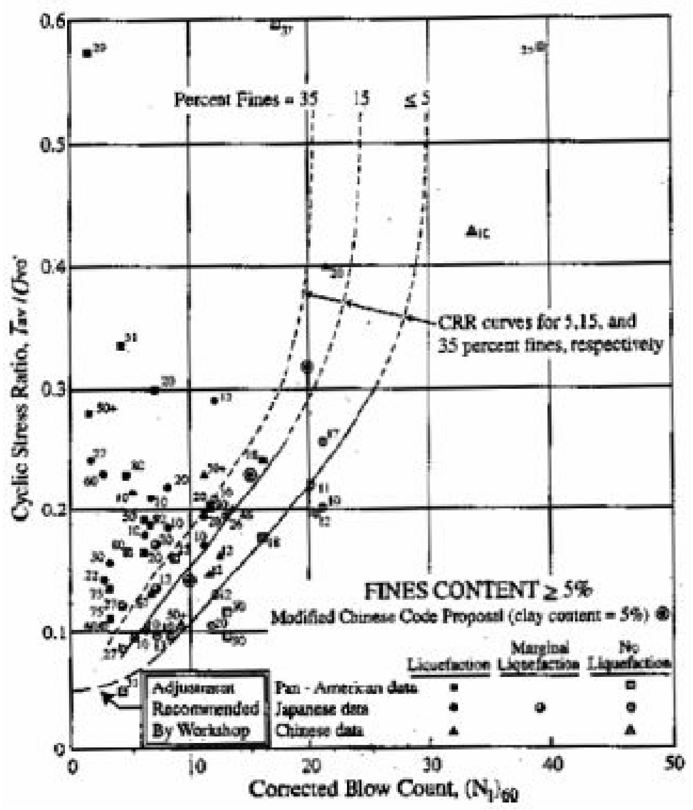

Figure 3.3 Empirical Relationships between the Cyclic Stress Ratio Initiating Liquefaction and $(N_1)_{60}$ Values for Silty Sands in M $_{7.5}$ Earthquakes (Youd and Idriss 1997).

3.7.2 Corrected SPT Blowcounts

The measured SPT blowcount (N_{SPT}) is first normalized for the overburden stress at the depth of the test and corrected to a standardized value of $(N_1)_{60}$. Using the recommended correction factors given by Robertson and Fear (1996), the corrected SPT blowcount is calculated with:

$$(N_1)_{60} = N_{SPT}.C_N.C_E.C_B.C_S.C_R \tag{3.1}$$

The first correction factor (C_N) normalizes the measured blowcount to an equivalent value under one atmosphere of effective overburden stress:

$$C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}} \le 2.0 \tag{3.2}$$

Where σ'_{vo} is the vertical effective stress at the depth of N_{SPT} and P_a is one atmosphere of pressure (101.325 kPa) in the same units as σ'_{vo}. The maximum value of 2.0 limits C_N at depths typically less than 1.5 m. The factor C_E is used to correct the measured SPT blowcount for the level of energy delivered by the SPT hammer. Using 60% of the theoretical maximum energy as a standard, this correction is given by:

$$C_E = \frac{Actual\ Energy\ Delivered\ to\ the\ Top\ of\ the\ Drill\ Rod}{0.60 * Theoritical\ Max.\ SPT\ Hammer\ Energy} = \frac{ER}{60} \qquad (3.3)$$

Where ER is the energy ratio and the theoretical maximum SPT hammer energy is 4200 lb (from 140 weight dropping 30 inches in each blow). The energy ratio (ER) should be measured for the particular SPT equipment used. When such measurements are unavailable, the energy ratio and correction factor can be estimated from the average values given by Seed et al. (1985):

Table 3.1

Country	**Hammer Type**	**Hammer Release**	**ER**	**C_E**
United States	Safety	Rope and pulley	60	1.00
United States	Donut	Rope and pulley	45	0.75
Japan	Donut	Rope and pulley, special throw release	67	1.12
Japan	Donut	Free Fall	78	1.30

The third correction factor, C_B, is for borehole diameters outside the recommended range. The following values are recommended (Robertson and Fear 1996):

Table 3.2

Diameter of Borehole	**C_B**
65 to 115 mm	1.00
150 mm	1.05
200 mm	1.15

The fourth correction factor, C_S, is for SPT samplers used without a sample liner. If the split spoon sampler is made to hold a liner but is used without one, the measured blowcount should be corrected with C_S=1.2. Otherwise, C_S=1.0 for a standard sampler.

The last correction factor is C_R, which is used to correct for the loss of energy through reflection in short lengths of drill rod. In the recommendations, values of the correction factor C_R are given for ranges of rod length. For the analysis of case studies, these recommended values of C_R were approximated with a linear equation:

Table 3.3

z	$\mathbf{C_R}$
$z \leq 3$ m	0.75
3 m$<z<$9 m	(15+z)/24
$z \geq 9$ m	1.00

Below you can find a simplified flowchart for evaluating the liquefied thickness of soil based on SPT blowcounts.

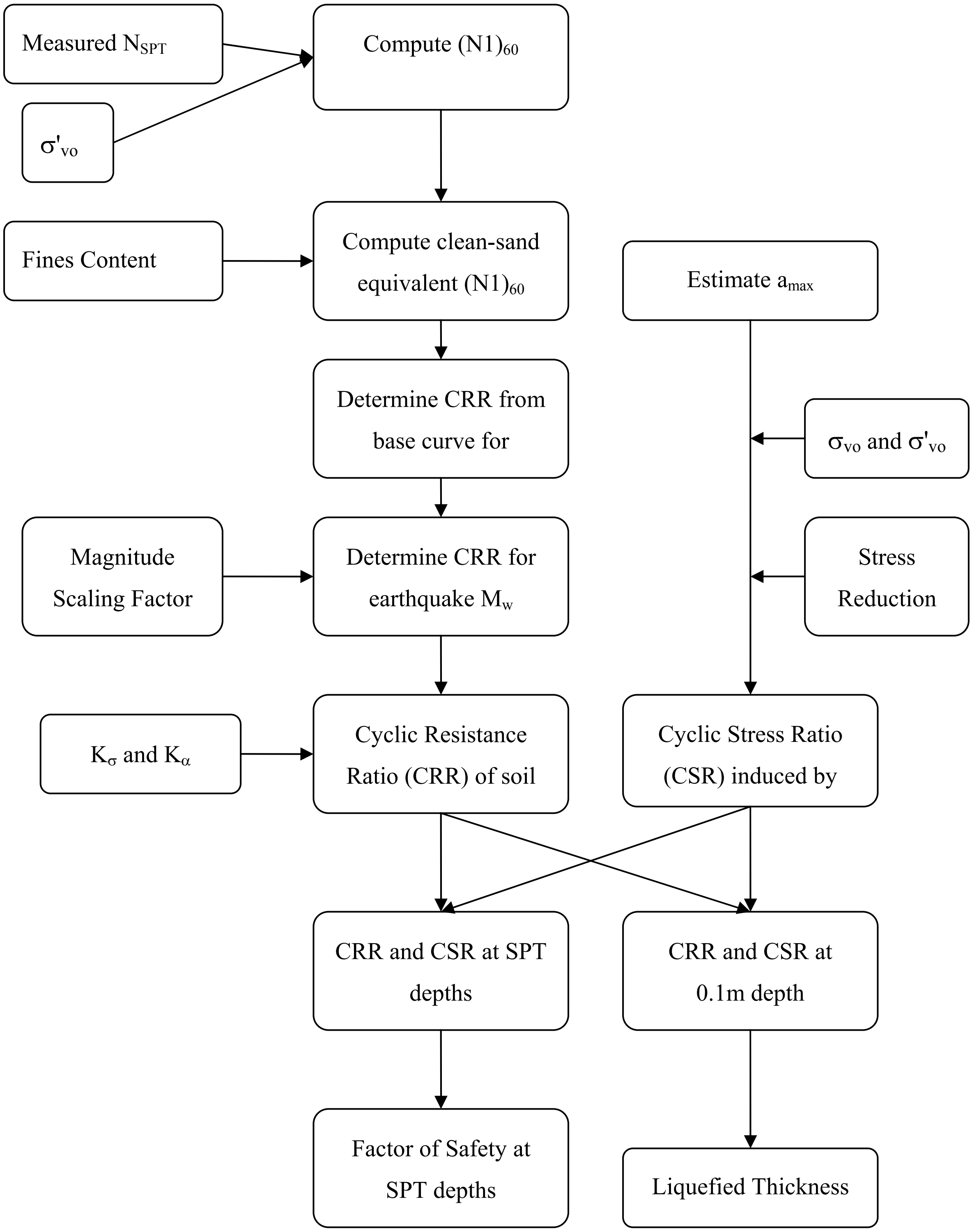

Figure 3.4 Flowchart for evaluating the liquefied thickness of soil based on SPT blowcounts.

4. CODES USED FOR THE ANALYSES

4.1 Californian Code

Given the highly variable nature of Holocene deposits that are likely to contain liquefiable materials, most sites will require borings to determine whether liquefiable materials underlie the project site. Borings used to define subsurface soil properties for other purposes (e.g., foundation investigations, environmental or groundwater studies) may provide valuable subsurface geologic and/or geotechnical information.

The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. However, cohesive soils with:

1. A clay content (percent finer than 0.005 mm) less than 15 percent,
2. A liquid limit less than 35 percent, and
3. A moisture content of the in-place soil that is greater than 0.9 times the liquid limit (i.e., sensitive clays),

are vulnerable to significant strength loss under relatively minor strains (Seed and others, 1983). Although not classically defined as “liquefaction” and so not addressed by these Guidelines, these soils represent an additional seismic hazard that, if present, should be addressed.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction. Most gravelly soils drain relatively well, but when: (a) their voids are filled with finer particles, or (b) they are surrounded by less pervious soils, drainage can be impeded and they may be vulnerable to cyclic pore pressure generation and liquefaction. Gravelly geologic units tend to be deposited in a more-turbulent depositional environment than sands or silts, tend to be fairly dense, and so generally resist liquefaction. Accordingly, conservative “preliminary” methods may often suffice for evaluation of their liquefaction potential. For example, gravelly deposits which can be shown to be pre-Holocene in age (older than about 11,000 years) are generally not considered susceptible to liquefaction.

In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, liquefaction hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths. If it can be demonstrated that any potentially liquefiable materials present at a site:

1. Are currently unsaturated (e.g., are above the water table),
2. Have not previously been saturated (e.g., are above the historic-high water table), and
3. Are highly unlikely to become saturated (given foreseeable changes in the hydrologic regime),

then such soils generally do not constitute a liquefaction hazard that would require mitigation. Note that project development, changes in local or regional water management patterns, or both, can significantly raise the water table or create zones of perched water. Extrapolating water table elevations from adjacent sites does not, by itself, demonstrate the absence of liquefaction hazards, except in those unusual cases where a combination of uniformity of local geology and very low regional water tables permits very conservative assessment of water table depths. Screening investigations should also address the possibility of local "perched" water tables, the raising of water levels by septic systems, or the presence of locally saturated soil units at a proposed project site.

Relatively thin seams of liquefiable soils (on the order of only a few centimeters thick), if laterally continuous over sufficient area, can represent potentially hazardous planes of weakness and sliding, and may thus pose a hazard with respect to lateral spreading and related ground displacements. Thus, the screening investigation should identify nearby free faces (cut slopes, streambanks, and shoreline areas), whether on or off-site, to determine whether lateral spreading and related ground displacements might pose a hazard to the project. If such features are found, the quantitative evaluation of liquefaction usually will be warranted because of potential life-safety concerns.

Even when it is not possible to demonstrate the absence of potentially liquefiable soils or prove that such soils are not and will not become saturated, it may be possible to demonstrate that any potential liquefaction hazards can be adequately mitigated through a simple strengthening of the foundation of the structure, as described in the mitigation section of this chapter, or other appropriate methods.

If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location which might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate than those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. Direct in-situ relative density measurements, such as the ASTM D 1586-92 (Standard Penetration Test [SPT]) or ASTM D3441-94 (Cone Penetration Test [CPT]).

2. Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). Current groundwater level data, including perched water tables, may be obtained from permanent wells, driller's logs and exploratory borings. Historical groundwater data can be found in reports by various government agencies, although such reports often provide information only on water from production zones and ignore shallower water.

3. Non-standard penetration test data. It should be noted that correlation of non-standard penetration test results (e.g., sampler size, hammer weight/drop, hollow stem auger) with SPT resistance is very approximate, and so requires

very conservative interpretation, unless direct SPT and non-standard test comparisons are made at the site and in the materials of interest.

4. Geophysical measurements of shear-wave velocities.
5. “Threshold strain” techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards is very low.

4.1.1 Quantitative Evaluation of Liquefaction Resistance

Liquefaction investigations are best performed as part of a comprehensive investigation. These Guidelines are to promote uniform evaluation of the resistance of soil to liquefaction.

4.1.1.1 Detailed Field Investigations

Engineering geologic investigations should determine:

1. The presence, texture (e.g., grain size), and distribution (including depth) of unconsolidated deposits;
2. The age of unconsolidated deposits, especially for Quaternary Period units (both Pleistocene and Holocene Epochs);
3. Zones of flooding or historic liquefaction; and,
4. The groundwater level to be used in the liquefaction analysis, based on data from well logs, boreholes, monitoring wells, geophysical investigations, or available maps. Generally, the historic high groundwater level should be used unless other information indicates a higher or lower level is appropriate.

The engineering geologic investigations should reflect relative age, soil classification, three-dimensional distribution and general nature of exposures of earth materials within the area. Surficial deposits should be described as to general characteristics (including environment of deposition) and their relationship to present topography and drainage. It may be necessary to extend the mapping into adjacent areas. Geologic cross sections should be constrained by boreholes and/or trenches when available.

4.1.1.2 Geotechnical Field Investigation

The vast majority of liquefaction hazards are associated with sandy and/or silty soils. For such soil types, there are at present two approaches available for quantitative evaluation of the soil's resistance to liquefaction. These are: (1) correlation and analyses based on in-situ Standard Penetration Test (SPT) (ASTM D1586-92) data, and (2) correlation and analyses based on in-situ Cone Penetration Test (CPT) (ASTM D3441-94) data. Both of these methods have some relative advantages. Either of these methods can suffice by itself for some site conditions, but there is also considerable advantage to using them jointly.

Seed and others (1985) provide guidelines for performing "standardized" SPT, and also provide correlations for conversion of penetration resistance obtained using most of the common alternate combinations of equipment and procedures in order to develop equivalent "standardized" penetration resistance values $(N_1)_{60}$. These "standardized" penetration resistance values can then be used as a basis for evaluating liquefaction resistance.

Table 4.1 Comparative advantages of SPT and CPT methods

SPT Advantages	CPT Advantages
Retrieves a sample. This permits identification of soil type with certainty, and permits evaluation of fines content (which influences liquefaction resistance). Note that CPT provides poor resolution with respect to soil classification, and so usually requires some complementary borings with samples to more reliably define soil types and stratography.	Provides continuous penetration resistance data, as opposed to averaged data over discrete increments (as with SPT), and so is less likely to "miss" thin layers and seams of liquefiable material.
Liquefaction resistance correlation is based primarily on field case histories, and the vast majority of the field case history database is for in-situ SPT data.	Faster and less expensive than SPT, as no borehole is required.

Cone penetration test (CPT) tip resistance (q_c) may also be used as a basis for evaluation of liquefaction resistance, by either (a) direct empirical comparison between q_c data and case histories of seismic performance (Olsen, 1988), or (b) conversion of qc-values to "equivalent" $(N_1)_{60}$-values and use of correlations between $(N_1)_{60}$ data and case histories of seismic performance. At present, Method (b), conversion of q_c to equivalent $(N_1)_{60,}$ is preferred because the field case history data base for SPT is well-developed compared to CPT correlations. A number of suitable correlations between q_c and $(N_1)_{60}$ are available (e.g., Robertson and Campanella, 1985; Seed and De Alba, 1986). These types of conversion correlations depend to some extent on knowledge of soil characteristics (e.g., soil type, mean particle size (D_{50}), fines content). When the needed soil characteristics are either unknown or poorly defined, then it should be assumed that the ratio

$$\frac{q_c\left(kg/cm^2\right)}{N\left(blows/feet\right)} \tag{4.1}$$

is approximately equals to 5 for conversion from q_c to "equivalent" N-values.

4.1.1.3 Geotechnical Laboratory Testing

The use of laboratory testing (e.g., cyclic triaxial, cyclic simple shear, cyclic torsional tests) on "undisturbed" soil samples as the sole basis for the evaluation of in-situ liquefaction resistance is not recommended, as unavoidable sample disturbance and/or sample densification during reconsolidation prior to undrained cyclic shearing causes a largely unpredictable, and typically unconservative, bias to such test results. Laboratory testing is recommended for determining grain-size distribution (including mean grain size D_{50}, effective grain size D_{10}, and percent passing #200 sieve), unit weights, moisture contents, void ratios, and relative density.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction (Evans and Fragasy, 1995, Evans and Zhou, 1995). Most gravelly soils drain relatively well, but when their voids are filled with finer particles, or they are surrounded (or "capped") by less pervious soils, drainage can be impeded and they may be vulnerable to liquefaction. Gravelly soils tend to be deposited in a more turbulent environment than sands or silts, and are fairly dense, and so are generally resistant to liquefaction. Accordingly, conservative "preliminary evaluation" methods (e.g., geologic assessments and/or shear-wave velocity measurements) often suffice for evaluation of their liquefaction potential. When preliminary evaluation does not suffice, more accurate quantitative methods must be used. Unfortunately, neither SPT nor CPT provides reliable penetration resistance data in soils with high gravel content, as the large particles impede these small-diameter penetrometers. At present, the best available technique for quantitative evaluation of the liquefaction resistance of coarse, gravelly soils involves correlations and analyses based on in-situ penetration resistance measurements using the very large-scale Becker-type Hammer system (Harder, 1988).

4.1.2 Evaluation of Potential Liquefaction Hazards

The factor of safety for liquefaction resistance has been defined as:

$$Factor\ of\ Safety = \frac{CSR_{liq}}{CSR_{eq}} \tag{4.2}$$

Where CSR_{eq} is the cyclic stress ratio generated by the anticipated earthquake ground motions at the site, and CSR_{liq} is the cyclic stress ratio required to generate liquefaction (Seed and Idriss, 1982). For the purposes of evaluating the results of a quantitative assessment of liquefaction potential at a site, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk. This factor of safety assumes that high-quality, site-specific penetration resistance and geotechnical laboratory data were collected, and that ground-motion data from DMG (Petersen and others, 1996) were used in the analyses. If lower factors of safety are calculated for some soil zones, then an evaluation of the level (or severity) of the hazard associated with potential liquefaction of these soils should be made.

Such hazard assessment requires considerable engineering judgment. The following is, therefore, only a guide. The assessment of hazard associated with potential liquefaction of soil deposits at a site must consider two basic types of hazard:

1. Translational site instability (sliding, edge failure, lateral spreading, flow failure, etc.) that potentially may affect all or large portions of the site; and
2. More localized hazard at and immediately adjacent to the structures and/or facilities of concern (e.g., bearing failure, settlement, localized lateral movements).

As Bartlett and Youd (1995) have stated: “Two general questions must be answered when evaluating the liquefaction hazards for a given site:

1. “Are the sediments susceptible to liquefaction?”; and
2. “If liquefaction does occur, what will be the ensuing amount of ground deformation?””

4.2 Eurocode 8 (prEN 1998-1:2003 & prEN 1998-5:2003)

EN 1998 applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes:

1. Human lives are protected;
2. Damage is limited; and
3. Structures important for civil protection remain operational.

The random nature of the seismic events and the limited resources available to counter their effects are such as to make the attainment of these goals only partially possible and only measurable in probabilistic terms. The extent of the protection that can be provided to different categories of buildings, which is only measurable in probabilistic terms, is a matter of optimal allocation of resources and is therefore expected to vary from country to country, depending on the relative importance of the seismic risk with respect to risks of other origin and on the global economic resources.

4.2.1 General Rules, Seismic Actions and Rules for Buildings

4.2.1.1 Identification of Ground Types

Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 4.2 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

Table 4.2 Ground Types

Ground Type	Description of Stratigraphic Profile	Parameters		
		vs_{30} (m/s)	N_{SPT} (blows/30cm)	C_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium/dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180-360	15-50	70-250
D	Deposits of loose-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vs > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	<100	-	10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

4.2.1.2 Potentially Liquefiable Soils

A decrease in the shear strength and/or stiffness caused by the increase in pore water pressures in saturated cohesionless materials during earthquake ground motion, such as to give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil, shall be hereinafter referred to as liquefaction.

An evaluation of the liquefaction susceptibility shall be made when the foundation soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water table level, and when the water table level is close to the ground surface. This evaluation shall be performed for the free-field site conditions (ground surface elevation, water table elevation) prevailing during the lifetime of the structure.

Investigations required for this purpose shall as a minimum include the execution of either in situ Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT), as well as the determination of grain size distribution curves in the laboratory.

For the SPT, the measured values of the blowcount N_{SPT}, expressed in blows/30 cm, shall be normalized to a reference effective overburden pressure of 100 kPa and to a ratio of impact energy to theoretical free-fall energy of 0,6. For depths of less than 3 m, the measured N_{SPT} values should be reduced by 25%.

Normalization with respect to overburden effects may be performed by multiplying the measured N_{SPT} value by the factor $(100/\sigma'_{vo})^{1/2}$, where σ'_{vo} (kPa) is the effective overburden pressure acting at the depth where the SPT measurement has been made, and at the time of its execution. The normalization factor $(100/\sigma'_{vo})^{1/2}$ should be taken as being not smaller than 0.5 and not greater than 2.

Energy normalizations requires multiplying the blowcount value obtained by the factor ER/60, where ER is one hundred times the energy ratio specific to the testing equipment.

For buildings on shallow foundations, evaluation of the liquefaction susceptibility may be omitted when the saturated sandy soils are found at depths greater than 15 m from ground surface.

The liquefaction hazard may be neglected when $\alpha \cdot S < 0.15$ and at least one of the following conditions is fulfilled:

1. The sands have a clay content greater than 20% with plasticity index PI > 10;
2. The sands have a silt content greater than 35% and, at the same time, the SPT blowcount value normalized for overburden effects and for the energy ratio $N_{1(60)} > 20$;
3. The sands are clean, with the SPT blowcount value normalized for overburden effects and for the energy ratio $N_{1(60)} > 30$.

If the liquefaction hazard may not be neglected, it shall as a minimum be evaluated by well-established methods of geotechnical engineering, based on field correlations between in situ measurements and the critical cyclic shear stresses known to have caused liquefaction during past earthquakes.

Empirical liquefaction charts illustrating the field correlation approach under level ground conditions applied to different types of in situ measurements are given in Annex B. In this approach, the seismic shear stress τ_e, may be estimated from the simplified expression:

$$\tau_e = 0.65 \alpha S \sigma_{vo} \tag{4.3}$$

Where σ_{vo} is the total overburden pressure, S is the soil factor and α is the ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g. This expression may not be applied for depths larger than 20 m.

Table 4.3 Soil parameter values

Ground Type	S
A	1.00
B	1.20
C	1.15
D	1.35
E	1.40

If the field correlation approach is used, a soil shall be considered susceptible to liquefaction under level ground conditions whenever the earthquake-induced shear stress exceeds a certain fraction λ of the critical stress known to have caused liquefaction in previous earthquakes. The value ascribed to λ for use in a Country may be found in its National Annex. The recommended value is $\lambda = 0.8$, which implies a safety factor of 1.25.

4.3 Turkish Code for Structures to be Built in Disaster Areas

The objective of this Part of the Specification is to define the minimum requirements for the earthquake resistant design and construction of buildings and building-like of structures or their parts subjected to earthquake ground motion.

The general principle of earthquake resistant design to this Specification is to prevent structural and non-structural elements of buildings from any damage in low intensity earthquakes; to limit the damage in structural and non-structural elements to repairable levels in medium-intensity earthquakes, and to prevent the overall or partial collapse of buildings in high-intensity earthquakes in order to avoid the loss of life.

4.3.1 Determination of Soil Conditions

4.3.1.1 Soil Groups and Local Site Classes

Soil groups and local site classes to be considered as the bases of determination of local soil conditions are given in Table 4.4 and Table 4.5, respectively. Values of soil parameters in Table 4.4 are to be considered as standard values given for guidance only in determining the soil groups.

Soil investigations based on appropriate site and laboratory tests are mandatory to be conducted for below given buildings with related reports prepared and attached to design documents. Soil groups and local site classes to be defined in accordance with Table 4.4 and Table 4.5 shall be clearly indicated in reports.

Table 4.4 Soil Groups

Soil Group	Description of Soil Group	Stand. Penetr. (N/30)	Relative Density (%)	Unconf. Compres. Strength	Shear Wave Velocity
A	1. Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks. 2. Very dense sand, gravel... 3. Hard clay, silty lay…	- >50 >32	- 85-100 -	>1000 - >400	>1000 >700 >700
B	1. Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity... 2. Dense sand, gravel... 3. Very stiff clay, silty clay...	- 30-50 16-32	- 65-85 -	500-1000 - 200-400	700-1000 400-700 300-700
C	1. Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity 2. Medium dense sand and gravel.... 3. Stiff clay, silty clay...	- 10-30 8-16	- 35-65 -	<500 - 100-200	400-700 200-400 200-300
D	1. Soft, deep alluvial layers with high water table... 2. Loose sand... 3. Soft clay, silty clay...	- <10 <8	- <35 -	- - <100	<200 <200 <200

Regarding the buildings outside the scope of given above, in the first and second seismic zones, available local information or observation results shall be included or published references shall be quoted in the seismic analysis reports to identify the soil groups and local site classes in accordance with Table 4.4 and Table 4.5.

In the first and second seismic zones, horizontal bedding parameters as well as horizontal and vertical load carrying capacities of piles under seismic loads in Group (C) and (D) soils according to Table 4.4 shall be determined on the basis of soil investigations including in-situ and laboratory tests.

Table 4.5

Local Site Class	**Soil Group According to Table 4.4 and Topmost Layer Thickness**
Z1	Group (A) Soils Group (B) Soils with $h_1 \leq 15$ m
Z2	Group (B) Soils with $h_1 > 15$ m Group (C) Soils with $h_1 \leq 15$ m
Z3	Group (C) Soils with 15 m< $h_1 \leq 50$ m Group (D) Soils with $h_1 \leq 10$ m
Z4	Group (C) Soils with $h_1 > 50$ m Group (D) Soils with $h_1 > 10$ m

4.3.1.2 Investigation of Liquefaction Potential

In all seismic zones, Group (D) soils according to Table 4.4 with water table less than 10 m from the soil surface shall be investigated and the results shall be documented to identify whether the liquefaction potential exists, by using appropriate analytical methods based on in-situ and laboratory tests.

4.4 Japanese Design Specifications for Highway Bridges (Part V. Seismic Design)

4.4.1 Evaluation of Sandy Soils with Potential to Develop Soil Liquefaction

4.4.1.1 Sandy Soil Layers Requiring Liquefaction

In principle, a liquefaction assessment of an alluvial saturated sandy soil layer characterized by the three following conditions shall be performed as specified in 4.4.1.2, because during an earthquake, it might liquefy, effecting a bridge.

1. The ground water level is less than 10 m from the surface of the ground at the site, and there is a saturated soil layer at a depth less than 20 m from the surface of the ground at the site.
2. A soil layer with a fine-grained fraction FC of 35% or less, or a soil layer with a plasticity index I_p of less than 15, even if the fine-grained fraction is higher than 35%.
3. A soil layer with a mean grain diameter D_{50} less than 10 mm, and a 10% grain diameter D_{10} of 1 mm or less.

4.4.1.2 Liquefaction Assessment:

For soil layers that require a liquefaction assessment in accordance with the provision in 4.4.1.1, the resistance ratio F_L against liquefaction shall be calculated by equation (4.4), and it shall be assumed that any layer for which this value is 1.0 or less will liquefy.

$$F_L = R/L \tag{4.4}$$

$$R = c_w . R_L \tag{4.5}$$

$$L = r_d . k_{hc} . \frac{\sigma_v}{\sigma'_v} \tag{4.6}$$

$$r_d = 1.0 - 0.015x \tag{4.7}$$

$$\sigma_v = \{\gamma_{t1} . h_w + \gamma_{t2}(x - h_w)\}/10 \tag{4.8}$$

$$\sigma'_v = \{\gamma_{t1} . h_w + \gamma'_{t2}(x - h_w)\}/10 \tag{4.9}$$

(For Type I earthquake motion):

$$c_w = 1.0 \tag{4.10}$$

(For Type II earthquake motion):

$$c_w = \begin{cases} 1.0 & (RL \le 0.1) \\ 3.3RL + 0.67 & (0.1 < RL \le 0.4) \\ 2.0 & (0.4 < RL) \end{cases} \tag{4.11}$$

Where,

FC: Fine-grained fraction (%). (Transit weight percentage of the soil grains with a diameter less than 75 μm)

I_p: Plasticity index

D_{50}: Mean grain diameter (mm)

D_{10}: 10% grain diameter (mm)

F_L: Resistance ratio against liquefaction

R: Dynamic shear strength ratio

L: Shear stress ratio during an earthquake

r_w: Modification factor based on earthquake motion properties

R_L: Cyclic triaxial strength ratio, it shall be found as specified in Chapter 4.4.1.3

r_d: Reduction coefficient in the depth direction of the shear stress ratio during an earthquake

k_{hc}: Design lateral force coefficient used with the ductility design method

σ_v: Total overburden pressure (kgf/cm^2)

$\sigma v'$: Effective overburden pressure (kgf/cm^2)

x: Depth from the ground surface (m)

γ_{t1}: Unit weight (tf/m^3) of the soil shallower than the ground water level

γ_{t2}: Unit weigh (tf/m^3) of the soil deeper than the ground water level

γ'_{t2}: Effective unit weight (tf/m^3) of the soil deeper than the ground water level

h_w: Depth of the ground water level(m)

The following has been determined based on the results of research conducted since the Niagata Earthquake and is supplemented by analysis of cases resulting from the Hyogo-ken Nanbu Earthquake.

The stipulations in the provisions for soil layers requiring liquefaction assessment are based on the following grounds:

1. Almost all past cases of liquefaction during earthquake occurred in alluvial sandy layers. But because liquefaction has occurred in soil layers other than alluvial sandy layers during the Hyogo-ken Nanbu Earthquake and other recent earthquakes, the range of soil layers requiring liquefaction assessment has been reviewed as described below.
2. The depth of the soil layers was set as within 20 m of the ground surface in light of past experience and the degree of its effects on structures.
3. As the lower limit of the grain size of a soil layer that requires liquefaction assessment, the earlier Seismic Design Specifications (February 1990) stipulated a minimum mean grain size D_{50} of 0.02 mm, but in response to the results of recent research, the limit is now stipulated as stated in the provision. In past cases, most layers found to have liquefied had a fine-grained fraction FC of less than 35%, but because liquefaction has also occurred in soil layers with an FC value over 35% but a low plasticity index, low plasticity silly soil for example, the assessment standards are now as stipulated in the above provision. Consequently, if the FC is less thin 35%, liquid and plasticity limit testing need not be performed.
4. As the upper limit of the grain size of a soil layer that requires liquefaction assessment, the earlier Seismic Design Specifications (February 1990) stipulated a maximum mean grain diameter D_{50} of 2 mm, but because observations of the effects of recent earthquakes including the Hyogo-ken Nanbu Earthquake have revealed liquefaction of gravely soil with a mean diameter higher than 2 mm. the upper limit has been revised as stipulated in the above provision. But the grain diameter indicated here shall be a value obtained by means of grain size analysis of specimens obtained by means of standard penetration testing. Standard penetration test specimens have a finer grain size than the in-situ material as a consequence of the effect of crushing

of their grains. The extent of this difference is not necessarily a uniform relationship because of variations in the hardness or coarseness of the grains, but a moan grain diameter of 10 mm in a specimen obtained by standard penetration testing roughly corresponds to in-situ material with a mean grain diameter of about 20 mm or more. The 10% grain diameter D_{10}, was set at max. 1 mm to account for the tact that the permeability of coarse gravelly soil with a low uniformity coefficient is high, and such soil resists liquefaction. Sandy soil and gravelly soil shall be distinguished by determining if the mean grain diameter D_{50} is less than 2 mm or is greater than 2 mm.

5. There has not been confirmed case of liquefaction of diluvial soil caused by any past earthquake, including the Hyogo-ken Nanbu Earthquake. Because the N value of diluvial soil is generally high and, as a result of diagenesis, its resistance to liquefaction is also high and there is small probability of the liquefaction of diluvial soil. But because in some regions, there is diluvial soil with a low N value or that which has lost its diagenesis ability, such diluvial soil should be the object of liquefaction assessments.

It stipulates that liquefaction assessments shall be performed for Type I and Type II earthquake motion used for the ductility design method. Because the cyclic triaxial strength ratio R_L fluctuates widely according to the cyclic properties of earthquake motion, it shall be corrected by equations (4.10) and (4.11) depending on whether it is Type I or Type II earthquake motion.

4.4.1.3 Cyclic Triaxial Strength Ratio

Cyclic triaxial strength ratio R_L shall be calculated by equation (4.12).

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (Na < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6\times10^{-6}.(N_a - 14)^{4.5} & (14 \le N_a) \end{cases} \quad (4.12)$$

Where,

Sandy soil case:

$$N_a = c_1.N_1 + c_2 \quad (4.13)$$

$$N_1 = 1.7N/(\sigma'_v + 0.7) \quad (4.14)$$

$$c_1 = \begin{cases} 1 & (0\% \le FC < 10\%) \\ (FC+40)/50 & (10\% \le FC < 60\%) \\ FC/20-1 & (60\% \le FC) \end{cases} \tag{4.15}$$

$$c_2 = \begin{cases} 0 & (0\% \le FC < 10\%) \\ (FC-10)/18 & (10\% \le FC) \end{cases} \tag{4.16}$$

Gravelly soil case:

$$N_a = \{1 - 0.36\log_{10}(D_{50}/2)\}N_1 \tag{4.17}$$

Where,

R_L: Cyclic triaxial strength ratio.

N: N value obtained from standard penetration testing.

N_1: N value converted to correspond to effective overburden pressure of 1 kgf/cm^2.

N_a: Corrected N value to accounting for the effects of grain size.

c_1, c_2: Modification factor of the N value based on the fine-grained fraction.

FC: Fine-grained fraction (%). (Transit weight percentage of the soil grains with a diameter less than 75 μm).

D_{50}: Mean grain diameter (mm).

It is stipulated that the equation used to compute the cyclic triaxial strength ratio R_L as stipulated in the provision shall be found by distinguishing sandy soil from gravelly soil based cm the results of laboratory undrained cyclic (triaxial testing using frozen undisturbed specimens and on the results of analysis of cases including those observed after the Hyogo-ken Nanbu Earthquake.

In the earlier Seismic Design Specifications (February 1990), the cyclic triaxial strength ratio was evaluated by supplementing the strength ratio obtained from the N value with the correction term of the strength ratio obtained from the mean grain diameter D_{50} and the fine-grained traction FC respectively, but under this specification, the effects of the grain size of sandy soil shall be evaluated by correcting the N value based on the fine-grained fraction FC. This change was made for the following reasons:

1. Concerning the effects on the cyclic triaxial strength ratio of the grain size properties of soil, it has been conduced that in sandy soil that is relatively fine-grained, the effects of grain size may be evaluated based on its fine-grained fraction FC.
2. A method accounting for the effects of grain size as an increment of the JV value provides a relatively higher fine-grained fraction and permits more appropriate evaluation of the strength of soil with a high N value than the method accounting for the effects of the grain size as the increment of the cyclic triaxial strength ratio.

Penetration testing to measure the N value should be performed based on the free drop method which results in the loss of little energy at the moment of impact. And because with equation (4.16), the N value of gravelly soil is measured a little high under the effects of the existence of gravel, the N value that has been obtained shall be reduced in accordance with the mean grain diameter D_{50} to evaluate the cyclic triaxial strength ratio. But because little data of this kind has been accumulated for gravelly soil and because the correction method presented in equation (4.16) is not fully reliable, the assessment may be done in another manner.

It has been argued that the cyclic triaxial strength ratio of soil in reclaimed land is lower than the value obtained by equation (4.12), but because insufficient data is available and differences between its strength properties and those of alluvial soil have not been clarified, this specification has not established special provisions governing soil in reclaimed land. More survey and research work must be conducted in this area.

On river beds and at other locations where the water level is above the surface of the ground, the total overburden pressure and the effective overburden pressure shall be found treating the ground water level as the surface of the ground. This is stipulated because water, which does not transmit shear force, does not act as an external force against the ground during an earthquake and because the load of the water above the ground surface, does not contribute to an increase in the effective overburden pressure.

But when considered particularly necessary, the most up-to-date detailed ground exploration and testing at the site, laboratory soil properties testing, and response analysis of the ground may be performed to assess liquefaction potential with reference to existing data.

5. ANALYSES

Below, it can be find a typical evaluation of the potential for liquefaction to occur by comparing equivalent measures of earthquake loading and liquefaction resistance of the SPT Log A-2, investigated at Adapazarı region according to the codes mentioned above. The most common approach to characterization of earthquake loading is through the use of cyclic shear stresses. The potential for liquefaction evaluated, by obtaining the data for SPT logs from the web address http://peer.berkeley.edu/turkey/adapazari/phase1/index.html at first. Then, degree of saturation (S), dry unit weight (γ_{dry}) and void ratio (e) values are assigned for the soil profiles. Bulk unit weight (γ_{bulk}) and (γ_n) values are calculated. After that, σ_{vo} and σ_{vo}' values are calculated respectively. Then, the necessary SPT (N) corrections are made according to the related code. The liquefaction potential is evaluated by comparing the earthquake loading (CSR) with the liquefaction resistance (CRR); this is usually expressed as a factor of safety against liquefaction, FS = CRR / CSR. A factor of safety greater than the values stated in the codes indicates that the liquefaction resistance exceeds the earthquake loading, and therefore that liquefaction would not be expected.

In Appendix A, it can be find the evaluation of the potential for liquefaction of 30 field logs investigated at that region, according to the 4 code mentioned above. Appendix B shows the maps and locations of the investigated field logs.

6. CONCLUSION

An attempt has been made in this thesis to point out the importance of the local soil conditions on the ground motion characteristics during earthquakes. It is evident that the degree of structural damages is directly related to the site properties. Therefore, careful consideration should be given to evaluate the significance of this phenomenon. However, even though it appears possible to establish some guidelines and to analyze the effects of various factors, such as bedrock depth, soil types, water table elevation and etc., the result obtained by analytical methods may not yield realistic results due to approximations made in defining the soil stratifications, due to simplifications made in defining soil properties and in modeling soil behavior, due to the assumptions made in defining the assumed earthquake motion in the underlying rock formation, and finally due to the mathematical model selected. This aspect of the deterministic approaches of various forms that may be adopted to evaluate the site condition necessitates the use of sound engineering experience and judgment to achieve realistic results. The numerical analysis performed would yield useful information that should be utilized to supplement a broader study in evaluating the site effects.

It can be seen from the results that with the greatest factor of safety, Californian code is the safest one. However; I believe that, Turkey, which is on the road of joining the EU, should carry out the European regulations for constructions, and especially for the structures that will be built in the disaster areas, the engineers should be loyal to Eurocode 8 (prEN 1998-1:2003 & prEN 1998-5:2003).

During the comparative liquefaction analysis according to Turkish Specification for Structures to be Built in Disaster Areas, Eurocode 8, Japanese and Californian Seismic Codes, it is concluded that, a careful consideration should be given while it has been analyzing and designing of the structures in that region; complementary documents should be added to the missing parts to liquefaction part of the Turkish Specification for Structures to be Built in Disaster Areas and a translation of Eurocode 8 should be made and should be in force in Turkey.