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March, 1993
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U.S. Department of Commerce
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National Institute of Standards and Technology
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CONTENTS

CHAPTER 1 INTRODUCTION

1.1 Background

1.1.1 Earthquakes and Liquefaction

Over the years, some of the most spectacular, and costly damage caused by earthquakes has been due to liquefaction of sands. A very rudimentary description of liquefaction can be given quite simply. When a suitably intense earthquake shakes a loose, saturated sand, the grain structure tends to consolidate into a more compact packing. Since all these movements are happening very rapidly, there is no chance for the volume to reduce through pore water dissipation. In an idealized sense, the incompressible pore fluid then takes up the applied stress, the effective stress approaches zero, and the deposit "liquefies." Since a liquid has no shear strength, disastrous consequences may occur. In actuality the liquefied soil has a residual strength, and the material exhibits complex stress-strain behavior.

The disastrous consequences of liquefaction was brought to the fore in 1964 by the Niigata and Alaska earthquakes of that year In Niigata, Japan, large buildings slowly rolled over on their sides, and pipes and tanks floated to the surface through the temporarily fluidized soil in which they were buried. Liquefaction also triggers earth slides and large displacements of earthen dams. A terrible disaster was narrowly avoided when the San Fernando dam suffered very large displacements due to the 1971 San Fernando earthquake, as shown in Fig. 1.1. Similar damage has occurred over the years in locations as diverse as China, Nicaragua, Japan, Charleston, SC, San Francisco, the Imperial Valley, CA, and Idaho.

The effects of liquefaction-caused damage to lifelines are especially costly. Damage to roads, rail, telecommunications, power, and pipelines of all types is always harmful, but is especially so during time of emergency. One of the most striking examples of the effect of lifeline damage on public safety is the occurrences during the 1906 San Francisco earthquake. After that earthquake, over 490 city blocks were totally destroyed by a fire, the largest, most deadly fire in U.S. history (O'Rourke et al., 1991). Little could be done to stop the spread of the fire since the pipelines carrying water were broken due to liquefaction-induced ground displacements. It was estimated that 56 percent of the municipal water supply was completely cut off. It should be noted that the same areas that suffered liquefaction-induced ground displacements in 1906 suffered similar displacements due to the 1989 Loma Prieta earthquake (O'Rourke et al., 1991).

It is hoped that this brief introduction gives the reader some sense of urgency as to the importance of being able to predict where liquefaction might occur, and how much ground displacement will be associated with this possible liquefaction. Further impetus can be gotten from perusing detailed reviews of historical damage due to this phenomenon given in EERI (1986) and Steinbrugge (1982). Descriptions of ground behavior during many past earthquakes can be found in Ambraseys and Sarma (1969). A detailed study on the possible disruption to U.S. society due to earthquake damage to lifelines is found in FEMA (1991).

Fig. 1.1 An aerial photograph of the Lower San Fernando dam after the 1984 San Fernando earthquake.

1.1.2 Purpose

This report was written to evaluate and interpret the current state-of-the-art of in situ methods of soil property measurement, which allow accurate prediction of liquefaction potential, and the possible displacements if liquefaction did occur. These two problems fit organically for the design engineer, or planner. If the liquefaction potential can be accurately and effectively estimated, the planner can avoid routing a pipeline or road through these deposits. If these areas can not be avoided, then the designer needs to know how much displacements the structure needs to withstand. If a cost-effective estimate of possible ground displacements can be made, and the designer finds the effects controllable, the planner might save money and time by designing for the expected displacements.

The report summarizes and evaluates significant technical papers in two areas, (1) in situ methods of estimating liquefaction potential, and (2) methods for estimating ground displacements due to liquefaction. Both topics are of direct import to the behavior of lifelines. This report makes no attempt to identify and enumerate every paper or technical publication written on these subjects. It serves, rather, as a thorough overview and evaluation of where the profession is today. The entire problem of liquefaction is extremely complex, and affected by many parameters, a lot of which can not even be measured. Therefore, all methods of estimation will have "problems," and even-handed discussions will point out many "negatives."

The report assumes some degree of technical sophistication by the reader, although an attempt is made to explain complicated or unfamiliar material. The liberal use of references allows the reader to find an understandable source of explanation for most topics discussed. The report takes a "critical," interpretive point of view when examining proposed methods. It is believed that this in-depth, elucidative approach differentiates this review of the state of liquefaction hazard procedures from previous such reports. It is in no way a recommendation to practicing engineers as to proper design method. However, all engineers can gain insight and sharpen their understanding of the subject by discussing the assumptions, assets, and liabilities of published methods, regardless of who the original authors are. It is believed that any interpretive review will involve the author's opinions. More importantly, there is no intention here of diminishing the readers opinions or views

1.1.3 A Note on Nomenclature

Over the years, the term "liquefaction" has come to mean many things to many people. The most common use of the word is to generally describe the situation where the pore water pressure builds up, approaching the in situ vertical effective stress during cyclic loading. Strictly speaking, this condition results in very different behaviors. If the sand is dense and acts in a dilative manner when strained, the soil can deform under the action of the cyclic loading, but does so in a controlled manner, with often small displacements, and without risk of catastrophic failure. Castro refers to this as cyclic mobility (e.g. 1987, 1977).

The second type of behavior occurs for loose sands which behave in a contractive manner when loaded. This behavior is what was described in the first paragraph of this report, and is what Castro calls "liquefaction." This liquefaction describes the undrained behavior of a contractive sand subject to cyclic excitation, resulting in fluidization and possible catastrophic failure.

Whitman (1985) acknowledges these three meanings for the term "liquefaction". His solution is to coin an artificial term for the general, all-encompassing definition -- "liquefailure". However, this term sounds bad and is not in general use. Adoption of this term in this report would be confusing. Instead, this writer hopes the usage will be in the proper context so that the reader is aware whether Liquefaction refers to the general behavior due to pore pressure buildup, or to Castro's mechanism. The behavior of dilatant sands will always be referred to as cyclic mobility.

1.2 Scope

The emphasis throughout this report is on in situ techniques for estimating a soil's behavior. This is because the author believes that the term "undisturbed sample" is an oxymoron when dealing with sands, especially very loose sands. The two topics of interest are covered in two chapters.

Chapter 2 discusses in situ methods of estimating the propensity of a soil to liquefy under earthquake loading. The methods covered include penetration tests such as the standard penetration test and cone penetration test, intrusive tests such as the cross-hole test and nuclear density gage, and non-intrusive tests like the spectral analysis of surface waves technique. Methods for interpreting the results of these tests are examined in detail.

Chapter 3 examines the various approaches to estimating earthquake-induced ground deformation. These methods include empirical relations as well as constitutive modeling.

Chapter 4 presents a summary of the report, and some brief recommendations by the author. A thorough and extensive bibliography follows. A disk with the abstracts of all cited papers in Pro-Cite format is available from the author.

Finally, it should be mentioned that the most thorough and complete work on the subject of liquefaction is the report written by the Committee on earthquake Engineering of the National Research Council, 1985.

CHAPTER 2 ESTIMATION OF LIQUEFACTION POTENTIAL BY IN SITU METHODS

2.1 A Framework of Understanding

2.1.1 Introduction

By definition, liquefiable soils are loose, particulate materials, almost always a sand or silty sand. For these materials it is very difficult to obtain a representative sample for laboratory tests, much less an undisturbed one. Peck (1979) states that it is "manifestly impossible" to obtain a completely undisturbed sample. Since the soil fabric, age of the soil deposit, amount of overconsolidation, cementation, and strain history all affect the way a soil deposit responds to earthquake excitation, the importance of carrying out tests on undisturbed material is obvious. Since most important characteristics of a soil are destroyed to an unknown degree by sampling, laboratory tests generally result in overly conservative design parameters (Ishihara, 1985; Peck, 1979). Therefore, in situ testing of soil deposits to estimate liquefaction potential becomes the method of choice.

At the present time there is no way to directly measure "liquefaction potential." What can be measured (directly or indirectly) are the various parameters that control a soil's propensity to liquefy. Before examining the various approaches to in situ testing, and the theoretical background of each, it is important to understand the various parameters that affect the potential for a soil to liquefy under cyclic loading. An early laboratory study of the effect of various parameters on the sand's shear modulus was done by Hardin and Drnevich (1972). The results are summarized in the Table 1. For example, one view of the liquefaction problem holds that liquefaction potential is a direct function of the shear stiffness of the soil. An in situ test must be able to "measure" the state of those parameters that control stiffness, or measure soil stiffness directly.

These same parameters also affect the results of the Standard Penetration Test, the most commonly run test in soil exploration (Seed, 1979). Results from other common tests methods used to estimate liquefaction potential such as the Cone Penetration Test, are generally controlled by these same parameters (Olsen, 1984). While empirical correlation is an extremely powerful approach, and often the only approach available to the practicing engineer, a physical relation between what the test measures and the mechanism of liquefaction must always be kept in mind.

The types of in situ tests currently being used for estimation of the liquefaction resistance of a soil profile can be broken into two very different classes: (1) intrusive tests and, (2) non-intrusive tests. Intrusive tests require that a device must somehow enter into the soil being tested. Non-intrusive tests can be run from the ground surface and require no direct mechanical interaction with the soil at depth. Intrusive tests are conceptually more straight-forward, and historically have been the most used.

Table 2.1 Parameters affecting the shear modulus of sands (Hardin and Drnevich, 1972).

Strain Amplitude	Very Important
Effective Mean Principle Stress	Very Important
Void Ratio	Very Important
Number of Cycles of Loading	Less Important
Octahedral Strength Envelope	Less Important
Effective Strength Envelope	Less Important
Overconsolidation Ratio	In-itself Relatively Unimportant
Frequency of Loading	In-itself Relatively Unimportant
Time Effects	In-itself Relatively Unimportant
Grain Characteristics	In-itself Relatively Unimportant
Soil Structure	In-itself Relatively Unimportant

NOTE - In-itself Relatively Unimportant means relatively unimportant in- and of- itself. However, these parameters, such as soil structure or grain characteristics, may affect another very important parameter greatly, affecting the very important void ratio parameter.

The various types of intrusive tests can be broken into two subgroups. The most common type of tests are the penetration tests, where the measured parameter is derived from the direct interaction of the device and the soil being deformed. This class includes tests such as the Standard Penetration Test (SPT), quasi-static Cone Penetration Test (CPT), the Flat Plate Dilatometer Test (DMT), and other variants of these methods. All these methods directly act on the soil, displacing in situ material and measuring the force needed to do so. In that sense these methods are attractive, since they are uncomplicated and there is always direct contact with the soil.

The other subgroup of intrusive tests are the tests carried out in a borehole. The borehole intrudes into the stratigraphy but the test methods measure how the soil alters a signal passed through the soil. The most common tests of this type are the seismic tests: the downhole and crosshole tests. Two newer, interesting, methods are the Suspension Logger and the Nuclear Density Gage. The non-penetrating tests are attractive since the measuring process itself does not affect the behavior of the soils.

The non-intrusive tests are a small, relatively new group of geophysical methods. These include the Spectral Analysis of Surface Waves (SASW), Seismic Refraction, and Back Calculation, sometimes referred to as System Identification (SI). Non-intrusive tests have the very great advantage of being cheap and easy to run, although the connection between the measured parameters and the gross behavior of the soils at depth is more esoteric than for the other currently available methods. The cost and labor of drilling bore holes or pushing a device deep into the ground are completely avoided, making large areal surveys economically feasible. Most importantly, the non-intrusive tests do not affect the in situ materials and therefore do not change the behavior of the soils during the measuring process.

With this brief conceptual framework in place, the various in situ methods for estimating liquefaction of sands will be examined in detail.

2.2 Penetration-Type Tests

2.2.1 Standard Penetration Test (SPT)

The SPT is by far the most common in situ test used in the geotechnical field. The mechanism for this test, shown in Fig. 2.1, is a split barrel (spoon) which is screwed onto a drill rod, and driven into the soil. A 63.5 kg hammer falling 0.76 m is the "standard" impact that drives the spoon into the soil. The device is usually lowered into an open borehole, or through a hollow stem auger. The hammer is then repeatedly raised and dropped onto an anvil connected to the top of the drill rods. The spoon is seated into the soil by driving it 150 mm, and then the number of blows needed to drive the device an additional 300 mm is counted. This value is the "blow count" or N-value (de Mello, 1971; ASTM D 2573).

The spoon is designed to capture a sampling of the soil it is penetrating through, and is split so that the specimen can be easily removed after the spoon is retrieved. However, due to the large aspect ratio of the tip of the spoon, and the large dynamic forces involved in advancement, the obtained sample is very disturbed. The sample is useful for obtaining the grain size distribution, Atterberg limits for cohesive soils, and cursory soil description (Peck et al., 1974). All direct information about the shear strength, soil fabric, void ratio (e), cementation, and relative density (D_r) are lost from the sample.

There are many variables that affect the N-values reported from different locations in different parts of the world. Since the goal is to have a "standard" penetration test, the raw N-values must be normalized as best as possible. Because the blow count is a function of the effective confining stress, and the confining stress is a function of depth, the N-value is normalized by the factor C_n to an effective overburden pressure of 98 kPa (1 TSF) (Peck at al., 1974; Gibbs and Holtz, 1957; etc.). Seed (et al., 1985, 1979) developed a chart for determining the normalization factor C_n, which however, is flawed since it necessitates deciding on a soil density a priori (Farrar, 1990). Procedure estimates a D_r from the corrected blow count.

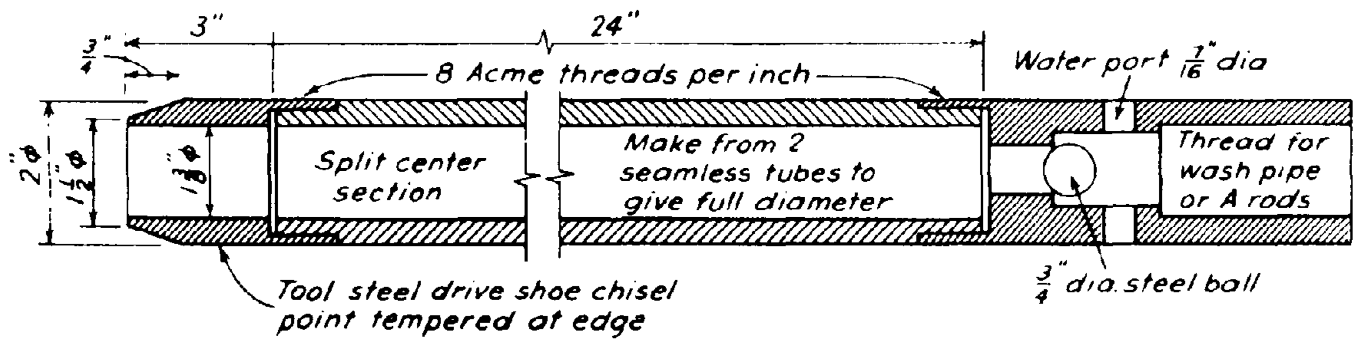

FIG. 2.1 Split barrel sampler for the Standard Penetration Test (from Peck et al. 1974).

The method of running the SPT is different in various countries and areas. Work by the National Bureau of Standards (Kovacs and Salomone, 1984) indicates that for most tests in the U.S., sixty percent of the theoretical energy is transmitted to the sampler. In Japan the average energy transmitted is seventy eight percent. In order to utilize data from all over the world in correlations, the recommendation is to normalize all values to a utilized energy of sixty percent. A table for this normalization is given in Seed et al. (1985). Further reference to normalizing international SPT data in connection to liquefaction properties of the soil can be found in Chung et al. (1986), which has a very complete bibliography.

The corrected N-value is labeled $(N_1)_{60}$, and is the parameter commonly used in correlations of liquefaction potential (Seed et al., 1985). Throughout the rest of this report, this is what will be referred to by the "N-value" unless otherwise explicitly stated. There are a very wide array of corrections proposed to account for the many other variables in the SPT, such as short lengths of drill rod, differences in sampling tube design, diameter of bore hole, frequency of hammer drop, etc., many of which are "purely speculative" (Farrar, 1990). The SPT can be seen as a simple, rough and ready empirical test. Applying an excessive number of corrections, many of which must be guessed at, defeats the purpose of the test and makes the results more uncertain rather than less.

The parameter measured by the SPT — the blow count — is not an innate material property such as the elastic constants, mineral composition, grain size, water content, void ratio, or even strength. The N-value can give an index of actual soil properties when careful correlations are made (Holtz and Kovacs, 1981). Although the N-value has been correlated with many soil properties, such as shear strength (Peck at al., 1974), the one soil property often associated with liquefaction susceptibility is relative density (D_r). Over the years correlations have been developed that relate N-values to D_r (e.g. Tokimatsu and Seed, 1987; Skempton, 1986; Seed, 1979; Marcuson and Bieganousky, 1977). However, it is doubtful that one unique relation can be found between pounding a pipe into the ground and a particular packing of soil grains, for all different soil varieties and depositional histories. The best that can be expected is a calibrated value for each given soil. Marcuson and Bieganousky (1977) state after completing a rigorous in situ test suite:

> It is concluded that the SPT is fairly repeatable in homogenous deposits; however, variations in density, structure, or lateral stress within the test medium will produce widely scattered N-values. Thus, estimates of in situ relative densities from N-values should be considered gross values or trends and should not be interpreted as accurate determinations for any specific case.

With this background, the various important correlations between N-values and liquefaction potential will be examined in detail.

2.2.2 Seed's Method

The current state of practice is the method pioneered by H. Bolton Seed at the University of California, Berkeley. This method has been updated in many reports and professional papers over the years (Tokimatsu and Seed, 1987; Seed et al., 1985; Seed et al., 1983; Seed and Idriss, 1981; Seed, 1979; Seed and Idriss, 1970). While the early presentations used a correlation between N-values and D_r as an intermediary step between blow count and liquefaction potential, the current approach bypasses this step and directly correlates N-values to liquefaction potential.

Seed's method is based on the shear stress a soil layer is subjected to by an earthquake. The maximum cyclic shear stress a sand strata at depth Z is subjected to during a given earthquake is estimated by the approximate equation (Seed and Idriss, 1971):

$$\tau_{max} = \frac{a_{max}}{g}(\gamma_t Z) \cdot r_d \qquad (2.1)$$

where τ_{max} = maximum shear stress
a_{max} = maximum horizontal acceleration at the ground surface
g = acceleration of gravity
γ_t = total unit weight of soil
Z = depth from the surface
r_d = stress reduction factor (1 at surface, decreasing to 0.9 at 10.5 m depth).

The deposit in the field is said to undergo an average stress τ_{avg} which is 0.65 of τ_{max}. The average shear stress is then normalized by the vertical effective stress $\bar{\sigma}$ to give the cyclic stress ratio :

$$\frac{\tau_{avg}}{\bar{\sigma}_v} = 0.65\,\frac{a_{max}}{g}\left(\frac{\sigma_v}{\bar{\sigma}_v}\right)_d r_d \qquad (2.2)$$

where σ_v = total vertical stress
$\bar{\sigma}_v$ = total vertical effective stress (σ_v-u)
u = pore water pressure.

The cyclic stress ratio serves as the link between the peak acceleration of an earthquake and the dynamic shear stress applied to soil at depth.

As stated previously, the SPT is the most commonly run test for field investigation, and there is a very large history of N-values from many sites around the world. Seed and his associates assembled a data bank of more than one hundred records of SPT tests taken at sites that liquefied, and sites that could have liquefied and did not. Most of these records were for sites excited by earthquakes with Magnitude 7-1/2. All the blow counts were corrected for all known variables to yield $(N_1)_{60}$ so that, in theory, all the SPTs were comparable. A plot, shown in Fig. 2.2, was then made of cyclic stress ratio vs. $(N_1)_{60}$, and a boundary drawn between the values for liquefied sites and non-liquefied sites.

As seen in Fig. 2.2, there is an obvious dividing line between the domain of where evidence of liquefaction was present, and where it was not, for Magnitude 7-1/2 earthquakes. A designer merely corrects the field blow counts to $(N_1)_{60}$, calculates the cyclic stress ratio for the design earthquake, and checks Fig. 2.2 to see if the sand should liquefy. Correction factors were given to extend this method to cover different magnitude temblors, and for silty sands. Based on the idea that the functional difference between magnitudes is only the number of stress cycles, a table of factors are used to modify the cyclic stress ratio for quakes other than Magnitude 7.5 (Seed et al., 1983).

A different interpretation of Seed's method was taken by Haldar and Tang (1979). They developed a probabilistic procedure for estimating the risk associated with the prediction of liquefaction, rather than a simple yes-no answer. The paper was written in regards to the early deterministic procedure of Seed and Idriss (1970), but the principles involved are still valid for the later versions of Seed's procedure. The investigations showed that the uncertainties of the seismic parameters exceed the uncertainties about the soil parameters leading to the measure of liquefaction resistance. The uncertainties about the soil's resistance to liquefaction will be governed by uncertainties about relative density and cyclic shear strength. In addition, the uncertainties about secondary parameters, or those not fully understood, are significant and should be further investigated.

The probabilistic approach removes many of the theoretical objections to Seed's method, and gives answers that are very useful to an urban planner. However, engineers are not used to working with probabilities rather than set numerical values, and the use of a probabilistic scheme requires values for uncertainties to be estimated that are not easily deduced. While acceptance of this type of approach is to be worked towards, its implementation will not happen soon.

2.2.3 Chinese and Japanese Methods

The test criteria set by the 1974 Chinese Building Code is shown in Fig. 2.2. These limits match that of Seed and his coworkers for the range of common accelerations. In addition, other workers in Japan (Iai et al., 1987; Kokusho et al., 1983; Tokimatsu and Yoshimi, 1983) have derived virtually identical curves once the blow counts are properly normalized (Ishihara, 1985).

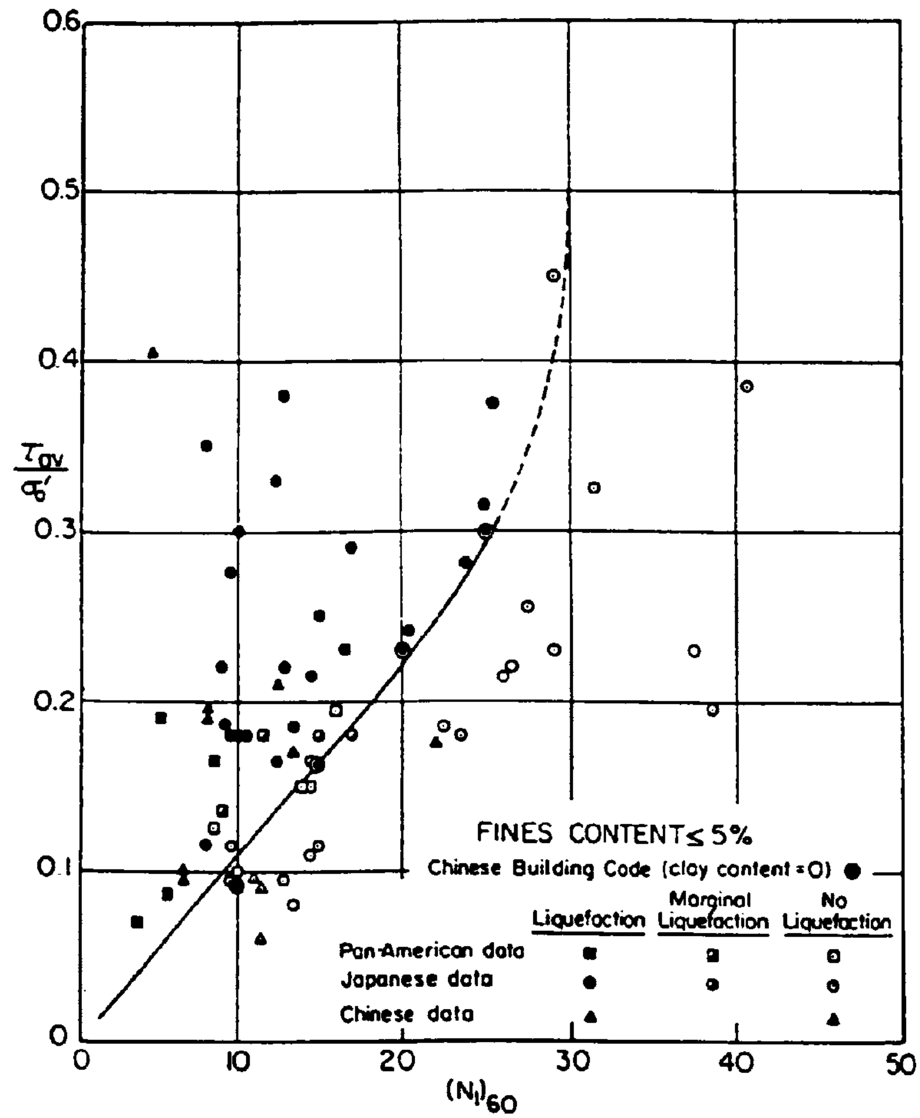

Fig. 2.2 Relationship between cyclic stress ratios causing liquefaction and $(N_1)_{60}$ values for clean sands for magnitude 7.5 earthquakes (from NRC, 1985).

It should be noted that all the various correlations utilizing the SPT are all compared to Seed's results. As pointed out by Peck (1979), the concept of "case studies" takes on an entirely new meaning in the field of liquefaction prediction. Since earthquakes can not be made to order, studies end up using several methods to give an estimation and then comparing them to the result Seed's method gives. The studies are after-the-fact analyses rather than predictive.

Rather than proving that any method is better than the others in predicting a sand's behavior during an earthquake, the comparisons show an agreement in the correlation method. Seed's method is the most direct, uses the largest database, and has a direct appeal to common sense. It is the de facto standard and it seems excessive to devote energy to further refining gross correlations in the hope of "becoming more accurate." The rational behind the Japanese adoption of their own correlations comes from wanting a more detailed relationship for their specific geologies and testing technique.

2.2.4 Ambraseys' Method

Ambraseys (1988), in the paper given for the first Mallet-Milne lecture, takes the same data Seed used and develops a more rational cyclic stress ratio which takes into account seismic parameters. Ambraseys starts by defining the seismic conditions that induce a sand deposit to liquefy. The data base used consists of 137 entries, all data being corrected for all known variables. Field observations show that for a given earthquake moment magnitude $\mathbf{M_w}$, observed evidence of liquefaction are all within a distance $\mathbf{R_f}$. The data gives a relationship such that for distance to fault $\mathbf{R > R_f}$ liquefaction is unlikely to occur for any site, and for $\mathbf{R < R_f}$ it is possible for liquefaction to occur but depends on soil shear strength. This boundary in moment magnitude-fault distance ($\mathbf{R_f}$) space is given by Eq. 2.3 :

$$M_w = (9.2x10^{-8})R_f + 0.90\log(R_f) + 0.18 \tag{2.3}$$

where M_w = moment magnitude of an earthquake
R_f = limit epicentral distance beyond which liquefaction does not occur.

Accepting the Joyner-Boore (1981) attenuation law allows Ambraseys to define a critical ground acceleration, $\mathbf{k_c}$, to be defined. This value is the ground acceleration associated with the fault distance (Eq. 2.3), as a function of quake magnitude, for which sands will liquefy. This relationship shows that the fault distance within which liquefaction may occur increases with increasing $\mathbf{M_w}$ while the necessary acceleration for that distance decreases. This reflects the effect of longer shaking, associated with larger events, on liquefaction potential.

The intent now changes to deriving relationships between an applied stress ratio causing liquefaction, and SPT blow count for different magnitude earthquakes, avoiding Seed's arbitrary scaling factor. Ambraseys believes that reducing different magnitude ground motions to equivalent uniform cycles destroys any consideration of the nature of the actual earthquake ground motions.

> It is not considered reasonable to allow the level of shear stress for a given magnitude earthquake to vary in a deposit only with a fixed number of equivalent cycles, without also including some consideration of the distance of the site from the seismic source, allowing for attenuation. Are the effects of an earthquake of Magnitude 7 the same next to the causative fault as they are 50 km away? Certainly, the introduction into a body of field data of the concept of equivalent uniform cycles of loading, or of a scaling factor as proposed by Seed and Idriss (1982) for instance, unduly complicates any attempt to quantify the observed behavior of deposits under seismic loading.

Ambraseys tests the hypothesis of the existence of a transition zone between the cyclic stress ratios associated with liquefaction which is a function of M_w and $(N_1)_{60}$. Assumptions are made that the duration and intensity of excitation at critical depth is related to M_w, and $(N_1)_{60}$ is related to the liquefaction resistance of the deposit. A defining boundary is given by :

$$\frac{\tau_{avg}}{\bar{\sigma}_v} = 3.29e^{(0.06(N_1)_{60})}\left[(N_1)_{60}\right]^{0.755}e^{(-0.81M_w)} \tag{2.4}$$

where τ_{avg} = average shear stress
$\bar{\sigma}_v$ = total vertical effective stress

with a boundary transition zone defined by a $dM_w = \pm 0.25$. For M_w 7.5 the Chinese, Seed, and Ambraseys curves match quite well. However, for different magnitudes there is a significant difference between the Ambraseys curves which rationally take account for temblor magnitude, and the Seed curves which are scaled by an "arbitrary" factor. A summary of the differences in the magnitude effect for the two methods is given in Table 2.2.

Table 2.2 Comparison of the effect of event magnitude between Seed and Ambraseys.

	Earthquake Magnitude Factor	
Magnitude	(Seed et al.,1983)	(Ambraseys, 1985)
8-1/2	0.89	0.44
7-1/2	1.00	1.00
6-1/2	1.19	1.69
5-1/2	1.43	2.86

Finally, Eq. 2.3 can be combined with the Eq. 2.4 to take into account the effect of source distance. The two limiting cases will be for the water table at the ground surface, and at the critical depth. The range of the cyclic stress ratio varies from 1.3 $\mathbf{k_c}$ to 0.62 $\mathbf{k_c}$. Figure 2.3 shows Ambraseys' final design chart which allows the assessment of critical acceleration from the corrected N-value for a clean sand located $\mathbf{R_f}$ from an earthquake with magnitude $\mathbf{M_w}$. Curves for the two limiting water table cases are given. It is of interest to note that critical $(N_1)_{60}$ values predicted from Eqn. 2.3 for liquefied sites at critical distances are not sensitive to the magnitude of acceleration, varying from eight to four for clean sands and well below four for sands with fifteen percent fines.

2.2.5 Analysis of SPT Methods

Seed's method, the simplest and most straightforward, is the de facto standard for estimating liquefaction potential. Virtually every report of a different method "proves" itself by comparison to Seed. As pointed out by Peck (1979), this is an interesting twist on the idea of proof. In fact, no method is actually proven as a predictive tool since all have been tested after the fact. The soil density has changed after being subjected to an earthquake, and there are large errors associated with estimations of site-specific acceleration. This is especially important for liquefied sites since the soil loses stiffness, and the amplitude and frequency of movement of the soil decreases as a function of soil strength, duration of shaking and length of shaking (e.g. Holzer et al., 1989).

Nevertheless, the standard method seems insensitive to these and other variables, and when investigations are made at sites that liquefy, test results show that it was to be expected from the blow count. A possible explanation for this is based on how "liquefaction" is defined for the chart. The case histories that go into the data bank for sands that liquefy are obvious cases. For there to be large surface manifestations, there must be large displacements of pore water. Dobry (1989) maintains that the SPT charts correctly predict the absence or presence of liquefaction manifestations rather than the actual buildup of pore water pressure to match the effective stress (the technical definition). Cases of cyclic mobility, slight liquefaction, or impending liquefaction will not show up in the data base. The result is a very robust set of data from which a strong correlation can be made.

The standard SPT-based method of estimating liquefaction susceptibility (as discussed above) is becoming self-fulfilling, since often N-values themselves are used to define whether a site liquefied or not. An example is found in the review report written by Farrar (1990, p. 12), "Analysis of SPT data using the $(N_1)_{60}$ method confirms the occurrence of liquefaction at site A."

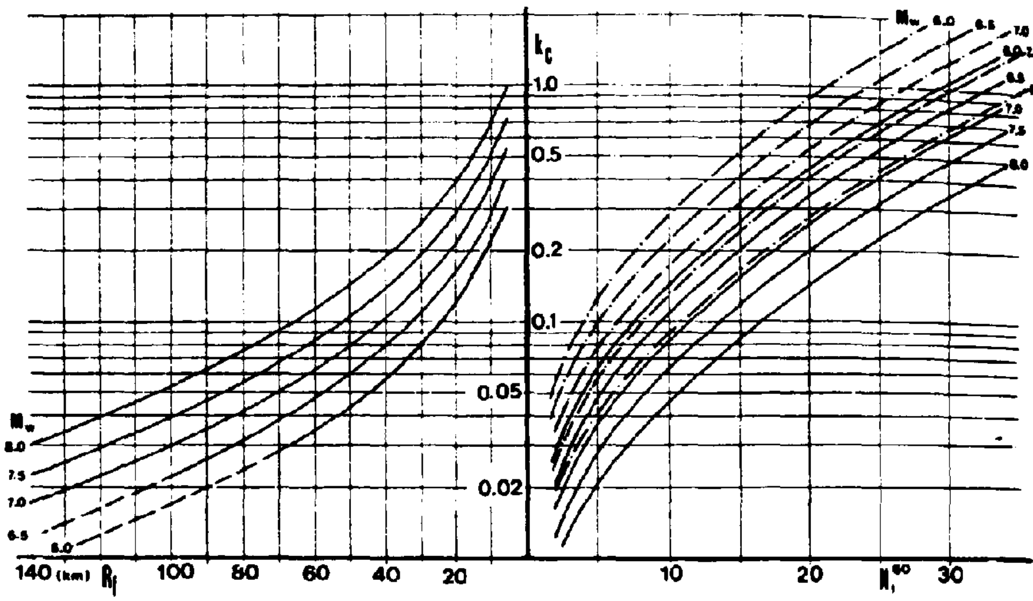

Fig. 2.3 Graphs to allow the assessment of critical acceleration (k_c) and corresponding $(N_1)_{60}$ at a clean sand site a distance R_f from the temblor of moment magnitude M_w. Water table at ground surface — solid lines; water table at critical depth — dashed lines (from Ambraseys, 1988).

There are significant technical problems associated with the SPT test. The results are very dependant on details of the field method used. Variables such as how many turns of rope are on the cathead, diameter of the cathead, how the rope is thrown, all affect the amount of energy input into the spoon. Based on the author's experience as a driller, there are other, unmentioned, variables such as how tight the drill rod is connected, how clean are the threads on the sections of drill stem, how straight is the drill string, does the hammer hit the upper stop on the up-stroke, condition of the spoon, cheating by the crew near quitting time, etc., that have a significant effect on blow count. Liquefiable sands often correlate to extremely low N-values, yet it is difficult to understand how the SPT can accurately discriminate between low blow count values much less than ten.

The SPT yields a value that is averaged over the 300 mm the spoon is driven. The spoon can penetrate 50 mm on the first blow and then take twenty more blows to go the next two hundred. In this case the thin, loose, layer over the stiff base would not be found. This insensitivity to thin layers has been reported in the literature and is one of the principle reasons to replace the SPT with the CPT (to be discussed in the next section) (Castro, 1991; Seed and Idriss, 1981).

Liquefaction is a relatively near-surface phenomenon in flat terrain, with no visible effects if it occurs under more than approximately five meters of cover (Ishihara, 1985). The SPT is not very accurate at near-surface depths (Farrar, 1990; Robertson and Campanella, 1985), and Seed et al. (1985) recommends a correction factor for short lengths of drill stem. Blow count is always corrected to be equivalent to a confining pressure of 1 kgf/cm^2, corresponding to a depth of over five meters. However, lifelines in particular interact with a shallow depth of soil.

While the methods of evaluating liquefaction potential using the SPT are valuable design tools. and in some sense our only design tool, the methods are a dead-end intellectually. The N-value does not measure any innate property of the soil It might give an empirical correlation for some property, but "blow count" appears in no physical theory. For geotechnology to move ahead in more analytical directions, effort should be made in areas that can give measures of actual physical properties. Data should continue to be added to the substantial data bank, but future research dollars are better spent devising in situ tests that measure properties that can be shown to be actual parameters controlling the potential for liquefaction.

2.2.6 "Improved" Standard Penetration Tests

Over the years there has been interest in making SPT results more repeatable and less a function of the test operator. Many of the variables affecting repeatability and accuracy have been discussed above. The single most important variable is the amount of energy that actually is transferred from the falling hammer into the drill string. The generally agreed upon method has been to use a rough factor to normalize blow count to the sixty percent energy transfer common in the United States. There is an ASTM standard designed to standardize the test procedure (ASTM D1586). However, studies at NBS (Kovacs and Salomone, 1982) and elsewhere (Schmertmann et al., 1978) still found very large variability.

In response to this variability, attempts were made to actually measure the energy input into the system (Kovacs, 1979; Schmertmann and Palacios, 1979). A commercial unit was even marketed (Hall, 1982). These attempts gave erratic results, with energies sometimes calculated to be greater than what was physically possible to input. It is believed by some (Sy and Campanella, 1991; Kovacs, 1984) that this problem was due to the force-time integration method used by these analyses. Errors were made in identifying the duration of the pulse, so the result of the integration over time was too large. This approach is the current ASTM method of measuring actual SPT energy, based on Schmertmann's work (ASTM D4633-86).

A proposed improvement is the force-velocity integration method (Sy and Campanella, 1981) which uses a more fundamental relationship to determine energy input. The use of an additional measured parameter, velocity, allows some significant a priori assumptions of the standard method to be dropped. The authors point out that the technique is the same used in dynamic pile driving monitoring (ASTM D4945-89). This method is said to give better definition of point of impact and improved insight into the total dynamics than the old method, and does not require constant cross-sectional area drill stem.

While these dynamic methods might be very useful for research investigations, they are very dangerous for estimating the liquefaction potential of a given site. The empirical correlation developed by Seed, and all the other researchers, are based on a large history of tests run, and results corrected, in the customary way. The idea of actual, real, energy input into the SPT is not taken into account, only the customary apparent energy given by the corrected blow count $(N_1)_{60}$. Being a strictly empirical correlation, it is based on a particular field implementation and is therefore only valid for the same method of implementation. The improved instrumented SPT devices might give more accurate data, but for now this more accurate data has minimum value for estimating liquefaction potential.

2.2.7 Cone Penetration Test

The Cone Penetration Test (CPT) is a penetration test that is extremely common in Europe (Mitchell, 1988). Over the years it has become more popular in the United States. In simplest form, the CPT is a 36 mm diameter steel cone with a sixty degree apex angle. The cone attaches to steel rods and is pushed into the soil at a constant rate of 20 mm/s. The tip resistance, q_c, is measured. In addition, there is a sleeve directly behind the cone tip which has a surface area of 150 cm^2 and measures friction along the sleeve, f_s. The test is procedure is well standardized and mechanized so that operator predilection has no effect (De Beer et al., 1988).

Since the test first appeared in the Netherlands in the 1930's, there have been many versions and improvements. The current incarnation uses load cells to continuously log q_c and f_s, incorporates piezometers to monitor pore water pressure, and inclinometers so that the deflection from vertical can be taken into account in calculating the actual depth of the device at any time. All these parameters can be continuously logged so that thin layers can be discovered. There are also additions to the device such as the Seismic CPT and the Nuclear CPT (Mitchell, 1988), which will be discussed in detail in a later section.

The CPT is a powerful tool for defining stratigraphy. Since the information is logged continuously, a complete record of the soil profile is produced. This is in contrast to the SPT which gives one average value every 450 mm. In fact, this is the foremost advantage of the CPT over the SPT (Tokimatsu and Seed, 1987; Seed and Idriss, 1981). Castro (1991) gives an example from a study of the Lower San Fernando Dam where the SPT did not identify a dangerous, thin, sand layer disclosed by CPT logs. The CPT test is not infinitely sensitive to thin layers, however. If a soft layer overlaying a stiff layer is too thin, the cone will measure some composite value combining the resistance of the two. The results are smoothed profiles rather than sudden, large, changes in soil properties.

An early, very unwieldy, attempt for direct correlation between CPT tip resistance and liquefaction potential was made by Robertson and Campanella (1985; Robertson et al., 1983). Unfortunately, there are very few case histories giving a direct correlation between CPT values and liquefied soils, so the main work has been to develop a correlation between the SPT and the CPT. The chief disadvantage for using the CPT to measure soil liquefaction potential is that the test is basically a drained test while liquefaction is an undrained phenomenon. It is believed that this inconsistency limits its use to merely providing correlations to the results (Tokimatsu, 1985). In addition, the CPT does not retrieve a soil specimen.

Another early correlation was made by Olsen (1984), a complex affair which involved both laboratory testing and cross-correlations, and the iterative solving for exponential coefficients. In the author's own prophetic words, "The CPT should be used first as a stratigraphy tool and second as a means of liquefaction assessment." Seed and De Alba (1986) derived a better correlation that took into account the effect of grain size on CPT results. This method seems to be superior; however, there are no soil samples taken with the CPT by which to accurately judge grain size. Others have defined different correlations and boundary curves (Shibata, 1987; Ishihara, 1985). Probably the best technique is the development of a correlation between the SPT and CPT on a site by site basis (Muraleetharan et al., 1991).

While the CPT is not a very good indicator of liquefaction potential in itself, its ability to delineate stratigraphy must not be forgotten. It will be seen that the device can be combined with some other techniques to yield an instrument of very great power.

Before ending discussion of the CPT, an unique interpretation of CPT results by Been (Been et al., 1986a & b) deserves discussion. He notes that the common correlations between q_c and a given material property assumes that all sands react the same for a common relative density and stress level. This, obviously, is not the case. Been proposes to use the CPT to measure the "state" of each sand, which is referenced to each sand's individual behavior.

The "state" of a sand refers to the steady-state concept (Poulos, 1981; Castro and Poulos, 1977) and has its roots in Casagrande's "critical void ratio" (Casagrande, 1940). Poulos describes the concept thusly (1981):

> The steady-state of deformation for any mass of particles is that state in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant velocity. The steady state of deformation is achieved only after all particle orientation has reached a statistically steady-state condition and after all particle breakage, if any, is complete, so that the shear stress needed to continue deformation and the velocity of deformation remains constant.

The steady-state is reached after large strains when the orientation of the particles reach an equilibrium with no changes in volume with further strain. The residual shear strength, S_{us}, is the undrained shear strength at large strains, and is the strength a liquefied sand can mobilize (Castro and Poulos, 1977), often referred to as the residual strength of a soil (Casagrande, 1940).

The state concept of a soil can be most easily understood by examining a plot of the steady-state line. The full characterization of the state of the soil is contained in the shear stress, effective normal stress, and void ratio, and can be shown in a pair of 2-D plots. However, Fig. 2.4 shows the void ratio-effective normal stress relationship, the most useful for the purpose of understanding liquefaction. A loose (contractive) soil weakens with strain since, in the undrained case, pore pressure increases and lowers the effective stress. In the drained condition, the volume decreases (contracts) until the material reaches the steady-state. This is the type of sand that undergoes liquefaction, since the undrained steady-state can withstand very little stress.

A dense (dilative) soil strengthens with monotonic strain as negative pore pressure develops. Under cyclic loading this type of sand undergoes cyclic mobility when pore pressure momentarily reaches the effective confining stress, accompanied by a limited accumulation of strain. However, the soil mass remains stable since it has a S_{us} equal to the static case, and greater than the imposed cyclic stress. In summary, a sand looser than the steady-state can catastrophically liquefy during an earthquake, while a sand denser will only undergo a moderate but controlled amount of displacement. These results are very reminiscent of early work done on static loading of loose sands (Bjerrum et al., 1961). In

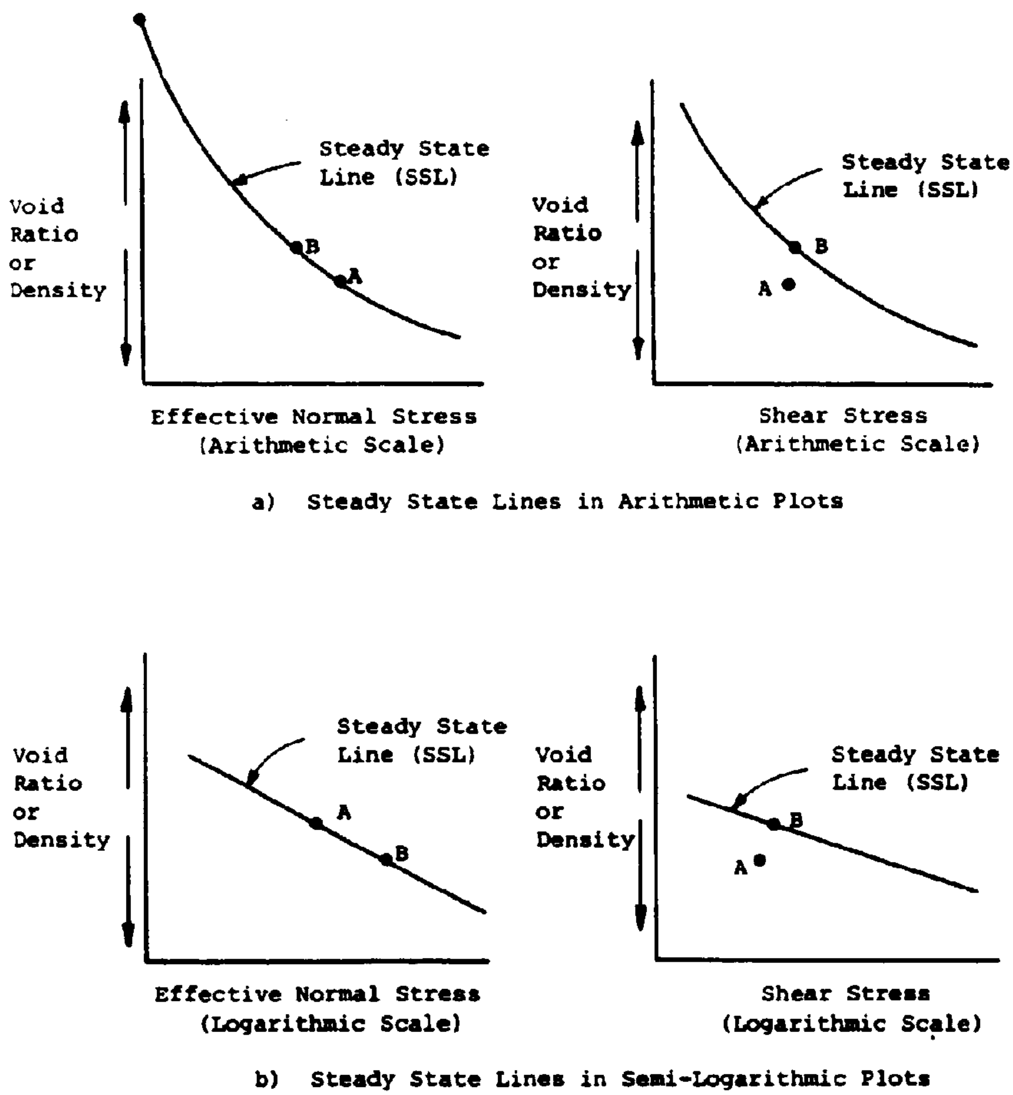

Fig. 2.4 Examples of a steady-state line shown in (a) arithmetic, and (b) semi-logarithmic plots (from Enos et al., 1982).

this approach, void ratio, or relative density, becomes the key parameter to measure. Note that this definition of "liquefaction" is very similar to the more recent views of Dobry (1989) which implies that only the sites that liquefy in this strict sense (not cyclic mobility) are included in Seed's classification.

The state parameter describes the state of the sand combining the void ratio and effective stress uniquely for each sand. Been proposes to measure the state parameter in situ with the CPT. Been presents a correlation between the state parameter and q_c, and some laboratory tests in a calibration chamber which he claims allows him to estimate the state parameter to within ten percent of the likely in situ range. Unfortunately, the data presented is not compelling. The concept is interesting and could yield important information about the expected type of behavior during earthquake excitation. However, a more convincing case needs to be made, especially a better explanation of the physics behind the correlation. Also, the collection of needed parameters such as K_0 is not trivial, and a full set of laboratory tests would need to be run for each sand to derive the correlation.

2.2.8 Other Penetration Tests

The CPT has been used to "carry" other type of tests into the soil. One such device is the Vibratory Cone Penetrometer Test (VCPT). In this device a 200 Hz vibrator is built into a CPT device. The vibrations supposedly build up undrained pore water pressures, and accordingly decrease strength, as in a real earthquake. The difference between static tip resistance, q_{cs}, and vibrational tip resistance, q_{cv}, should give an indication of liquefaction potential. The liquefaction potential D is given by:

$$D = \frac{q_{cs} - q_{cv}}{q_{cs}}. \tag{2.5}$$

where q_{cs} = static tip resistance
q_{cv} = vibrational tip resistance
D = liquefaction potential.

As D approaches 1, the liquefaction potential increases.

A brief description of this device was given by IMSES of Italy in 1985 (Mitchell, 1988), but the Japanese reported a test sequence in 1985 (Sasaki et al., 1986). The correlations between D and the cyclic stress ratio from laboratory tests presented by Sasaki et al. (1986) were not compelling, and it would be difficult to make a decision as to whether a site would liquefy or not. The results from plotting D versus triaxial stress were better correlated but were not sensitive to the known effects of overburden stress on liquefaction potential.

This device is believed to show some promise (Farrar, 1991), and has many advantages such as allowing a fairly continuous record of liquefaction potential, accurate results for sands with a large fines content, in situ results that are independent of laboratory problems, and the rapid and inexpensive nature of the method. However, at this time the results are qualitative at best. More studies need to be undertaken to ascertain D at sites that have liquefied. This should be an affordable task in Japan where there are a large number of potential sites in close proximity. The effects of different vibration frequencies and rate of advance need to be studied too.

Another promising penetration-type test is the Flat-plate Dilatometer Test (DMT) (Lutenegger, 1988; Marchetti, 1975). As the name suggests, the DMT is basically a thin (14 mm), flat plate that is pushed into the ground in a quasi-static manner. The reasoning is that the plate is so thin that there is "no disturbance" of the soil to be tested. In the center of the plate is a 60 mm diameter flexible diaphragm. At the desired depth, the back of the diaphragm is pressurized, and the pressure at which the diaphragm *JUST BEGINS* to move, pressure P_0, is measured. A second variable P_1 is the pressure applied when the diaphragm expands 1 mm. Various indices are calculated from these two pressure values, the most important for liquefaction analysis being the Horizontal Stress Index, K_D, given in Eq. 2.6.

$$K_D = \frac{P_0 - u_0}{\bar{\sigma}_v} \tag{2.6}$$

where K_D = horizontal stress index
P_o = pressure at initiation of diaphragm movement
u_0 = in situ pore water pressure
$\bar{\sigma}_v$ = vertical effective stress.

The traditional application of the DMT is to use K_D as the index parameter for evaluating liquefaction potential (Robertson and Campanella, 1984; Marchetti, 1982). It is believed that K_D is sensitive to the relative density, in situ stresses, stress history, cementation, and ageing of a soil. These are parameters that have been shown to affect the liquefaction potential of a soil. How these parameters affect the stress index is not known (Reyna and Chameau, 1991). Marchetti (1982) proposes a simple boundary between liquefaction and non-liquefaction in terms of Seed's cyclic stress ratio:

$$\frac{\tau_{av}}{\bar{\sigma}_0} \approx \frac{K_D}{10}. \tag{2.7}$$

The correlation of Robertson and Campanella (1984) is designed for drained (slow) penetration and expansion, and gives results much more conservative than Marchetti.

Reyna and Chameau (1991) propose a different boundary line since they maintain that Marchetti is incorrect for small earthquake excitations while Robertson and Campanella is too conservative for large earthquake excitations. However, all this correlation is based on three field tests in the Imperial Valley, CA, and the other correlations are based on one set of data.

While the proponents of the DMT tout the good repeatability of K_D, there are serious problems associated with the test. The horizontal stress index is a function of P_o, the pressure required to just begin to move the membrane. This means that only a extremely small volume of soil immediately in contact with the inserted plate is being tested. The results are for a disturbed soil, but as Schmertmann (1984) says,

> ...even hydraulic pushing may introduce enough vibrational disturbance (or possibly only displacement disturbance in extremely loose sands) to exceed some "critical" level and compact/collapse/densify the sand structure to almost the same degree as the driving vibrations.

The DMT measures disturbed soil while the SPT measures the actual disturbing of the soil. Attempts to define the shear modulus of the soil with DMT parameters have also yielded poor correlations (Thomann and Hryciw, 1991; Belotti et al., 1986).

2.3 Non-Penetration - Intrusive Tests

2.3.1 Strain-based Approach to Cyclic Behavior of Sands

In opposition to the stress-based approach to dynamic behavior of sands represented by Seed is a group that presents a strain based approach (Dobry et al., 1982). The fundamental parameter becomes an elastic constant, the shear modulus G. Knowledge of the shear-wave (S-wave) velocity becomes important due to the basic elastic equivalency

$$G_{max} = \rho V_s^2 \tag{2.8}$$

where G_{max} = elastic shear modulus at small strains
ρ = mass density
V_s = shear-wave velocity.

An early approach to assess the liquefaction potential of sands was based on the existence of a threshold strain for sands (Dobry et al., 1981). Determination of the cyclic strength of an in situ soil is not a simple task since laboratory measurements are very sensitive to sampling disturbances. Among the properties affecting cyclic strength that can be disturbed during laboratory testing are: relative density, soil fabric, overconsolidation ratio, time under static pressure (aging), and previous exposure to shaking. The common method of determining a soil's potential for liquefaction has been a correlation between SPT values and field data. Dobry et al. (1982) attempts to move beyond this strictly empirical approach, which can only be approximate by nature.

Based on the results of Dobry et al. (1982), shear strain, rather than shear stress, is shown to be the fundamental factor controlling buildup of pore water pressure during cyclic loading, i.e. liquefaction/cyclic mobility. This seems intuitively obvious since in the real world strain is the parameter controlling densifying behavior. In deriving a model for the liquefaction and densification of sands, Nemat-Nasser (1986) notes

> ...the distribution of the orientation of the contact normals measured relative to the normal of the overall macroscopic shear plane...this distribution has a profound effect on the sample's potential to densification under drained conditions and, therefore, on its liquefaction potential when saturated and undrained. This result also suggests that the distribution of the dilatency angles and, therefore, the fabric of a granular material in simple cyclic shearing, is more directly related to the total shear strain rather than to the shear stress.

Seed himself says (Seed et al., 1983, p.476)

> In reality, however, liquefaction is a phenomenon which results from a tendency for volume decrease in a sand due to application of cyclic shear strains..., and volume changes are more uniquely related to cyclic strains than cyclic stresses...

Laboratory strain-controlled tests yield results that are more independent of differences in variables such as density, fabric, and sample disturbances than stress controlled tests. Also, the principle factors that control liquefaction susceptibility control soil stiffness.

The key concept of the strain approach is that of "threshold strain", γ_t. This is the strain at which the sand grains actually start sliding relative to each other as opposed to merely deforming elastically. The relative motion of grains is what causes contraction or dilation of the soil fabric and builds up pore water pressure (Dobry et al., 1992). The threshold strain is related to earthquake excitation by threshold ground surface acceleration, A_t, which is the surface acceleration that causes pore water pressure buildup to initiate:

$$\frac{A_t}{g} = \frac{\gamma_t \left(\dfrac{G}{G_{max}}\right)_t}{g Z r_d} V_s^2 \tag{2.9}$$

where A_t = threshold ground acceleration
g = acceleration of gravity
γ_t = threshold strain
$(G/G_{max})_t$ = modulus reduction factor at threshold strain.
Z = depth from ground surface
r_d = Seed's depth reduction factor

The modulus reduction factor reflects the weakening of the sand when the grains begin to move relative to each other ($\gamma > \gamma_t$). The concepts of threshold strain and strain weakening are illustrated by the stress-strain curve for a typical sand shown in Fig. 2.5. In this case the threshold strain level is approximately 0.001 percent, where the stiffness begins to degrade with increasing strain.

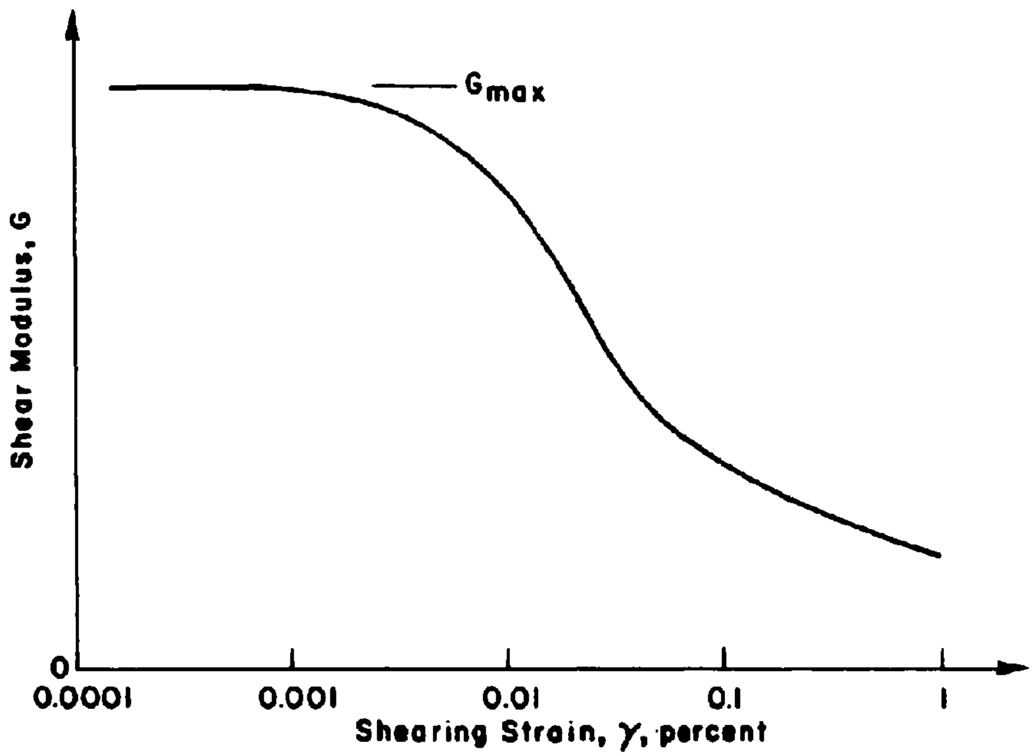

Fig. 2.5 A generic modulus reduction curve for a clean sand (from Hoar, 1982).

In practice, threshold strain is estimated since field geophysical tests can only induce very small strain(e.g. $1x10^{-8}$) , and laboratory tests show that the value lies in a narrow band between $1x10^{-4}$ and $2x10^{-4}$, and only rises to $3x10^{-4}$ for very overconsolidated sands (Ladd et al., 1989). The modulus reduction curve used is derived from laboratory experiments (Iwasaki et al., 1978; Seed and Idriss, 1970). For a given γ_t, the modulus reduction factor can be read from a general curve since the reduction factor has been shown to be very insensitive to sand type, testing technique, fabric, and overconsolidation (e.g. Tokimatsu and Seed, 1987; Hardin and Drnevich, 1972). The cause of this insensitivity might be that once the grains start moving past each other, much of the influence of these parameters are erased. However, the reduction factor is always influenced by confining pressure

Equation 2.8 shows V_s as the measured parameter yielding G_{max}, which can be known exactly if density is measured rather than estimated. The problem with density is secondary since it lies in a fairly narrow range for sands and is only raised to the first power (see Eq. 2.8). Shear velocity has a large range for different sands and locations. It is interesting to note that the age of the sand has a very large effect on increasing V_s. Youd (Youd and Hoose, 1977) notes that, "Holocene deposits have been more disturbed by liquefaction than Pleistocene deposits." This might be due to "preloading" from earlier shaking increasing compaction, K_0, and toughening fabric; in addition to a greater probability of physicochemical cementation and overconsolidation. Charts are presented to show the influence of γ_t and V_s on resistance to pore water pressure buildup (Dobry et al., 1982).

An overconsolidated layer will have a very high resistance to liquefaction partly due to a high threshold strain. However, there might be a problem in measuring the overconsolidation ratio in the field in order to reproduce it in the laboratory when measuring γ_t. One solution is consolidating the laboratory sample until V_s is equal to that in the field (Tokimatsu et al., 1986).

There are several advantages of using the strain approach to estimating liquefaction potential. The most important is that it is based on the in situ measurement of shear-wave velocity of *UNDISTURBED* soils. Since the strains involved in the geophysical techniques are very small, there is strong theoretical grounding to this testing method based on the theory of elasticity. For an important part of the analysis, there are no empirical correlations involved. Measured shear-wave velocities are valid for the near-surface deposits of interest in lifeline analysis, and can be run in hard-to-sample soils such as gravel. The material properties are not affected, and since a large volume of soil can be involved, macro-fabric can be tested.

The strain approach is not without limiting assumptions. Dobry et al. (1982) point out several important assumptions made in their presentation. The soil unit weight was held to be constant and not changing with depth. Particular estimated values of threshold strain and modulus reduction factor were used. Finally, the charts show an increasing propensity for liquefaction with depth (since depth is in the denominator of the Eq. 2.9). This is not the case in reality, since V_s increases with depth as a result of increasing density, age, and confinement.

There are also problems associated with this method of estimating potential for liquefaction. Since what is being estimated is the initiation of pore water pressure buildup, the method will be very conservative since effects only manifest themselves when the pore water pressure approaches the effective confining stress. However, correlations between V_s and sites that have actually liquefied or not during earthquake excitation have been successfully made in much the same manner as done for the SPT, although at this time too few correlations have been made to establish statistical significance. The advantage for the use of V_s would be those mentioned in the previous paragraph. It will also be shown that velocity surveys can be done cheaply, rapidly, and accurately.

2.3.2 Shear-wave Velocity Measurements

Dobry's interpretation of the behavior of particulate media as being controlled by shear strain (Dobry et al., 1982) implies that knowledge of a soil's stiffness can be used to estimate liquefaction susceptibility. This stiffness-based approach is relatively new, and velocity measurements are not routinely made during soil investigations. Therefore the data base is not nearly as large as for the SPT approach. There is a small but growing literature showing the efficacy of the stiffness approach. Many case studies comparing seismic velocities to liquefied sites have been made (Stokoe et al., 1989; Stokoe and Nazarian, 1985; Nazarian and Stokoe, 1984). Another study (Stokoe et al., 1988) applies the shear-wave method to two hard-to-sample soils, a debris slide at Mount St. Helens, Washington, and a liquefied gravel at Borah Peak, Idaho, with good results. Other workers using this approach include De Alba (De Alba et al, 1984) and Sirles (1988). In Japan, Tokimatsu (Tokimatsu and Uchida, 1990; Tokimatsu et al., 1986), and Satoh (Satoh et al., 1991) have found useful relations between S-wave velocity and liquefaction potential.

Knowledge of the shear-wave velocity of a soil profile is also important for seismic ground motion analysis. The increase in displacement amplitude as a wave travels from layer to layer is inversely proportional to the layer impedance $\surd(\rho V_s)$ (Aki and Richards, 1980). Joyner and Fumal (1984) report a field study at 33 strong motion sites where S-wave velocity was used to successfully predict site behavior based on impedance contrasts through the column.

Over the years, many methods of measuring seismic velocity have been developed. The most important ones from the point of view of predicting liquefaction potential will now be examined. Velocity itself cannot be measured directly. Instead, the time of travel across a known distance is measured and velocity calculated by Eq. 2.10:

$$Velocity = \frac{Distance}{Time}. \tag{2.10}$$

In this case, what is timed is the acoustic wave mode of interest, the Shear-wave. Other wave modes, such as the Rayleigh-wave can be used for other purposes. However, there is a problem with using the Primary-wave mode since the soils of interest are saturated and have a P-wave velocity much slower than water. The resultant velocity from a P-wave measurement in near-saturated soils is the velocity of sound through water.

2.3.3 Cross-Hole Method of Measuring Shear-Wave Velocity

The most straight-forward method of measuring S-wave velocity is the cross-hole technique. As the name implies, the measurement is taken between two receivers at equal depth, with the energy traveling only through the horizontal layer of interest. The usual setup uses three bore-holes, one as the source, and time measured over the interval between the other two holes. The interval method provides a more accurate timing with the effect of the higher-speed bore-hole casing canceled (Hoar, 1982). Sources can be chosen to input only the wave mode desired: P-, SV-, or SH-waves. This test can be run with a great deal of accuracy and serves as a velocity benchmark.

The one technical problem associated with the cross-hole test is refraction. If a slow, thin layer overlies a fast layer, the first arrival is not the direct S-wave, but the critically refracted head-wave (Hoar, 1982). Also, if the velocity increases through a layer, the ray-path becomes curved. These are small problems that can be simply accounted for (Hiltunen and Woods, 1988). The main impediment to the wide-spread use of the cross-hole test is the cost. The test requires three bore-holes at each test location, carefully cased holes, an inclinometer so that the actual distance between inclined holes can be calculated, and time.

2.3.4 Down-Hole Method of Measuring Shear-wave Velocity

The down-hole method in an intrusive seismic test that requires just one bore-hole. The receiver is lowered in the hole to desired depths, and a shear-wave source is activated on the surface some small distance from the bore-hole. Care must be taken to guard against the signal traveling down the fast casing and chosen as the first arrival (Hoar, 1982). The energy travels on an inclined path, through all the intervening layers between the surface and depth of receiver. This method is cheaper, quicker, and easier than the cross-hole method.

Since the signal travels through the entire profile, from surface to depth of receiver, the measured velocity becomes a weighted average of the profile, and thin layers can easily become lost. Part of this smearing mechanism is that Snell's law shows that the travel path is not straight because of the different impedances of different layers. Mok (Mok et al, 1988; Mok, 1987) sets forth a solution to this problem based on inverse theory (Menke, 1984). In this method the ground is modeled as a stack of horizontally homogeneous layers. Inverse theory is applied to back-calculate a profile that gives the best match to the measured data in the least-square sense. In several case studies, Mok has shown this method able to accurately discriminate the actual various layers.

2.3.5 Seismic Cone Penetrometer for Measuring Shear-wave Velocity

An interesting variant to the down-hole test is offered by an extension of the cone penetrometer - the Seismic Cone Penetration Test (SCPT). This device first appeared in 1986 (Robertson et al., 1986), and eliminates the need for a bore-hole to obtain down-hole velocities. With this device, geophones or accelerometers are included in the CPT. The velocities are measured in the standard way during short pauses in the CPT penetration. The application of Mok's inversion technique could mitigate some of the averaging problem inherent in the down-hole method, while the CPT would yield the detailed stratigraphy needed to increase the robustness of the inversion calculation.

The few reports of case studies (Campanella and Stewart, 1991) indicates the promise of this device. However, there is at present a dearth of independent case studies to prove the efficacy of the tool. A slightly different application report uses the SCPT in cross-hole tests (Baldi et al., 1988) with very good results.

2.4 Other Non-Penetration — Intrusive Tests

2.4.1 Nuclear Density Gage

Extremely interesting reports have been published about using a nuclear density gage, similar to the gamma-gamma loggers commonly used by the petroleum industry, down a bore-hole to quantitatively measure in situ soil density (Plewes et al, 1988; Cowherd, 1986). The device itself is quite small and can operate in a 50 mm diameter hole. Similar gages have been used during highway construction for years to verify compaction of fill and sub-base, and are covered by ASTM Standard D 2922-81. Use of these gages to ASTM specification gives a result with error (one standard deviation) between two and three percent.

The in-hole devices use gamma ray backscattering to measure density. Photons are emitted from the gamma source, which then interact with the electrons of the media being tested. Through the Compton effect, this interaction releases weaker, scattered photons. The rate of occurrence of the scattered photons is proportional to the electron density of the host media, hence to the bulk density of the media. If the density of the skeleton and pore fluid is known, porosity can be measured with this method. Newer devices use up to an eight millicurie source, strong enough to allow the hole to be lined with steel casing if needed. The volume of material tested is also quite large, so that the actual density of virgin material is measured. For an average sand, the weighted volume of influence is approximately 160 mm in radius, and 300 mm in height.

For accuracy greater than the ASTM specification and complete accounting for the effect of the casing, a calibration curve can be made in the laboratory. An added advantage of this device is that actual density is measured exactly and cheaply by measuring the in situ density, and comparing the value to the minimum and maximum densities for clean sand measured in the laboratory. Knowledge of relative density is one of the prime factors controlling the sand's potential to liquefy, and was also shown to be the prime variable for the Poulos/Castro steady-state approach to sand behavior. This method shows great promise for making the steady-state approach a viable method of in situ analysis.

Varieties of this device also measure in situ moisture content using fast neutrons, giving more information about the undisturbed state of the soil. This measurement is covered by ASTM Standard D 3017-78.

The down-hole density gage was used to verify the improvement of in situ density at a mine tailings dam after a soil improvement program was undertaken (Halley and Jacobs, 1988). A large embankment (73 m high, 1100 m long) in the Dominican Republic was found to be unable to withstand the design earthquake due to the low relative density of the material. A densification program was initiated, and in situ density successfully monitored using a portable borehole logger. Calibration in the laboratory allowed fast and accurate measurement of in situ soil density.

The Dutch have combined a similar device in a cone penetrometer (Sully and Echezria, 1988; Nieuwenhuis and Smits, 1982). This combination eliminates the need for a borehole, making the method even less expensive and faster. The Dutch probe measures approximately 0.5 m into the surrounding soil. A similar device has been constructed in France (Ledoux et al., 1982), and has proven itself useful for identifying inclusions with similar penetration resistance as the host material, such as peat beds in soft clay.

2.4.2 Suspension Device for Measuring Shear-wave Velocity

An alternative non-penetrating test that requires a bore-hole is the suspension logger. This device was invented by Kitsunezaki (1980) to be freely suspended in a fluid-filled bore-hole, with the fluid the couplant between the device and the surrounding material. The source is built into the top section of the sonde and imparts a doublet-type impulse into the sidewall through the incompressible fluid. The horizontal displacement at this point causes a shear displacement further down the bore where the receiver is located at the bottom of the sonde (about one meter). The shear field is coupled to the neutral buoyancy receiver by the bore fluid. A pressurized collar separates the upper and lower sections of the bore to minimize coupling through the bore fluid.

This device is now being marketed (Ohya, 1986). It is not surprising that the device works very well in strong materials that do not require casing and will not exhibit significant softening due to the excavation of the bore. However, from Kitsunezaki's paper it appears that the displacement being measured is for the bore wall only, not deep in the material free field. Softening due to unloading after excavation would produce slower velocities, and larger displacements, than representative. Also, in cased holes the test would give more information about the casing than the weak material behind. The company who markets this device admitted these problems in theory but claimed that in practice "correct" values are measured (Michalson, 1992).

This device is interesting in concept and is probably a very effective tool for velocity surveys in tight formations. However, liquefiable materials are by nature very loose sands that would loosen near-bore. These materials are also near surface where fluid pressure

alone would not be able to keep the bore-hole open, necessitating a cased hole. Therefore, the suspension logger does not seem useful for estimating liquefaction potential without independent trial or further theoretical backing.

2.5 Non-Intrusive Tests

2.5.1 Spectral Analysis of Surface Waves

The Spectral-Analysis-of-Surface-Waves (SASW) method is a relatively new non-intrusive testing method for determining shear wave velocity profiles of soil sites. This in situ test is nondestructive and is performed entirely from the ground surface. While the earliest attempt to make use of this method was proposed by Jones (1958), the method has only been practical since 1984 (Nazarian, 1984; Nazarian and Stokoe, 1983). Measurements are made at strains below 0.001 percent where elastic properties of the materials are independent of strain amplitude, yielding a calculation of G_{max}.

The displacements being measured in SASW testing are due to the surface or Rayleigh-waves (R-waves). The R-wave travels along the surface of the earth, effectively traveling through a depth of material proportional to wavelength. Practice has shown that the thickness of material sampled by the R-wave is approximately one-third of the wavelength (Gazetas, 1982; Heisey et al., 1982). The source and two receivers are located on the surface, so there is no destructive intrusion into the ground. The source is usually quite simple, generally a hammer of some sort. For shallow surveys a small hammer can be used to generate the higher frequencies of interest. For deep surveys, a large dropped weight has been used to generate the very long wavelengths required.

For a homogeneous, isotropic, elastic material, each frequency of vibration input into the medium will travel at the same velocity. Frequency, Rayleigh wavelength, and R-wave velocity are related by the fundamental equation:

$$V_R = f \cdot \lambda_R \tag{2.11}$$

where λ_R = Rayleigh wavelength
f = frequency
V_R = R-wave velocity.

It has already been stated that different frequencies travel through different depths of the soil. Therefore (from Eq. 2.8 $V_s^2 = G/\rho$), if the stiffness of the material varies with depth, different frequencies will travel at different velocities. These frequency-specific velocities are known as the phase velocities, and the phenomena known as dispersion. Dispersion allows different depths to be sampled from the surface and is the key to SASW.

The hammer blow, or other impact, that is used as the source yields a signal that is made up of many frequencies. Spectral analysis is the tool used to break up the signal into frequency components and facilitate the construction of the dispersion curve. This analysis and the subsequent inversion of the dispersion curve to determine the S-wave profile are computer intensive and is a major reason for the late development of the surface wave technique.

In a field test, two receivers are used to run a SASW test. An imaginary centerline is drawn on the ground and the two receivers placed an equal distance D from this line. The source is input along the centerline and a distance 2D from one of the receivers. The signals from the two receivers are digitized and recorded, and can be stacked for increased signal-to-noise ratio. The source is then moved to a distance 2D on the other side of the array to minimize errors in the data (Nazarian and Stokoe, 1984). This process is repeated at increasingly larger distances, doubling D each time.

The signals from the two receivers are used to compute the cross power spectrum and coherence function (Newland, 1984). The coherence is used to determine the quality of data on a frequency-by-frequency basis. Only the information that is not contaminated by background noise or other sources is used for further analysis. The relative phase shift at each frequency is taken from the cross spectrum and used to calculate the travel time across the known receiver separation distance:

$$t = \frac{\phi}{360f} \tag{2.12}$$

where t = travel time between receivers
ϕ = relative phase shift (degrees)
f = frequency (Hz).

and the frequency by frequency phase velocity is then calculated by Eq. 2.13:

$$V_{ph} = \frac{D}{t}. \tag{2.13}$$

where V_{ph} = phase velocity
D = receiver separation distance

Wavelength is now simply calculated by applying Eq. 2.11.

The dispersion curve is made by plotting the wavelength against the matching phase velocity, as shown in Fig. 2.6. Physical significance can be observed by remembering that each wavelength is roughly associated with a depth of 0.33 λ_R. Many redundant points are plotted on the dispersion curve since there is overlap of information for the different receiver separations. This also fills in gaps left by areas of low coherence for a given separation. The manual construction of dispersion curves is very tedious and requires operator judgement to eliminate excessive and low-quality date points (Hiltunen, 1981). To solve this problem, Nazarian (Nazarian and Desai, 1993) has automated the construction of the dispersion curve and following inversion process. Other solutions to make the construction of the dispersion curve more efficient have been proposed (Gucunski and Woods, 1991; Satoh et al., 1991).

At this point, the in situ shear-wave velocity profile must be deduced from the experimental R-wave dispersion curve, an inverse problem (Menke, 1984). The most simplistic solution is to assume that the S-wave velocity is 110 percent of the phase velocity and the depth sampled by each frequency is 0.33 of the wavelength. Some workers assume this approach is sufficient (Satoh et al., 1991). However, it is obvious that the resultant velocities will be an average of the velocities of the several layers comprising the effective depth. Inversion avoids this limitation. Inversion is a very intricate and complicated process which has been done using a manual trial and error (forward modelling) approach (Stokoe and Nazarian, 1985; Nazarian, 1984). Recently Nazarian (Yuan and Nazarian, 1993) has been able to automate this process which rapidly estimates the velocity and thickness profile of the site. In addition, an estimate of uncertainty is given.

The input soil parameters needed to apply SASW to a site are Poisson's ratio, and mass density. The effect of density and Poisson's ratio are small (Nazarian, 1984) and are estimated. Layer thickness and S-wave velocities are the unknowns. If the soil profile is known beforehand, e.g. from a CPT log, the velocity solution will be more accurate since averaging across actual layers will be avoided (Rix and Leipski, 1991). It should be noted that the inverse process is not unique, although certain physical concerns allow the determination of a best answer. Unlike the ideal crosshole case, the signal usually travels through more than one layer at a time so that the effect of any one layer can not be absolutely removed.

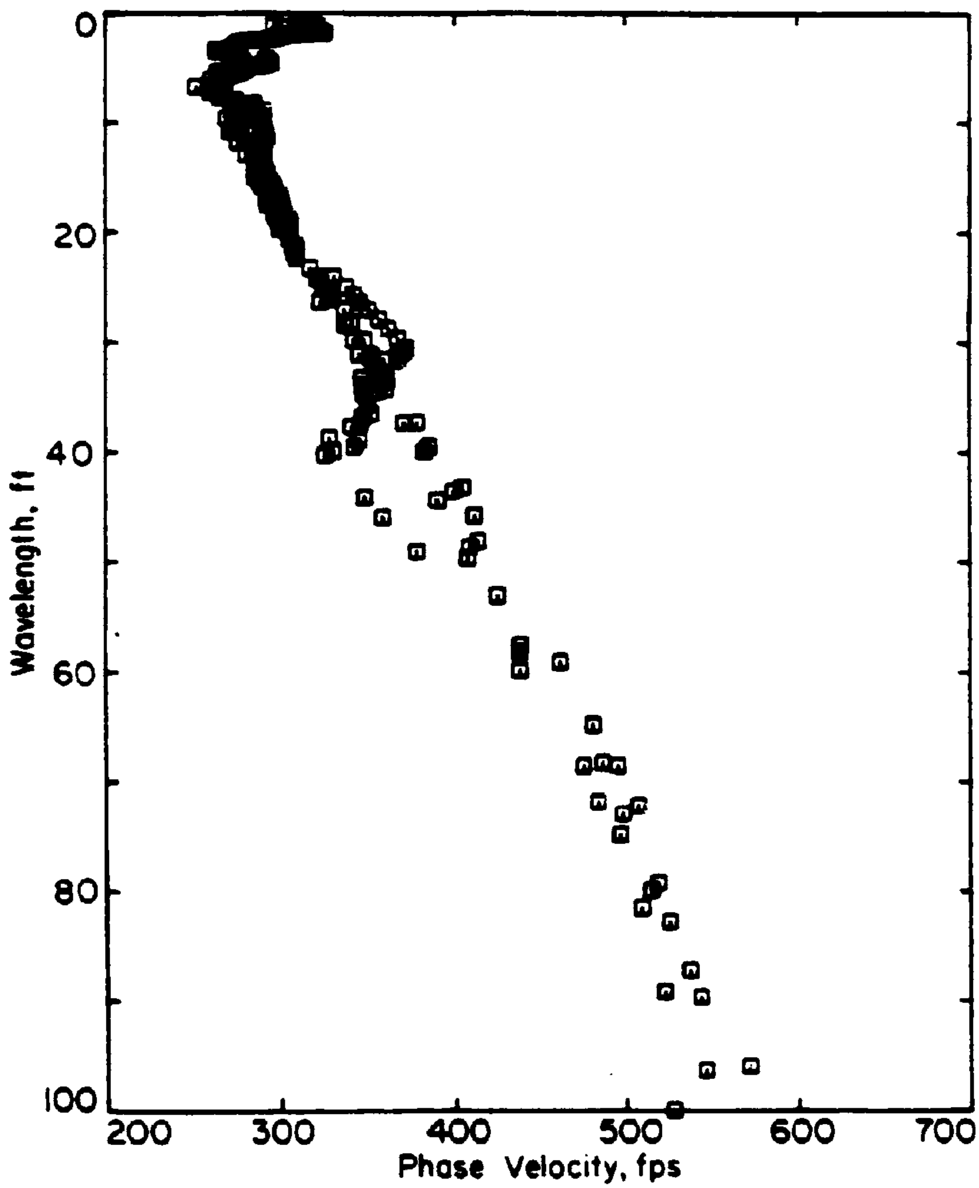

Fig. 2.6 A typical dispersion curve. This curve shows all receiver spacing, including overlaps, for the Wildlife site, Imperial Valley, CA (from Nazarian, 1984).

With the advent of automation of the entire computational process, SASW is a cheap and rapid method of site evaluation. Large areal expanses can be covered since boreholes are not needed. SASW can be tied in with a SCPT so that accurate stratigraphic control can be maintained. A computer-controlled swept frequency shaker can be used as a common source, and the knowledge of the input excitation gives better data and more accurate results.

There have been many field studies undertaken in the last few years to verify the soundness of the SASW method to predict velocity (stiffness) profile. The validity of using velocity or stiffness to predict liquefaction potential has been discussed above and some of the field trials have focused on this problem. Early studies were done at several sites in the Imperial Valley, six of which liquefied either during the 1979 Imperial Valley or the 1981 Westmoreland earthquake (Stokoe and Nazarian, 1985; Nazarian and Stokoe, 1984). Comparison between the SASW results and crosshole results showed a maximum difference of slightly less than ten percent. It was discovered that soils with a S-wave velocity of less than 135 m/s liquefied. Further work indicates that an upper bound of 200 m/s exists (Nazarian, 1992), with soils having a S-wave velocity between 135 and 200 m/s susceptible to liquefaction. Further field work to enlarge the data base could possibly make this estimate as sure as the current state-of-practice technique.

Nazarian gives an example from a site near El Paso, TX where the SASW and crosshole results were virtually identical down to a depth of eighteen meters (Yuan and Nazarian, 1993). In this trial the automated SASW and manual forward modeling SASW gave near identical profiles.

The usefulness of SASW in hard-to-sample material has also been shown (Andrus et al., 1992; Stokoe et al., 1988). Gravel materials at Borah Peak, ID, which liquefied during the 1983 earthquake were tested and found to have a S-wave velocity of between 90 and 120 m/s and liquefaction was successfully predicted using two different velocity prediction methods. Similar studies were made at a debris slide at Mount St. Helens, WA and Lake Jackson Dam, WY.

Studies have been made to compare the velocity profiles from SASW and crosshole tests (Hiltunen and Woods, 1988). It has been previously suggested that the crosshole velocities are usually slightly larger. This study proposes that up to thirty percent of the velocity differences between the two methods can be explained by the crosshole method not accounting for the curved ray-path between receivers.

Private firms are starting to use the SASW method and report good success (Barker and Stevens, 1991). Satoh, while using a rudimentary version without inversion, claims that the method has become routine in Japan (Satoh et al., 1991). Tokimatsu (Tokimatsu et al., 1991) applied SASW in the San Francisco Marina district, with very good correlation between damage and low velocity material. The velocities were less than the 135 m/s threshold.

There are also problems and limitations with the SASW method. The SASW method yields no soil sample for determining soil texture and actual stratigraphy. This is an important limitation. The shear wave velocity measured is for small-strain states, which are very different than the large-strain behavior of the soil during strong motion excitation. Finally, there has been some questions raised as to the uniqueness of data interpretation.

2.5.2 Seismic Refraction

Seismic refraction (Telford et al., 1976) has been used to measure P-wave velocities for a variety of uses, such as oil exploration and site investigation. However, these applications were more interested in higher velocity materials such as rock. This method by nature will smear and average the velocities of intervening layers. It is not believed to be applicable for very near surface exploration of slow, low contrast soil profiles. This method can not discern slow layers underneath faster layers, while SASW can.

CHAPTER 3 ESTIMATION OF SURFACE DISPLACEMENT DUE TO EARTHQUAKE EXCITATION OF SATURATED SANDS

3.1 The Problem of Ground Displacement

3.1.1 Introduction

If liquefaction of sands during earthquakes did not have some effect on the quality of life, studying the problem would only be an academic exercise. In fact, damage and destruction of lifelines and buildings due to earthquakes is often primarily due to effects of liquefaction. Striking examples are the initiation of very large flow slides and lateral spreads during the 1964 Alaska earthquake, the foundation failures due to the 1964 Niigata quake, and the wide-spread damage in the Marina district due to sands re-liquefied by the 1989 Loma Prieta temblor.

While the effects on structures often gather more headlines than the effects the earthquake has on lifelines, lifelines too suffer severe damage. The massive destruction from the 1906 San Francisco earthquake is primarily due to fire caused by damaged lifelines and the subsequent inability to put out these fires due to damage to lifeline systems. The importance of lifelines, and investigation on the effects of earthquakes on these structures, is now recognized as an important problem, with a growing literature (e.g. O'Rourke and Hamada, 1992, 1991, 1989; Hamada and O'Rourke, 1992; Prakash, 1991). Estimates of the economic disruption due to lifeline damage from liquefaction has been made, and range in the tens-of-billion dollars (Bivins, 1992).

The damage caused by liquefying sands is due to attendant soil displacements. This chapter will develop and evaluate many of the methods available for estimating these displacements. This is a necessary first step to the design of lifeline structures that can withstand the displacements induced by earthquake loading, and the evaluation of the safety and reliability of existent structures. This chapter is neither a compendium of displacement values gleaned from case studies, nor a thorough investigation of the nature of the failure mechanisms leading to these displacements (see NRC, 1985).

3.1.2 Failure Mechanisms

Liquefaction is not a single unique behavior of saturated sands due to large cyclic excitations. It is a complex class of responses, some of which were discussed in Chapter 2 and reviewed in detail by the National Research Council document (NRC, 1985). Because of this mechanistic variety, there is also a variety of approaches to estimating the associated deformations. The type of deformation can be broadly broken into vertical settlements and lateral displacements. Vertical settlement is due to the consolidation of the sand grains as they settle out of their liquefied state. Initially there is no settlement since undrained conditions are assumed, but as the excess pore water dissipates, the surface will settle by an amount roughly equal to the water lost from the stratum.

Surface settlement is often associated with the appearance of sand boils. Complete loss of effective stress ($u=\bar{\sigma}_v$) does not have to occur to allow significant settlement. It has been shown that settlements can occur when the excess pore water pressure reaches 50-60 percent of the effective overburden pressure (De Alba et al., 1975). The sand does not necessarily undergo a large degree of consolidation either, since there is a history of subsequent liquefaction of historically liquefied sands (Youd, 1988; Ambraseys and Sarma, 1969).

Lateral displacements can be caused by two general mechanisms; the distinction made by Castro, and discussed in Chapter 2. In review, the steady-state (large strain) behavior of a soil is a function of the packing, or relative density D_r, of the sand grains. If the sand is loose enough to act in a contractive manner when sheared, the sand will be strain-weakening following the peak stress. The sand will continue to contract, and pore pressure continue to build, until the steady state is reached. The sand will fail catastrophically once the peak shear strength is reached, and continue to flow until the driving stress is less than the steady-state strength of the soil. The stress strain behavior is illustrated in Fig. 3.1. This behavior is what is commonly referred to as "liquefaction".

If the sand behaves in a dilative manner when sheared, it is strain-hardening in nature. The steady-state strength is the peak strength of the soil and it will always behave in a stable manner. As illustrated by Fig. 3.2, strain will accompany imposition of cyclic stress, but no further strain will occur after the cessation of the cyclic stress and return to the static state of stress. There can be no catastrophic failure. This response to cyclic loading was called "cyclic mobility" by Castro (e.g. 1975). At this time it is unclear as to which mechanism is involved in lateral spreading.

Since there are at least three broad behaviors by which sands can respond to large cyclic excitation, the methods for estimating the associated soil deformations will also be different. An additional problem is the enormous number of factors that can affect the amount of deformation. These parameters take into account the nature of the liquefiable soil layer, the overlying soil, topography, nearby structures, and parameters of the exciting earthquake. Some of the parameters discussed by researchers to date are given in Table 3.1. This list of twenty-six different governing parameters is informal and probably does not include all that have been considered, but it does give an idea of how many different physical manifestations must be considered in order to make even a preliminary estimate of the amount of ground displacement resulting from an earthquake.

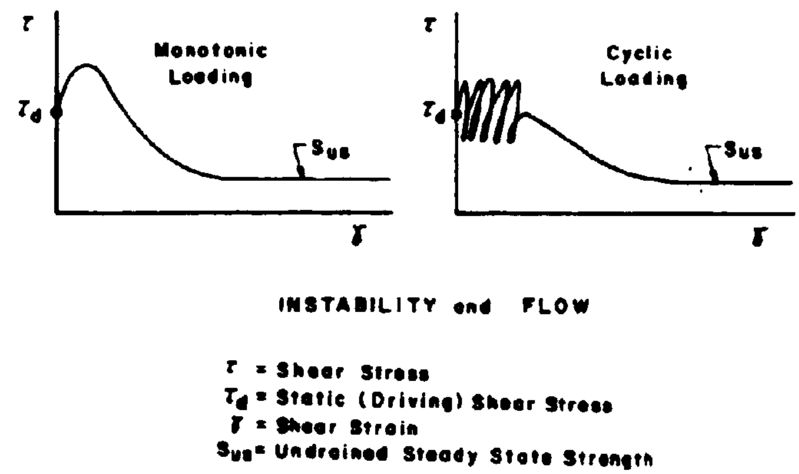

Fig. 3.1 Unstable behavior of contractive soil under static and cyclic loading (from NRC, 1985).

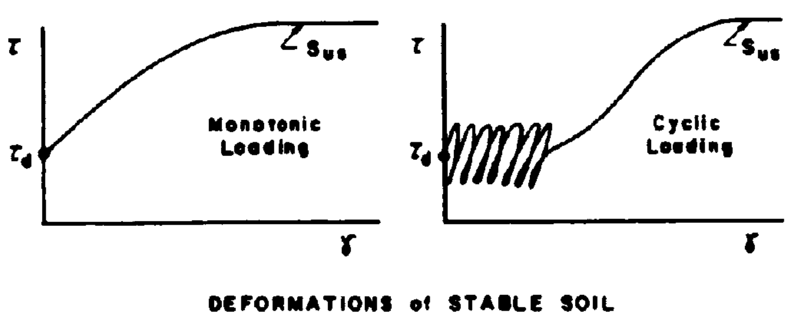

Fig. 3.2 Stable behavior of dilative soil under static and cyclic loading (from NRC, 1985).

Table 3.1 Physical parameters that have been used in estimating ground displacement due to liquefaction, reported in the literature.

1. Slope at the base of the liquefiable layer.
2. Slope of the surface of the liquefiable layer.
3. Length of the slope.
4. Width of the sloping area.
5. Thickness of the overburden.
6. Thickness of the liquefiable layer.
7. Depth to water table.
8. Stiffness of overburden.
9. Stiffness of liquefiable layer.
10. Static shear strength of overburden.
11. Static shear strength of liquefiable layer.
12. Dynamic undrained strength of liquefiable layer.
13. Steady-state strength of liquefiable layer.
14. Stiffness degradation of liquefiable layer.
15. Plasticity parameters of liquefiable layer.
16. Grain size characteristics of liquefiable layer (distribution, size, average size).
17. Relative density of liquefiable layer.
18. Blow-count of the liquefiable layer, $(N_1)_{60}$.
19. Pore water pressure behavior in the liquefiable layer.
20. Hydraulic conductivity of the overburden.
21. Hydraulic conductivity of liquefiable layer.
22. Static driving force.
23. Distance from earthquake source.
24. Intensity of shaking.
25. Duration of shaking.
26. Aerial extent of liquefiable layer, and boundary conditions.
27. Maximum induced shear strain in the liquefiable layer.
28. Maximum induced shear stress in the liquefiable layer.

3.2 Estimating Vertical Deformations

3.2.1 When is Vertical Settlement a Risk?

The generation of excess pore water pressure due to strong ground motion does not necessarily lead to ground deformation. For instance, the appearance of sand boils is an indication of a buildup of pore water pressure, but the appearance of sand boils is usually not associated with an engineering problem (Castro, 1987). However, there has to be a loss of volume in the soil system equal to the loss of water.

It is possible that a buried soil stratum liquefies, but that there is no outward manifestation. The possibility of this happening can be seen as mainly a function of the thickness of the overburden layer and the thickness of the liquefiable layer (Ishihara, 1985). Ishihara collected data from many case studies from around the world, and derived an empirical relation linking the two thicknesses to the manifestation of liquefaction. These relations are given by the chart shown in Fig. 3.3. The implication is, that for an earthquake acceleration of 0.2 — 0.25 g, there will be no surface damage if the overburden is more than three meters thick. Note, however, that this particular rule of thumb has been known not to hold true at sites where lateral spread occurs (Youd, personal communication).

Based on a large field and laboratory study, Florin and Ivanov (1961) state that the in situ state of stress of the liquefiable layer is controlled by external vertical loads, whether from overburden or structures. A thick overburden reduces the ability of the sand to liquefy, and the time the liquefied sand can stay fluid. The results of many field observations convinced them that it is virtually impossible to liquefy even a very loose sand with ten to fifteen meters of cover. In terms of remediation, even a very thin freely-draining cover will reduce the amount of settlement due to shortening the duration of liquefaction (Florin and Ivanov, 1961).

Large vertical settlements can be a real engineering problem. Large total settlements (up to one meter) appeared after the 1979 Imperial Valley, CA, 1906 San Francisco, and 1984 Alaska earthquakes. Differential settlement is an associated problem, causing damage to lifelines after the 1985 Chile earthquake and the 1957 and 1985 Mexico City earthquakes. Based on data from a variety laboratory tests, actual settlements following earthquakes, and centrifuge tests, Dobry (1989) gives expected ranges of volumetric strain shown in table 3.2. These strains show that absolute displacements can be sizable. The associated differential settlement will be even more destructive to structures of all kinds.

Table 3.2 Volumetric strains due to liquefaction of sands (after Dobry, 1989).

Density	Volumetric Strain (ϵ_v%)
loose sand	1.5 to 5.0
dense sand	< 0.2

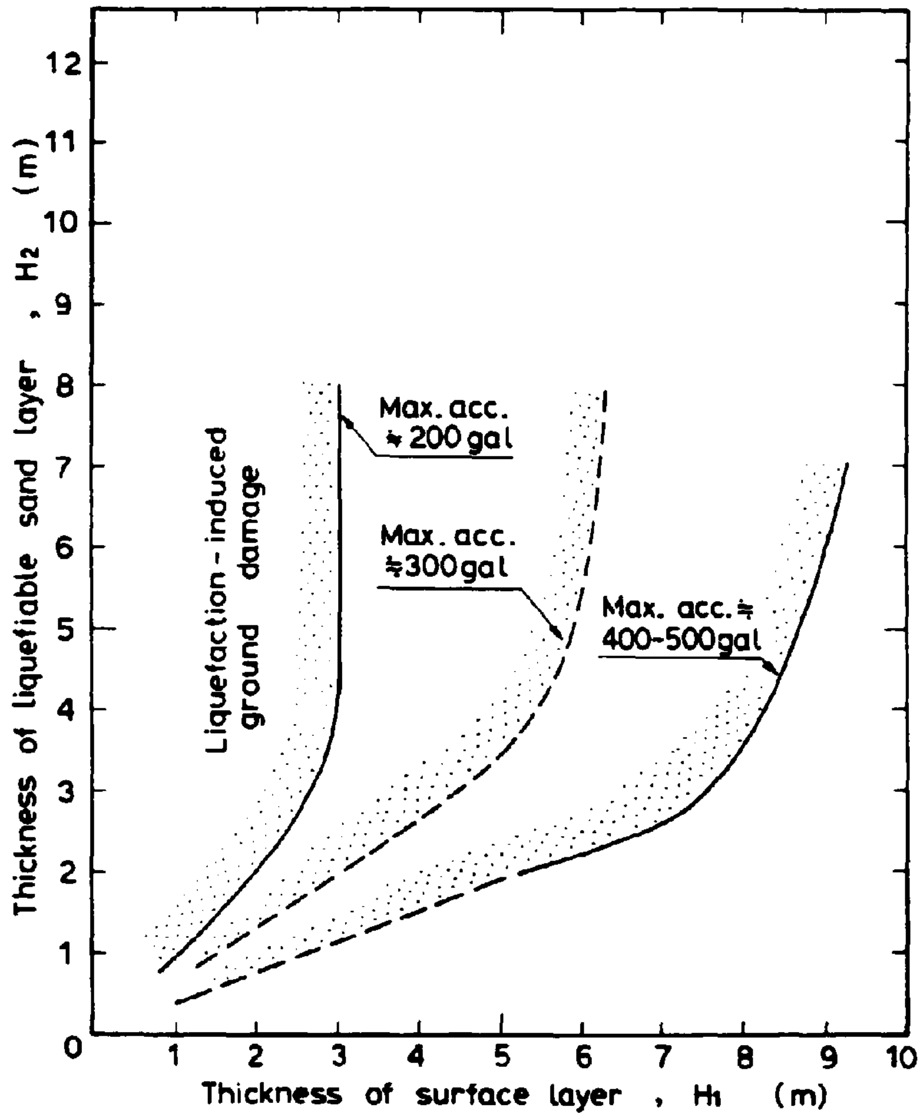

Fig. 3.3 Boundary curves for relationship between liquefaction-induced damage and overburden thickness (from Ishihara, 1985).

3.2.2 Settlement Estimation Procedures based on Laboratory Tests

An obvious approach to estimating the settlement of a liquefiable sand layer is by measuring the behavior of the sand in the laboratory. This method has a long history, with an early attempt made by Silver (Silver and Seed, 1971). Cyclic triaxial tests were run on samples of dry sand with different relative density. Silver noted that compaction-related strain becomes significant for relative densities less than sixty percent. The vertical strain after drained (dry) compaction was then correlated to cyclic shear strain for the different densities of sand. This approach assumes that in the field, all the excess pore water is forced out of the relevant soil layer. However, since many sands at a given site reliquefy, compaction in the real, undrained case is expected to be less complete than for consolidation in the drained state (in addition to differences between dry and saturated behavior).

An early study using cyclic triaxial tests found a good correlation between volumetric strains and excess pore water pressure (Lee and Albaisa, 1974). This relation held for strains less than required to reach liquefaction, and showed that the strains would be less than one percent. Lee reasoned that a well-designed site should only be subjected to 60 — 80 percent of the cyclic stress required to cause liquefaction, so the pre-liquefaction settlements are of interest. Chung and Yokel (1984) noticed the same correlation for resonant column tests. They outlined a method to use the cyclic strain approach and this relationship to estimate settlements.

Tatsuoka et al. (1984) decided to use constant-volume cyclic torsional shear tests to model the in situ conditions in the laboratory. A very elaborate hollow-cylinder apparatus was constructed to avoid affecting natural behavior and insuring one-dimensional consolidation. Tatsuoka found that volumetric strain correlated much better with maximum shear strain than excess pore pressure, with strain increasing for looser sands. These tests, however, were run with dry sand, so any results for conditions approaching liquefaction are obviously meaningless. The authors claim that the measured vertical stress ratio operates the same as the pore pressure ratio, but the mechanism causing collapse of a dry sand is so very different than that causing liquefaction that the results of Tatsuoka's project must be more thoroughly proved.

3.2.3 Settlement Estimation Procedures based on Standard Penetration Tests

As for estimating the liquefaction potential of a soil, the SPT serves as the state-of-the-practice for estimating settlement of sand due to earthquake excitation. The method is presented in a paper by Tokimatsu and Seed (1987). This method starts by developing a relationship between volumetric strain and relative density as a function of shear strain. The curve presented is based primarily on the results of Tatsuoka et al. (1984), just discussed. Tokimatsu claims the results are for strain after liquefaction; however, close reading of the Tatsuoka paper shows that the tests were actually "constant-volume cyclic tests" run on air-pluviated dry sand. Additional data points are calculated from the results of Lee and

Albaisa (1974), although shear strain was not measured when the tests were run. Tokimatsu and Seed note that there is only a small amount of data (14 points) on which to base their conclusion, and allow that the correlation may be in error by ±25 percent.

The authors note that once a soil liquefies, most "historical" factors controlling behavior have little effect, so laboratory problems such as sampling and handling effects become negligible. The relationship derived in the laboratory, shown in Fig. 3.4, is therefore valid for estimating field behavior. The authors also present a relationship between the corrected SPT blow count, $(N_1)_{60}$, and relative density, D_r. This relationship, shown by the parallel horizontal axis of Fig. 3.4, is open to great uncertainty (Schmertmann, 1972).

The last variable needed to use Fig. 3.4 for estimating volumetric strain is the shear strain induced by an earthquake. The relationship between SPT blow count and induced shear strain was introduced in 1985 (Seed et al., 1985), involving the cyclic stress ratio (discussed in Chapter 2). These correlations can all be combined to yield the volumetric strain given the corrected blow count and cyclic stress ratio, which incorporates the nature of the temblor. This relation, for a clean, saturated sand, is shown in Fig. 3.5. The authors note that there is a paucity of field data against which verify their method. A few field points are shown in Fig. 3.5, and indicate some correlation.

The acceptance of this method is essentially a matter of faith in the author's judgement, and a lack of alternatives. The initial relation is based on a very few data points, most of which are from a controversial test. There is also a troubling non-independence of relationships, since both relative density and induced shear strain are estimated from the same SPT value. The relative density and induced shear strain are supposed to be independent pieces of information. However, as Lee pointed out (Lee and Albaisa, 1974), it is only possible to estimate static strains to within 25 percent to 50 percent, so if the infinitely more complicated dynamic problem can be solved with anything close to that degree of accuracy, the solution is very good indeed.

The importance of correlations of material properties and the standard penetration test goes beyond Tokimatsu and Seed. A recent example is the method proposed by Ishihara and Yoshimine (1992) to estimate vertical displacements of sand. The authors derived a set of curves from cyclic simple shear tests relating volumetric strain to relative density and the factor of safety against liquefaction. The values for relative density and factor of safety are derived from correlation with SPT blow count (or CPT penetration resistance). The method can possibly be simplified to a direct relation between penetration resistance and volumatric strain. Settlement is calculated by integration the strains over layer thickness. Tokimatsu and Yoshimine (1992) combine results from cyclic direct shear tests (strains) and double-amplitude cyclic axial shear tests (factor of safety) with no discussion of possible differences between the tests, such as the pooling of free water at the top of double-amplitude specimens.

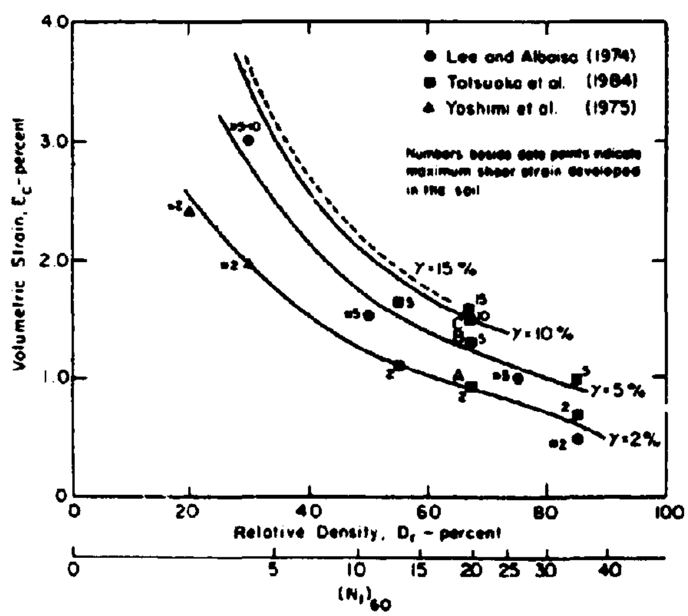

Fig. 3.4 Relationship between volumetric strain, induced shear strain, relative density, and blow count for sands (from Tokimatsu and Seed, 1987).

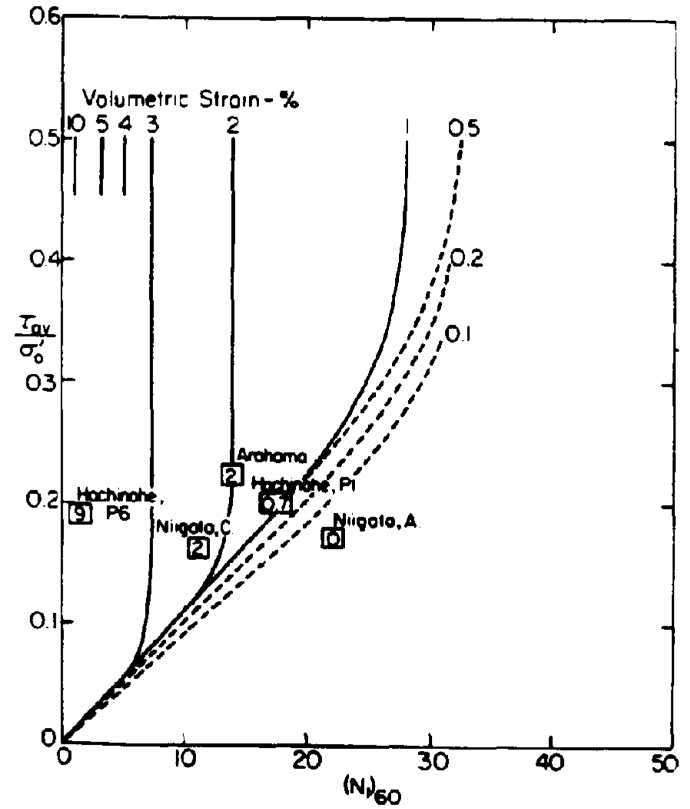

Fig. 3.5 Chart for determination of volumetric strain from blow count and cyclic stress ratio. Field performance from selected temblors is given (from Tokimatsu and Seed, 1987).

Dobry (1989) suggests that the reason the SPT works well for predicting liquefaction-associated phenomena is that there is a good correlation between blow count and the amount of water expelled by a liquefied layer. A dense sand might "liquefy" (i.e. $u=\bar{\sigma}_v$) but there still be little compaction, or water expelled. This will result in the charts correctly predicting a lack of "liquefaction manifestations". This is the behavior expected from cyclic mobility. The relative density and thickness of the liquefied layer both determine how much water can be expelled (and how extreme the manifestations) and therefore are controlling parameters for prediction of settlement.

A recent report (Pease et al., 1992) applied both the Tokimatsu and Seed (1987) and the Ishihara and Yoshimine (1992) methods to actual vertical displacements measured in the San Francisco Marina district after the 1989 Loma Prieta earthquake. Pease chose to use CPT soundings rather than SPT values since he believed that the CPT results were much more sensitive to subtle stratigraphic textures. The authors claim the Ishihara method is slightly more "flexible" but "subject to variations in interpretation by the user." Both methods are very sensitive to input accelerations used. Pease notes that the methods gave good estimates of the actual displacements, with the Tokimatsu method slightly more accurate. They also note that given the relatively thin layering of silt and clay at this particular site, estimates made from SPT data would have been very incorrect (Pease et al., 1992).

3.2.4 Analytical Settlement Estimation Procedures

A "semi-empirical" residual strain method (RSM) has been developed at the Massachusetts Institute of Technology (Stamatopoulos et al., 1991). A study has been undertaken to apply this general method to predicting settlements under and near a tank foundation, which will be discussed here (Bouckovalas et al., 1991). The RSM attempts to take into account the complexity of the dynamic process by incorporating amplitude, rate, and duration of excitation, soil density, permeability, and stress/strain history of the deposit. This method can take into account partial drainage, a problem seldom addressed in the liquefaction literature.

The modelers note a similarity between soil/pore water behavior under creep, and cyclic loading. This similarity leads to modeling the cyclic behavior using the Maxwell fluid paradigm, with the number of loading cycles replacing the time variable of the creep model. The model results in two constitutive equations for strain, due to average and deviatoric stresses, which are solved by the finite element method. The two constitutive equations are coupled through compatibility of soil displacements and pore pressures.

As for any constitutive model, the more thoroughly various aspects of behavior are accounted for, the more input parameters are needed. For the RSM it appears that nine constants and four parameters describing material behavior are needed. The authors claim that these values can be easily derived from common static and dynamic laboratory tests. However, the effect of sampling on in situ properties was not addressed.

The RSM was benchmarked against a centrifuge test of a storage tank (Lambe and Whitman, 1982). The sand was fully instrumented beneath the center and edge of the tank, and in two free field positions. The excess pore pressures in the free field approached the vertical effective stress, indicating that initial reduction of cyclic shear strength took place. Beneath the tank the excess pore pressures stayed well below the level necessary to degrade soil stiffness, especially below the edge of the tank. Beneath the center of the tank there was also indications of partial compaction of the sand during shaking. For all the tests, the final settlement was greater beneath the edge of the tank than under the center, and settlement in the free field was considerably greater than under the tank.

The theoretical predictions consisted of estimates of pore pressure and actual displacements. Calculated and measured pore pressure estimates in the free field match well for completely undrained conditions, but under the structure the model indicated that partial drainage took place even during the shaking. The calculated displacements were accurate for beneath the structure, but half that measured in the free field. The settlement accumulation rates matched very well in all locations. The results are promising, and indicate that this method is a useful tool for increasing conceptual understanding. At this time the method falls short as a useful prediction tool since the proper input values of all the parameters are not clear, and calibration of variables with known results in essence is a very elaborate and involved curve fitting procedure.

3.2.5 Compaction of a Liquefied Soil Column

At the instant of liquefaction, the media consists of solid particles suspended in a viscous fluid. The problem of estimating volumetric strain of this liquefied mass is actually that of modeling the solidification and consolidation of the suspended sand particles. This problem has been set out in a very insightful paper thirty-two years ago (Florin and Ivanov, 1961), with preliminary equations given. This work combined case histories, large-scale field tests including blasting, and theory. The problem seems to have been completely solved by Scott (1986). Observations of Florin and Ivanov will be discussed, and Scott's solution outlined.

Because of the great number of parameters involved in the liquefaction phenomena, Florin and Ivanov focus on the intensity of dynamic disturbance, in situ stress state, and hydraulic gradient, as the critical parameters needed for estimation. The displacements attendant to liquefaction will be a function of the duration of liquefaction and the viscosity of the liquefied mass. In turn, the duration of liquefaction will be affected by the thickness of the sand, permeability, change in the void ratio during compaction, drainage intensity and path, and duration of seismic excitation.

Results from the laboratory and large-scale field tests showed that the liquefaction behavior of a sand is different under shock and vibrational excitement. Shock loading liquefies the whole depth of the layer at once, and the reconsolidation is independent of intensity. For vibrational excitation, as by an earthquake, the liquefaction starts at the top of the layer, which carries the least load, and proceeds progressively downward as the fluidized portion relieves effective stress from the underlying sand grains. The excess pressure then dissipates over time as the grains begin to settle out at the bottom of the layer, with a liquefaction front now progressing from the bottom of the layer to the top. The time-dependant pore pressure behavior for the process of progressive liquefaction and compaction is illustrated in Fig 3.6 for a large laboratory test.

Laboratory tests in the U.S.S.R. showed that any loose cohesionless material will liquefy. Because soils such as gravels have such a high permeability, they stay liquefied for such a short time that there are often no external manifestations such as sand boils or settlements. The efficacy of sand drains and free-draining surcharge as soil improvement techniques is easily calculated by this approach. Calculations and various charts showing the effects of stratified deposits on liquefaction propensity and intensity were presented by Ambraseys and Sarma (1969). Investigations also showed that a small amount of entrained gas can greatly reduce the propensity to liquefy, and reduce displacements once the soil does liquefy. This is due to the gas decreasing the intensity and velocity of the stress waves.

Scott (1986) models this process as two advancing shock fronts. As the sand particles begin to settle out, they form a solidification front which travels upwards from the bottom of the deposit. As the sand drops away from the top of the layer, a front between clear water and suspended sand forms. This settlement interface between sand and water is the shock front moving downwards. Working from the equation for the velocity of the solidification front given by Florin and Ivanov, Scott gives a solution for the interaction of solidification and consolidation. Using the pore pressure distribution known from the solidification solution, Scott adapts Gibson's theory for consolidation of a sedimented clay layer to model the downward-moving consolidation process.

Deformation is then the combination of the settlement from sedimentation of the grains and the consolidation of the settling layer. The settlement caused by sedimentation, which proceeds linearly in time, will be much larger than that caused by consolidation. The solution is based on a certain amount of idealization of the problem — the sand grains all simultaneously reach terminal velocity equal to the original hydraulic conductivity, estimation of a relevant coefficient of consolidation, and assuming relatively small total settlement.

Verification of Scott's theory requires a large enough test such that the compressibility of the sand grains becomes important, and monitoring pore pressures until they come to complete equilibrium. Unfortunately, shaking table tests do not usually meet these requirements. Centrifuge tests are quite suitable, but Scott notes that at the time of his paper being written, only the report of [illegible]be (1981) presents enough detail to allow an analysis. Figure 3.7 shows the fit between the actual reduction of pore pressure compared to the estimated values, shown by circles. The correspondence is very good. The fine straight line shows the results calculated assuming no grain compressibility. The estimated displacements were also in good correspondence to those measured.

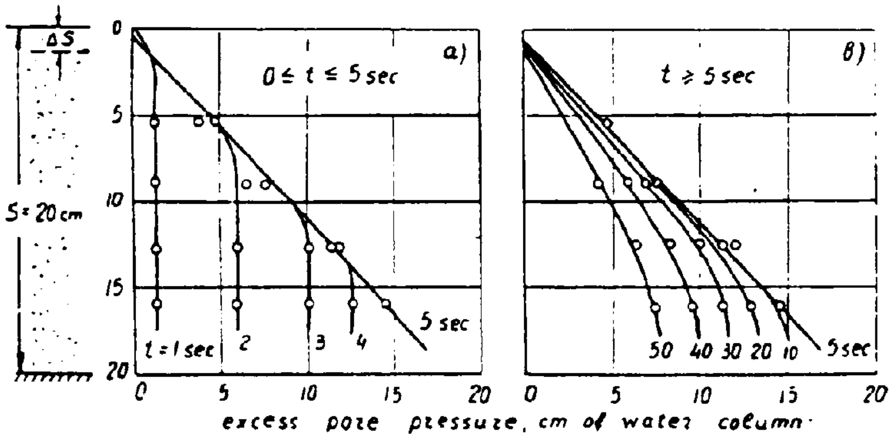

Fig. 3.6 Distribution of excess pore water pressure through time in a sand stratum excited by a vibratory source (from Florin and Ivanov, 1961).

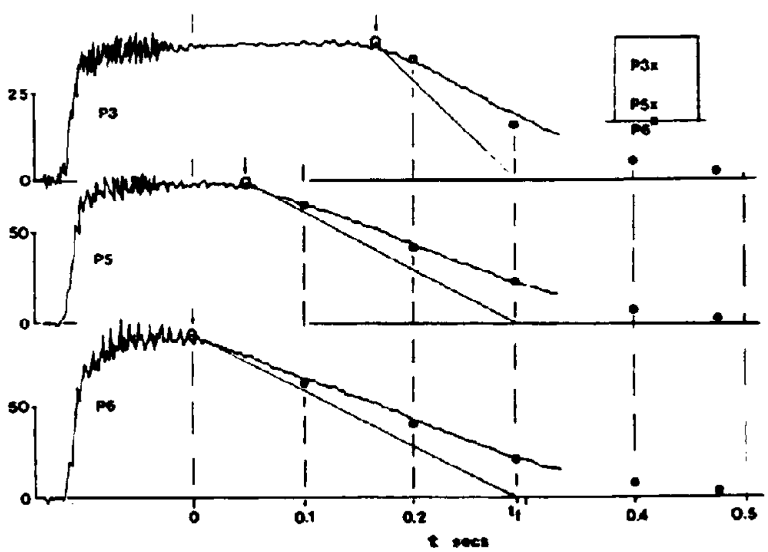

Fig. 3.7 Excess pore water pressure versus time at locations shown. Circles are calculated results, solid line is experimental data (from Scott, 1986).

The approach of Scott, and Florin and Ivanov, is attractive due to its simplicity and the small number of parameters needed. It is unfortunate that more case studies were not analyzed, and that no one since 1986 has continued the work. This might be due to the difficulty in defining the soil properties needed to undertake the analysis. There is a problem in confidently measuring properties for a variable real-life material. A true test of the method might be to apply it to field data from the Wildlife site or Lotung, or the large-scale field tests run by Florin and Ivanov.

3.3 Estimating Horizontal Deformations

3.3.1 A Framework of Understanding

The major hazard of liquefied soils is the occurrence of horizontal displacements. Almost all the alarming photographs of wide-spread destruction from an earthquake are for damage done by large horizontal movements of the soil surface (e.g. NRC, 1985; Steinbrugge, 1982). The movement of entire neighborhoods during the 1964 Alaska earthquake is a startling example of large-scale horizontal displacements. A body of work has developed in the attempt to estimate horizontal ground displacements, which will now be examined.

In order for ground displacement to take place, a driving force must be present. If a loose sand actually liquefies, the driving shear stresses are "static stresses required for static equilibrium....correspond(ing) to those one would calculate in a stability analysis." (Castro, 1987). This situation involves three general geometries, illustrated in Fig. 3.8. Liquefaction of a sloping stratigraphy leads to a general flow failure or slope failure, such as at the Lower San Fernando dam in 1971. This behavior has been a common occurrence during several Latin American earthquakes. A sloping lower boundary leads to lateral spreads when the sand liquefies and looses shear strength. The lateral displacements of soil towards a river or open cut was illustrated by the large-scale damage done by the 1964 Niigata earthquake. The amount of slope needed to drive the movement is very small — 0.3 to 3 degrees if there is a nearby free face.

Driving shear stress can also be provided by man-made structures such as buildings and earth embankments. A railroad embankment caused large (> 1.5 m) displacements above a liquefied layer in Niigata (Ishihara, 1985). In addition to the driving stresses caused by structural loading, damage to the structure can occur. This includes the tilting and sinking of surface structures as well as the floating of buoyant submerged structures such as tanks and pipelines. Well known examples of these behaviors occurred in Niigata. A very through analysis of this problem, based on several shake table tests, was reported by Ishihara and Takeuchi (1991). The paper is especially useful for its qualitative description and conceptual evaluation.

In the case of cyclic mobility, where the soil maintains some shear strength and the static forces are less than the steady-state strength, the earthquake excitation itself drives the soil displacement. This is a stable situation, rather than one leading to catastrophic failure.

A mechanistic solution is generally used for this situation, although constitutive models are being written to solve for the displacements directly. Whether soil liquefies or undergoes cyclic mobility, the displacements will be a function of the steady-state strength of the soil, although the shear strength of a loose, liquefied sand might be better described as viscosity.

3.3.2 Simple Empirical Approaches to Estimating Displacements

The most straight-forward approach to estimating possible horizontal displacements due to liquefaction is to compile a database of known displacements, and correlate the displacements with measurable parameters. A difficulty is choosing parameters, since so many different geotechnical, topographical, and seismological variables affect the results. The accurate measurement and compilation of in situ displacements, and the relevant parameters, is also a formidable task.

The first important study of this kind was done by Hamada (Hamada et al., 1987, 1986). Detailed mapping was done in Noshiro City after the 1983 Nihonkai-Chubu temblor. The displacements were scaled from before-and-after air photographs. Over 2,000 points were analyzed, with maximum lateral displacements of three to five meters noted. Hamada combined this data with information from Niigata (1964) and San Fernando (1971) for a regression analysis. The parameters chosen as relevant were thickness of the liquefied layer and the steepest slope angle of either the bottom of the liquefied layer or ground surface, in percent. Soil properties are represented by the thickness of liquefiable layer, which is calculated by the Japanese Bridge Code procedure. The result of the regression is given by Eq. 3.1

$$D = 0.75 \sqrt[2]{H} \sqrt[3]{\theta} \tag{3.1}$$

where D = displacement
H = thickness of liquefied soil layer
θ = greater of the slope of bottom layer or ground surface (in percent).

The fit between the actual data and the regression equation is shown in Fig. 3.9, with the range one-half to two indicated by the dashed lines. Finn (1988) points out that the database is biased by several factors. There is strong influence from Noshiro City, which had small slopes and displacements, and the large displacements from Niigata. The estimates of slope are unreliable at areas of rapid change, such as near river banks. Finn recommends separating extreme topography into specific studies. The included temblors were all close in magnitude, and the soils very similar, so generality is not present in Hamada's equation.

Other researchers looked at the same case studies as Hamada, and derived other regression equations based on what they though were the controlling parameters. Miyajima et al. (1988) focused on the permanent ground displacement after the 1983 Nihonkai-Chubu earthquake, using the raw data from Hamada et al. (1986). Miyajima broke the areas

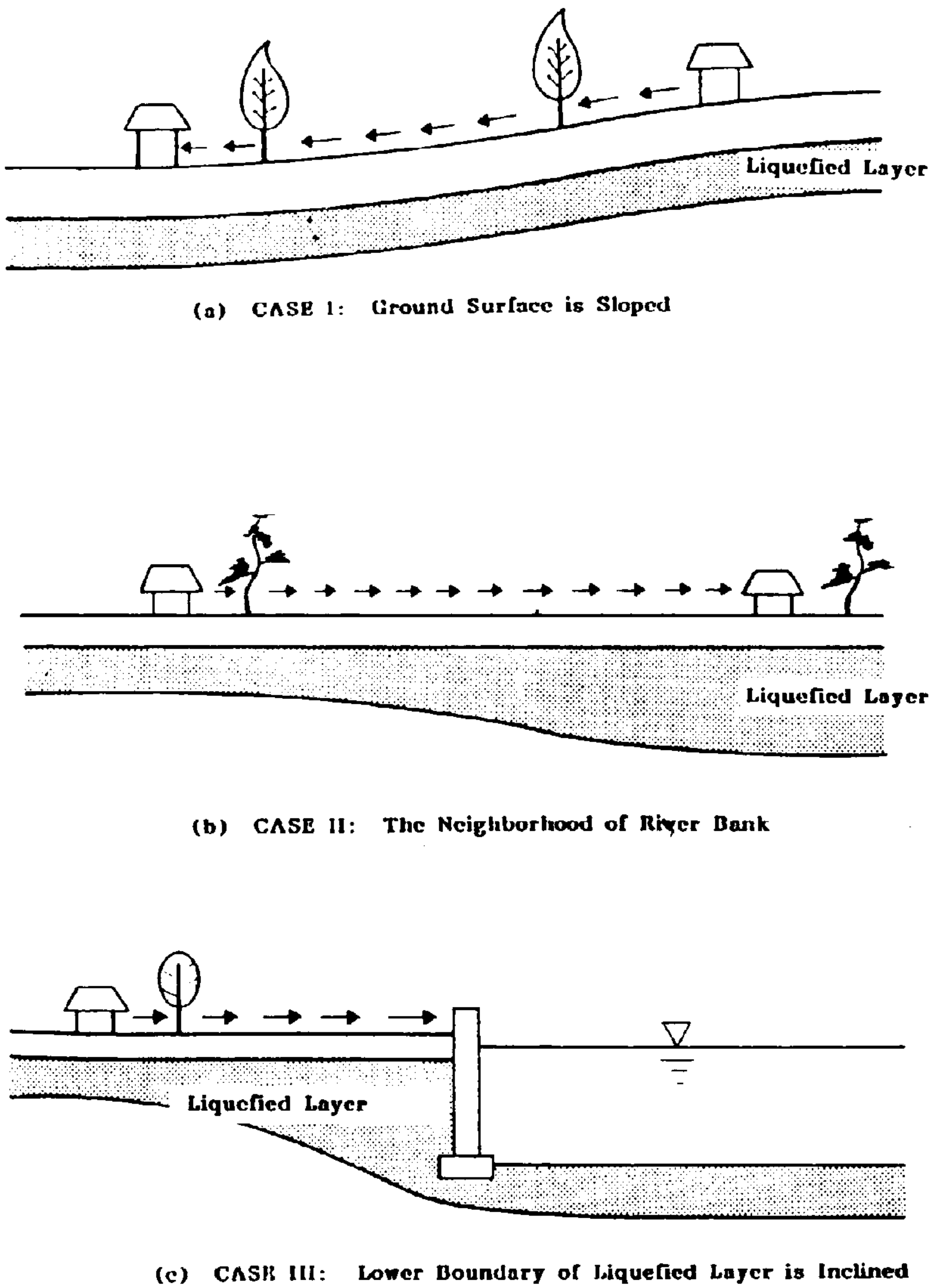

Fig. 3.8 Topographical and soil conditions for liquefaction-induced permanent ground displacement (from Hamada, 1991).

showing displacement into three categories: (1) non-liquefied areas, (2) liquefied areas without sand boils, and (3) liquefied areas with sand boils. These three categories relate to aspects of the buildup and dissipation of excess pore water pressures (Dobry, 1989).

The correlation between displacement and ground slope for each category was different. The displacement associated with non-liquefied areas was the least (< 1 m), while the areas with sand boils exhibited the greatest movement. The correlations for all three cases are rather poor, with a very large variance. The authors found a "better" correlation between maximum ground displacement and the width of the mobilized zone, which they found to be shaped like a half-sinusoid in the plan view.

From the field data, and from shaking table tests to evaluate the correlation of width of displaced zone with permanent lateral displacement, Miyajima found that "The maximum [displacement] value seems to be directly proportional to the width of the loose sand deposit." There is also a good correlation with the slope of the ground. Note, however, that this approach is inherently flawed — there is no way that this analysis can lead to a predictive equation since the width of the mobilized zone is an a posteriori variable. This fallacy is carried further in Miyajima et al. (1991), where the duration of liquefaction is added as a controlling variable, another a posteriori parameter.

Jian and Lianyang (1991) apply a rational technique towards the choosing of physical parameters by which to estimate displacement. The authors use a Fibonacci search method of optimization. Five factors were chosen to represent the in situ condition and excitation. They are (1) earthquake magnitude, (2) distance from earthquake hypocenter, (3) depth of water table, (4) depth of sand, and (5) average SPT blow count. A database of forty data points was compiled from the literature based on thirteen different earthquakes, where liquefaction did or did not occur.

The system was trained on twenty randomly selected data points, which included calculating weightings for the various factors. When the system was tested on the remaining twenty points, the authors were able to predict the actual displacement quite well, claiming a correct rate of 95 percent. The analysis shows that the SPT blow count and temblor moment magnitude were the most important factors, with depth to the water table the least important. The results are promising, and as the authors point out, more field data is needed to strengthen the analysis. Further detail about the analysis and approach is also needed.

3.3.3 Advanced Empirical Approaches to Estimating Displacements

Through his many years at the U.S. Geological Survey, Youd has worked towards a rational-empirical method for estimating permanent displacements from liquefaction of soils. Through a review of case histories, Youd appraised the amount of damage structures undergo for different amount of ground displacement (Youd, 1976). The approximated

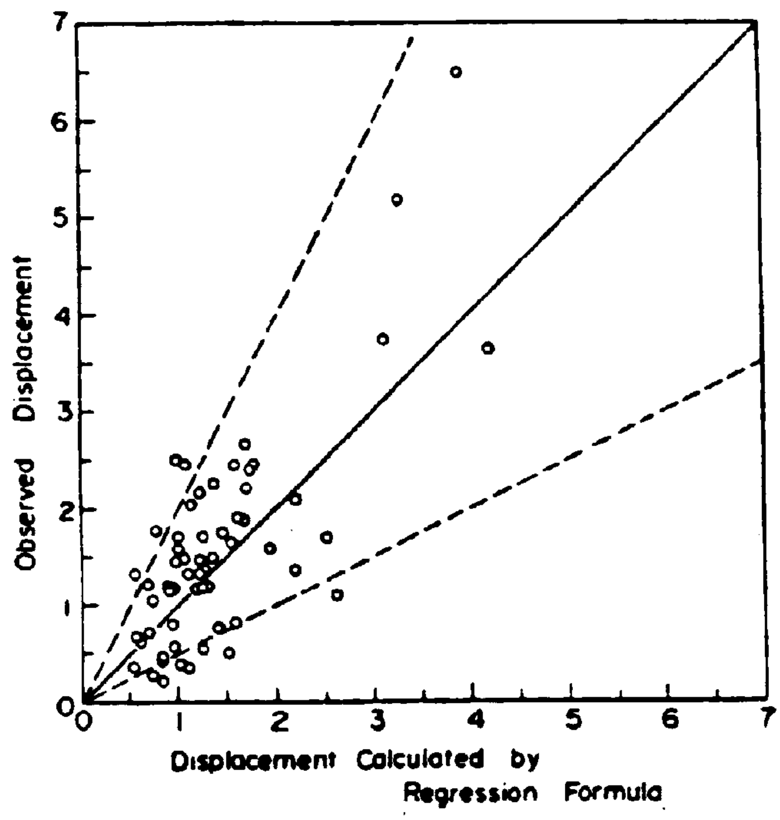

Fig. 3.9 Comparison between actual permanent horizontal displacements and those calculated by Hamada (from Hamada et al. 1986).

ranges are shown in Table 3.3. The damage also varies with type of construction and type of motion, i.e. extension is more damaging than compression.

The severity or damage potential of the displacement can be expressed by the liquefaction potential, S (Youd and Perkins, 1987). S is defined as the amount of permanent ground displacement in millimeters divided by 25 (i.e. maximum displacement in inches). The relation between displacement, liquefaction severity, and damage is shown in Table 3.3. Youd lists fifteen different factors that affect the severity of liquefaction-induced displacement, covering seismological, geological, topographical, and geotechnical factors.

S will vary throughout a region which undergoes liquefaction, so a metric was formulated to give the general severity of ground displacement over a region, called the Liquefaction Severity Index, LSI. Avoiding the overgeneralization of Hamada et al. (1986, see Finn, 1988), Youd and Perkins insist that the LSI be evaluated for distinct topographic and geologic environments. In general, the LSI was derived for "wide active flood plains, deltas, or other areas of gently-sloping late Holocene fluvial deposits." (Youd and Perkins, 1987). LSI ranges from one to one hundred, with anomalously large displacements given the maximum value of one hundred. Also note that the LSI is the ***maximum*** severity in an area, with other lesser displacements also present. It is a conservative, limit estimate.

Displacements and parameters associated with six earthquakes in the western United States and Alaska were analyzed. Youd's analyses showed that most of the geological, topographical, and geotechnical factors canceled out, since the comparisons were done for roughly equivalent regions. For instance, the SPT blow counts ranged from two to ten, and only failures more than ten meters in extent were included. The final regression equation of Youd and Perkins is given by Eq. 3.2:

$$\log(LSI) = -3.491 - 11.86\log(R) + 0.98M_w \tag{3.2}$$

where R = epicentral distance in kilometers
M_w = moment magnitude.

Table 3.3 Estimated damage associated with ground displacements (after Youd, 1976).

DISPLACEMENT, mm	S	DAMAGE
50 - 100	< 5	minor
120 - 600	5 - 20	intermediate
> 760	> 30	major

A graphical presentation of the equation is given in Fig. 3.10, where comparison with estimates using Seed's method are also shown (as the threshold line with $N_1 = 5$). The extensive LSI data base has been utilized as a comparison source by Baziar (1991).

Youd and Perkins argue that the quality of the data available to users does not warrant the inclusion of second order terms. Also, many of the sites analyzed probably corresponds to silty sand rather than clean sand (Baziar, 1991). This equation is only valid for the western U.S., and possibly Alaska. Application to other regions is dangerous because attenuation is higher in the western portion of the country, and the impedance contrast is greater between bedrock and the soil, leading to much higher accelerations in the east.

Recently, work has been done using a multiple linear regression analysis to take into account a great number of important variables in a predictive equation (Bartlett, 1992; Bartlett and Youd, 1993). This analysis examined forty-three detailed factors, from eight different earthquakes, in order to account for seismological, topographical, geological, and geotechnical effects on the permanent displacement of ground due to liquefaction. Bartlett's database comprised 448 horizontal displacement vectors, SPT blow counts from 270 bore holes, and nineteen observations from Ambraseys (1988).

Since the temblors at Niigata and Noshiro City were so similar, the analysis could initially be run for these two site independently of the seismological factors. This analysis showed that the two types of spreading — towards a free face and down gentle slopes — would need separate predictive equations. The free face equation was derived for Niigata, the sloping ground equation for Niigata and Noshiro City, and then the U.S. temblors included to account for the seismic and other site-specific conditions.

The final analysis resulted in two lengthy equations based on six parameters. The free face relation is given by Eq. 3.3

$$\begin{aligned} Log(D_H+0.01) = &-6.57+1.1M_w-0.97Log(R)-0.01R+0.7Log(W) \\ &+0.03T_{15}-0.03F_{15}-D50_{15}-0.01(N_1)_{60FS} \end{aligned} \tag{3.3}$$

and the sloping ground relation is given by Eq. 3.4

$$\begin{aligned} Log(D_H+0.01) = &-6.1_1.1M-Log(R)-0.001R+0.4Log(S) \\ &+0.04T_{15}-0.3F_{15}-1.1D50_{15}+0.01Z. \end{aligned} \tag{3.4}$$

where M = earthquake moment magnitude
- R = epicentral distance to earthquake
- W = Ratio of free face height to distance to free face
- T_{15} = cumulative thickness of saturated sandy layers with $(N_1)_{60}$ less than 15
- F_{15} = average fines content of saturated granular layers included in T_{15}
- $D50_{15}$ = average mean grain size of layers included in T_{15}
- $(N_1)_{60FS}$ = SPT blow count corresponding to lowest factor of safety in the profile
- Z = depth to lowest factor of safety against liquefaction in the soil profile.

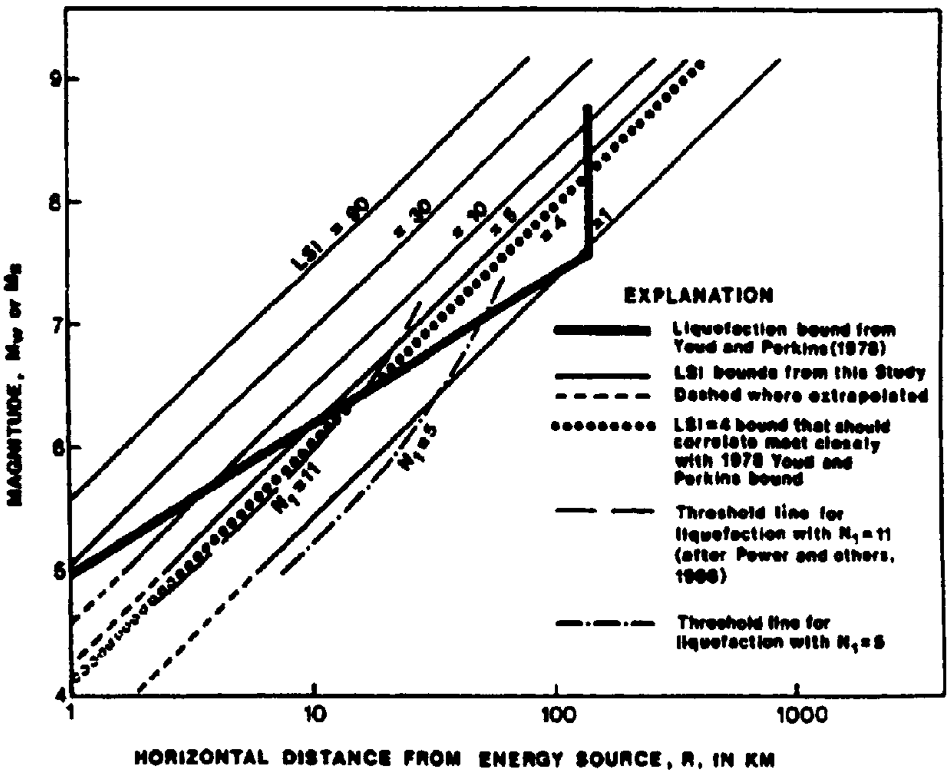

Fig. 3.10 LSI curves on a temblor magnitude versus epicentral distance plot with comparative information from past studies (from Youd and Perkins, 1987).

There are limitations to the use of these formulae, based on the data set used to derive the correlations. For instance, the method is not suitable for the eastern U.S. where the attenuation relation is much different than for the western U.S. and Japan, from where the data came. However, the Bartlett and Youd have made corrections for this limitation in an as yet unpublished NCEER report.

The Bartlett and Youd correlations are a definite improvement over past attempts to estimate liquefaction-caused displacements empirically since they take into account information from all major areas of controlling factors: seismological, topographical, geological, and geotechnical parameters. This inclusion lends a certain degree of intellectual grounding to the empirical relationships. What is now needed is application of the improved relationships to areas not included in the data base in order to fully evaluate their efficacy.

3.4 Estimating Displacements in Stable Situations

3.4.1 Description of the Problem

Liquefaction of a sandy soil is not the only, or most common, behavior due to earthquake excitation. If the sand is moderately dense, it will behave in a dilative manner when cyclically loaded and is strain hardening. As was shown in Fig. 3.2 (also see Chapter 2), the cyclic stress-strain curve for this material is monotonically increasing and the steady-state strength is the maximum strength of the sand. Earthquake loading reduces the strength of the susceptible layer, allowing some small displacement, but the steady-state strength is always greater than the driving shear stress. Makdisi and Seed (1978) say that the accumulated strains will be very small due to the almost elastic behavior of the soil in this condition.

Another form of non-catastrophic "controlled" displacement is termed lateral spreading. In this case, the driving shear stresses alone are less than the steady-state strength of the soil. Good examples of this type of displacement are the damage done by the 1971 San Fernando earthquake to Juvenile Hall (Youd, 1973), and deformations at Hebgen Dam (Sherard et al., 1963). Conditions leading to lateral spreading also involve shallow slopes, or even flat terrain close to an open face such as a river. In all these cases, the added force from the earthquake acceleration is the force over-and-above static stress, that forces motion.

There are two very different approaches towards estimating the possible lateral displacements. The most commonly accepted method is a mechanistic approach first put forth by Newmark (1965), which makes use of the steady-state strength of the liquefaction susceptible sand. This method has been accepted by virtually all members of the geotechnical community (e.g. Baziar et al., 1992; Mabey, 1992; Marcuson et al., 1990; Castro, 1987), and will be described in this major section. The other approach is a full nonlinear finite element approach, and will be described in the next major section.

3.4.2 Newmark's Method

Newmark visualized blocks of solid soil overlaving the liquefied material. The inertial forces induced by the temblor cause the blocks to move on the (sufficiently) weakened soil. The blocks will tend to move towards a free face (e.g. incised river channel) rather than towards the fixed boundary, or graben, in the opposite direction. It can further be assumed that loose soil and debris will fill the fissures formed in the firm material so that the block will move only towards the unsupported (free face) side. Pictorial examples of this scenario are given in Fig. 3.11

The force needed to initiate motion for a given block of soil is calculated just as for the static case of a block on an incline. The horizontal force that is required to just move the block is called the yield acceleration, and is equal to the product of the mass of the block and the mobilized soil shear strength. In reality, the shear strength of the interface is the strength of the weak, potentially liquefied layer, and is the applicable strength used in a plot such as Fig. 3.2. Newmark chose to model the soil as rigid-plastic in order to ease computation. For the case where the sand is very loose and contractive, the shear strength would be the steady-state strength, as indicated in Fig. 3.1, leading to a slope or flow failure.

As long as the horizontal accelerations due to an earthquake are below the yield acceleration, there is no relative movement of any part of the soil profile. When the acceleration is greater than the yield acceleration, the block of soil will move from the time the acceleration exceeds the threshold, until the velocity of the block becomes zero. Note that the motion is only assumed to take place in the down-hill (or free face) direction, since a tremendous amount of acceleration would be needed to move the block either uphill, or against the active earth pressure in the flat ground case (see Fig. 3.11b). Castro notes that displacements greater than one hundred millimeters are usual only for peak accelerations more than five times greater than yield acceleration (Castro, 1987).

An in-depth analysis of the Newmark method is given in Baziar (1991), including a derivation for the case of a fully submerged block excited by sinusoidal acceleration. The essence of the analysis is given by Fig. 3.12. When acceleration exceeds the yield acceleration at t_1, the block begins to move until its velocity returns to zero at time t_3. The amount of displacement is found by twice integrating the acceleration curve for the time of motion (from t_1 to t_3). The cumulative displacement, d_c is the sum of all cycles, as shown in Fig. 3.12.

Baziar derives an equation for estimating the cumulative amount of lateral displacement, given in Eq. 3.5:

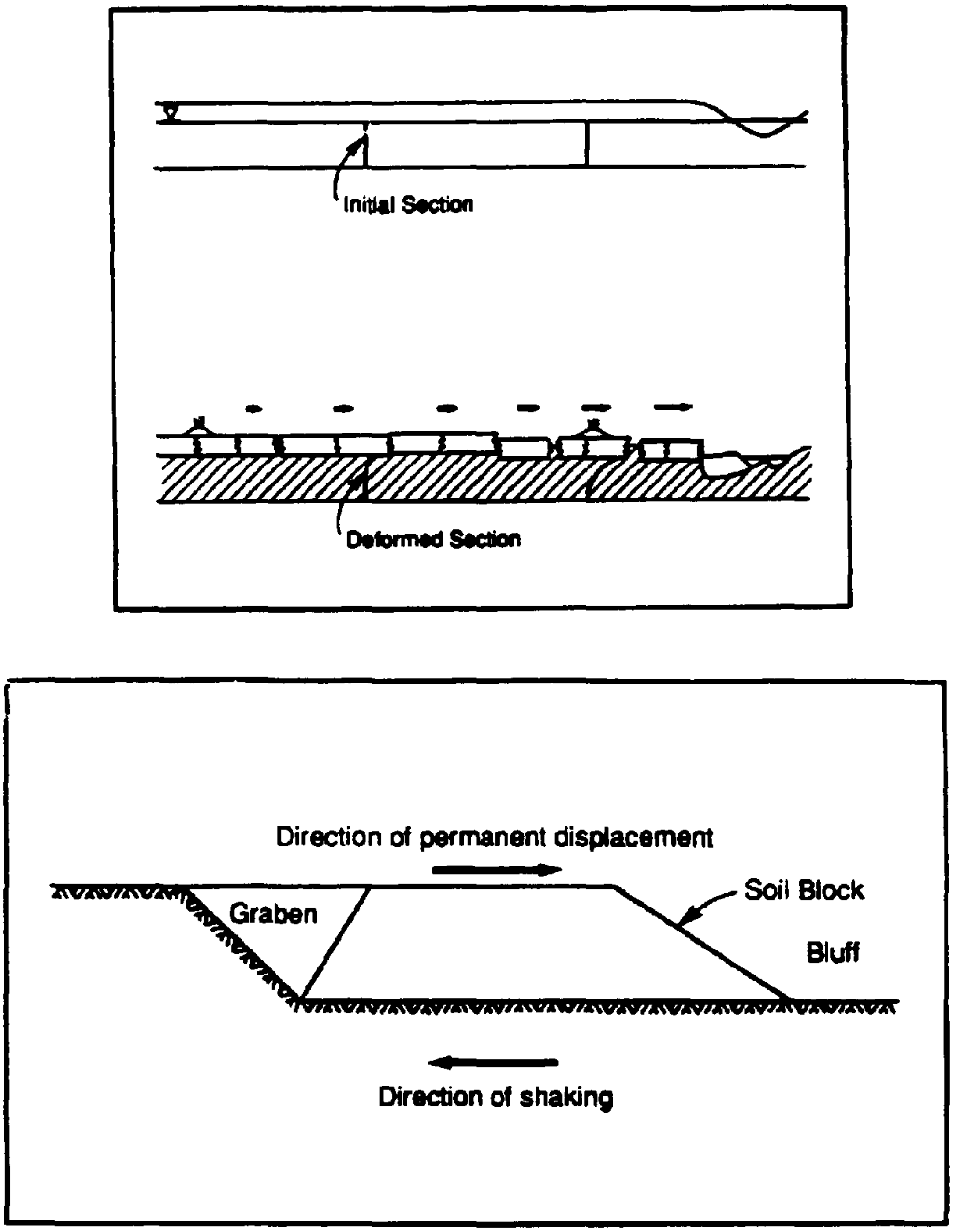

Fig. 3.11 Geometries leading to lateral spread during earthquake strong shaking (from NRC, 1985).

$$d_c = a_p N T^2 \left(\frac{1}{2\pi}\right)^2 f\left(\frac{a_y}{a_p}\right) \tag{3.5}$$

where d_c = total induced lateral displacement
a_p = peak horizontal ground acceleration
N = number of cycles of earthquake acceleration exceeding yield acceleration
T = period of acceleration ($1/f$)
f = frequency of acceleration
a_y = yield acceleration, resistive force of ground to sliding.

It was found that the displacement will be a function of the ratio of yield to peak acceleration, with the solution given by a chart (Baziar, 1991). The same conclusion was reached independently by Yegian et al. (1991). Equation 3.5 also shows that the amount of displacement is inversely proportional to the square of the frequency of excitation. This result appeals to common sense, since if the frequency of acceleration is very fast, there is hardly time for the block to move. This behavior is also consistent with the rigid-plastic idealization chosen by Newmark.

The frequency dependant behavior, hence the applicability of the Newmark method, was tested experimentally and found to hold true only for contractive (loose) sands (Baziar, 1991). The experiments, special cyclic triaxial tests, showed that the frequency of acceleration had no effect on displacement for dilative materials. In addition, the rigid-plastic material model is not followed by dilatant materials, which have no constant-strength plateau. Castro (1987) implies that, for a low number of cycles (stress path below the yield envelope), the sliding block analysis is correct for dilative sands, although the amount of displacement involved is insignificant. It would seem possible to derive an expression for applying a Newmark-type analysis for the dilative case, with permanent displacement being a function of cumulative displacement. In addition, Baziar's modeling of the Newmark situation in an axial cyclic triaxial test might not be strictly accurate, in the sense that the cyclic simple shear test might model the in situ motions more accurately.

In carrying out his extended study, Baziar (1991) ran his series of undrained cyclic triaxial tests on very loose, remolded, layered, pluvially deposited silty sand in an attempt to simulate the actual in situ depositional history. The results of these tests suggested a simple correlation for estimating the steady-state strength of a soil, S_{us}, based solely on vertical effective stress:

$$S_{us} = 0.145\bar{\sigma}_v \tag{3.6}$$

While this relation gives a very convenient estimate of a property difficult to accurately measure (Marcuson et al. 1990), it is somewhat troubling to have a correlation for shear strength which takes no aspect of the soil itself into consideration.

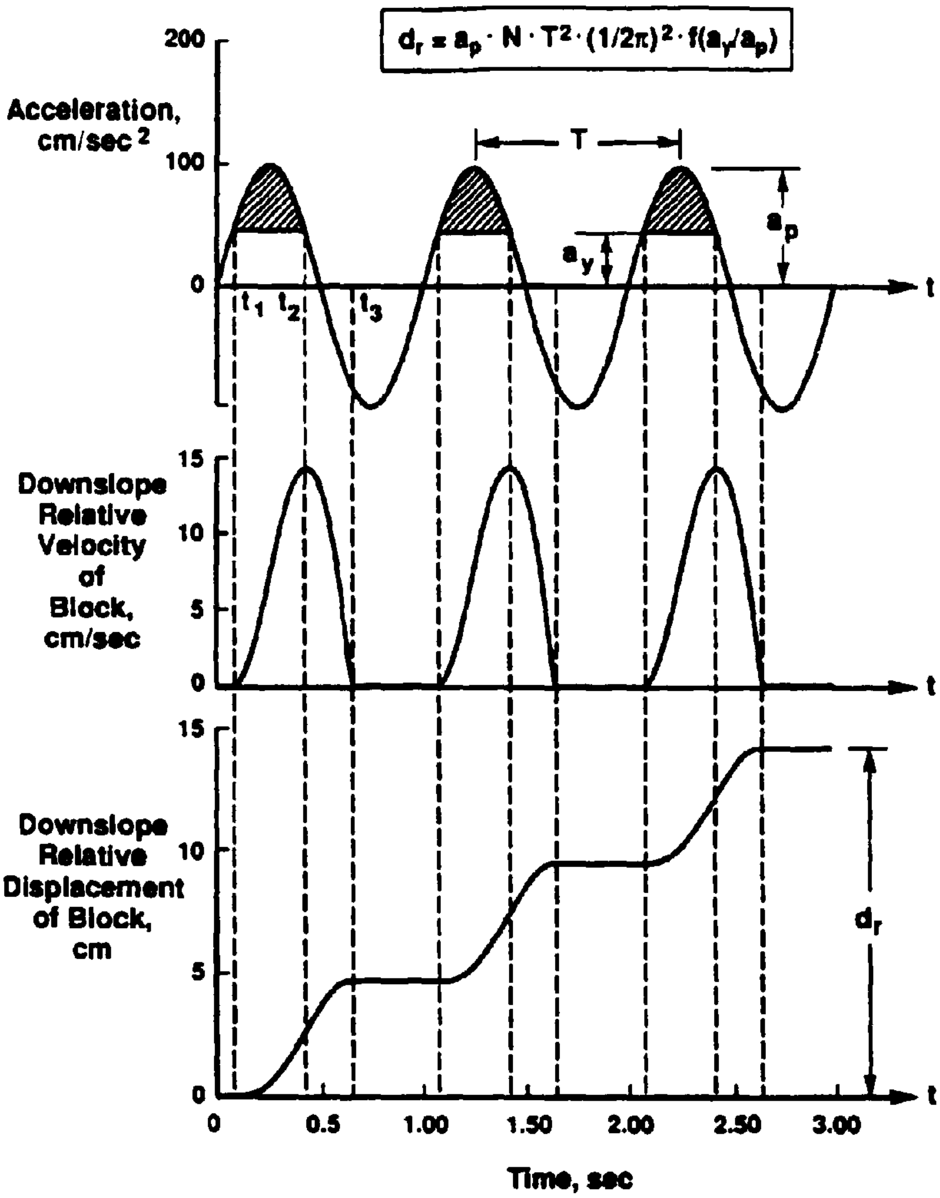

Fig. 3.12 Graphical depiction of the ground displacement calculated by the Newmark method (from Bazier, 1991).

Baziar applied his combined method (Baziar and Dobry, 1991) to the Wildlife Site, Imperial Valley, CA, and to the standard soil conditions used by Youd and Perkins (1987) in developing the LSI. The analytical results compared, but indicated that predictions will not be better than a factor of two or three (i.e., no better than Hamada et al., 1986). Finally, Baziar combined his approach with a regional attenuation relation, in a manner similar to Youd and Perkins (1987). Comparison with the conclusions of Youd and Perkins show the two different approaches giving much the same results.

Mabey (1991) combined the Newmark sliding block analysis with a stochastic ground motion scheme. The method also integrates a correlation between steady-state strength and corrected SPT blow count. This was done by incorporating a large data base of historical lateral spread displacements. Estimates would be predicted by using the lowest $(N_1)_{60}$ value to assess residual strength, and then subjecting the modeled system to a suite of simulated ground motions. Mabey compiled a large variety of such results and formulated a crude regression relation to allow preliminary estimates to be done by hand, although actual sliding block analysis is recommended for any serious work. At this point in time, the accuracy and usefulness of this approach is not known.

A very similar approach to Newmark's method was taken by Yegian et al. (1991). The application was geared towards estimating the permanent displacements of earthen dams and large embankments due to earthquake excitation. Yegian et al. uses actual acceleration histories to estimate displacements, taking into account the magnitude of the temblor in the predictive model. Error theory is then used to model the uncertainties involved in the prediction, resulting in a computer code that "provides the probability that the permanent deformation of a critical sliding mass will exceed a specified value." The probabilistic approach is especially useful for planners, but it is unclear whether the accuracy of the resulting predictions are good enough to warrant the effort.

Makdisi and Seed (1978) customized the application of Newmark's method to the evaluation of earthquake-induced deformations of earthen dams. The concept's assumptions remain the same as Newmark's, but a rational approach to estimating the actual embankment accelerations due to the shaking was incorporated. The yield acceleration — that needed to just force the embankment to move — was calculated as a function of the geometry of the soil "block" and cyclic strength of the weak layer. A two-dimensional finite element analysis utilizing an equivalent linear strain-dependant soil model was used to calculate the earthquake-induced average accelerations that a given block would be subjected to. This step rationalized the application of the sliding-block procedure to large embankment that will have a dynamic response of their own, different from the free field. The final calculation of displacement was done with the double-integration method described above, and illustrated in Fig. 3.12.

A recent improvement to the Newmark sliding block method was proposed to incorporate a more thorough description of the post-liquefaction stress-strain behavior of sand (Byrne et al., 1992; Byrne, 1991). Byrne claims that the rigid-plastic model used by

Newmark (1965) "...cannot in its present form account for the large strains and displacements that occur within the zone of liquefaction." Byrne bases his analysis on the fact that the post-liquefaction stress-strain response of a loose sand is not rigid-plastic, but rather strain hardening up to a limiting strain, whereupon the sand takes on its steady-state strength.

Byrne proposes a soil model where the stiffness of the soil varies linearly until limiting strain. However, Byrne (1991) uses the same variable (SPT blow count) to calculate the supposedly independently measured quantities of stress and strain (which he uses to calculate his stiffness, and displacements), residual strength, and limit displacement. The method itself appears to be an improvement on Newmarks's approach by accounting for actual soil behavior, and N-degree of freedom systems. However, due to lack of the necessary input data, the examples given become empirical correlations based on the corrected SPT blow count, and the method looses its theoretical uniqueness.

3.4.3 An Attempt at Direct Calculation

An attempt has also been made to derive a simplified analytical expression for lateral spread based on a minimum energy principle (Tokida et al., 1992; Towhata et al., 1991, 1989). The work is based on shake table tests conducted to determine the effect of the length of slope on down-slope flow due to liquefaction. Due to interaction with the end of the tank, specimens with shorter slopes showed less strain. It was also found that a dense overburden helps retard liquefaction.

For the analytical model, the soil is expected to deform with a quarter-sinusoid distribution -- zero at bottom and maximum at the top of the layer. The soil will flow with constant volume, and as a liquid governed by the hydraulic gradient. The upper confining layer acts as a horizontal bar with Young's modulus E. A value for displacement at the top of the liquefied stratum is derived by minimizing the potential energy of the ground using the variational principle. This is usually a complicated computation involving the entire strain tensor, but has been simplified by the authors.

The solution to the problem is a very long and involved formula. The formula is comprised mostly of geometric factors describing the slopes and thicknesses of the firm sub-base, liquefied layer, and unsaturated cover layer. The only material parameters involved are the unit weights of the liquefied layer and overburden, and the Young's modulus of the overburden. In fact, the material properties only serve as arbitrary constants, since constant, arbitrary values are assigned to them when the analysis is applied to case histories. This observation is borne out by the simplified practical method suggested in the later paper (Tokida et al., 1993), where displacements are estimated by the following regression equation derived from shake table tests

$$D = a \cdot L^{b} \cdot H^{c} \cdot T^{d} \cdot \theta_{s}^{e} \qquad (3.7)$$

where L = length of possible flow area
H = average thickness of liquefied layer
T = average thickness of the overburden
θ_s = slope of ground surface in percent
a — e = regression coefficients different for center and upper slope segments.

The results of test analyses on twelve sites with lateral spread in Niigata and Noshiro City indicate that this method matches field data well. It should be noted, however, that the length of slide was known a posteriori in these cases, and therefor the method will not work in a predictive mode.

3.4.4 Summary and Illustrative Case Study

A variety of mechanistic and empirical approaches to estimating the extent of lateral spreading induced by earthquake excitation of saturated sands have been presented. It is obvious that no method in fact models the actual physical behavior of the soil. This is the case because there is at present no generally agreed upon, detailed, understanding of what happens after liquefaction, and no method to measure the relevant parameters even if there was agreement.

While scientifically unsatisfying, the methods are useful and surprisingly acceptable for practical purposes. Extremely simple methods, such as Hamada et al. (1986), give results to within a factor of two or three. Considering the complexity of the dynamic processes at work, and the current inability to measure important factors without affecting the resultant value, estimation to within a factor of three is very good. It is even acceptable for static analysis of sand displacements. It is hoped that the work of Bartlett (1992) will improve on the state of current predictive ability, since his relations take into account all general aspects controlling the displacement behavior.

A possible area of future improvement is the estimation of residual, or steady-state strength. Both the empirical SPT-based approach championed by Seed, and the laboratory-based approach pushed by Castro, are based on making very accurate guesses of the in situ void ratio of a sand (Marcuson et al., 1990). They point out

> ...engineers now have available to them two methods of estimating the strength of a liquefied soil: the steady-state strength approach developed by the GEI group [Castro et al.] and the empirical correlation of residual strength with SPT blow counts developed by Seed and his coworkers. The steady-state approach has sound theoretical and experimental foundations, but does have the inherent problem that in most soils the slope of the steady-state line is small. Thus, achieving the requisite accuracy in many cases requires meticulous and expert work to obtain *undisturbed* (italics added) samples of the best quality. Changes in void ratio

of the samples must be precisely and accurately controlled, or else in situ void ratios must be accurately determined by some other means. The use of SPT correlations has the attraction of being relatively inexpensive, so that it is feasible to do large number of tests in order to characterize the variability of materials such as those found in hydraulic fill deposits. The method is purely empirical, with no theoretical foundation, and the data base supporting it is so far not very large. It also involves substantial corrections. It should also be borne in mind that SPT tests for any kind of seismic safety evaluation should be carried out with a degree of care that is often not achieved in ordinary foundation investigations.

The great importance of determining in situ void ratio is the reason for the emphasis on the development of the down-hole nuclear density gage (see Chapter 2). This device could possibly allow accurate measurement of actual, undisturbed, in situ density for a large volumetric area.

At this point, it is beneficial to present some details from a case study illustrating some of the concepts that have been discussed above. Egan and Wang (1991) gave a summary of some of the liquefaction-related ground deformations at Treasure Island, CA, due to the 1989 Loma Prieta earthquake. Treasure Island is a small man-made island in the San Francisco Bay. It was constructed in the 1930's by pumping hydraulic fill behind a set of rock dikes set on the Bay bottom.

Egan and Wang (1991) reported large-scale lateral spreading, from 120 to 300 mm, took place over areas as large as 170 m. Witnesses reported that all displacements took place during the shaking, and that no visible deformations appeared after the seismic excitation stopped (Egan and Wang, 1991). Based on analyses after Newmark (1965) and Makdisi and Seed (1978), back calculations indicated that the undrained residual cyclic stress was approximately 19 KPa (400 psf), a reasonable value.

The effects on lifelines was considerable. Forty-four utility line breaks were reported on Treasure Island due to the temblor. Many of the pipeline breaks were due to lateral spread, although another batch appeared to be due to differential settlement associated with discrete soil blocks undergoing lateral displacement (see Fig. 3.11a).

Vertical differential settlement was widely observed close to structures. The differential settlement was on the order of 50 to 150 mm. The entire island appeared to subside by almost this amount. Some areas subsided by as much as 0.6 m, corresponding to the locations of the thickest deposits of liquefaction-prone fill (see Dobry, 1989). Application of the techniques of Tokimatsu and Seed (1987) was found to over-estimate the actual settlement by a factor of two. Better correlation was found using a hybrid method in which the authors combined elements of Tokimatsu and Seed, a probabilistic method of estimating liquefaction susceptibility, and the influence of grain size championed by Lee and Albaisa (1974).

3.5 Direct Computation of Post-Liquefaction Deformations - Constitutive Modelling

3.5.1 Introduction

The question at hand is the ability to model how a soil system will behave after the saturated sand layer changes from a particulate solid into a liquid. The interest is not in the displacement of the individual grains in the liquefied layer, or even in the permanent deformations of the liquefied layer itself, but rather of the entire soil column, including the intact layers above. This problem is different than modeling the liquefaction potential problem, or pore water pressure buildup problem, which could address the problem locally and use accepted constitutive models. While there are any number of constitutive codes, from simple to exquisitely complex, which model soil behavior up to the point of outright liquefaction more or less well (e.g. SHAKE, DESRAMOD, TARA, etc.), there are very few direct solutions for post-liquefaction behavior (Martin, 1989).

As recently as September, 1989, Martin was able to state about directly calculating post-liquefaction deformations: "Our ability to model such behavior using mechanistic constitutive relationships expressed in terms of effective stress, is very limited at the present time." Since that time, there has been progress, some of which will be reported here. The models to be discussed are for the large post-liquefaction permanent displacements rather than for the residual cyclic strains incurred during cyclic loading. A very thorough review on that topic can be found in a paper by Finn (1988).

3.5.2 Constitutive Models Using the Finite Element Method

Once the soil liquefies, the modeler is essentially describing a completely different material than seconds before, and new constitutive relations must be found. One important difference is that the material becomes strain hardening with the induced large displacements, and therefore can transmit a significant shear force into the overlying material (Byrne, 1991; Martin, 1989). The common approach has been to formulate a set of constitutive equations believed to account for the coupled grain-pore water behavior, and solve them using the finite element method.

Zienkiewicz et al. (1991) use a generalized plasticity theory to represent the cyclic behavior of granular soils. This theory assumes that soil behavior depends on load increment, but is independent of loading rate. The liquefaction of a granular soil implies that the plastic flow is non-associative, with plastic strain accumulating during unloading of the material. The model needs two elastic constants for the material and seven other parameters (describing the plastic behavior) to completely characterize the soil.

Zienkiewicz et al. give two examples of how well their method predicted displacement of a laboratory cyclic test. Figure 3.13 compares the results for a cyclic test which completely liquefied, while Fig. 3.14 gives the comparison for displacements due to cyclic mobility. The correspondence to this laboratory test is excellent, although much more evidence is needed before deciding that the model accurately predicts field behavior. Measuring the seven material constants also seems a presently insurmountable task.

Finn (1990) used the nonlinear finite element program TARA-3 (Finn et al., 1986) to analyze post-liquefaction displacements of soils. For this analysis, the steady-state strength of the liquefiable soil is the critical parameter. When some trigger threshold is reached, the program analyzes the soil structure behavior based on the residual strength of the liquefied layer. A constraining differential equation is solved by the program, which leads to progressive deformations until equilibrium is reached with the driving stresses.

The program was applied to two case studies. One was a section of displaced ground in Niigata. The results were mixed, partly because of uncertainty in assigning steady-state strengths. The other was a conceptual study of the Sardis Dam in Mississippi. The analysis allowed various methods of stability remediation to be studied and compared. A sensitivity study of the import of various values of the steady-state strength was also run. The ability to evaluate varied and different solutions and sensitivities is one of the strong points of the direct modeling method.

Keane and Prevost (1991) have attacked the post-liquefaction problem alone, rather than postulating a unified theory that combines all possible behavior of the soil. In particular, they have modeled the behavior of the combined soil-pore water fluid using an Arbitrary Lagrangian-Eulerian kinematic description combining a moving material and an arbitrarily moving mesh. The method can model both compressible and incompressible fluids. Generally, four material parameters are needed: density, dynamic viscosity, fluid bulk modulus, and an "algorithmic parameter". The dynamic viscosity acts as the residual shear strength of the combined soil-pore water fluid. It is not clear how the dynamic behavior of this quantity will be accounted for. The authors state that the work is in its very early stages, and ultimately will be coupled with a non-linear soil model to allow estimation of the entire liquefaction process.

Yoshida (1989) proposes a finite element solution for the displacement of the already-liquefied soil. Yoshida's method predicts ground deformation with no accounting for earthquake motion. He claims that the direction and extent of displacement is a function of topography, and not of seismology. "In other words, earthquake load works to trigger liquefaction, but hardly control the magnitude and direction of large ground displacements." This is in direct contradiction to the findings of Youd (Youd and Perkins, 1987; Bartlett, 1992). Details on the numerical scheme are found in the paper.

The method is "tested" by running a parametric study on Case C of Fig. 3.8 — the sloping base - flat top situation. The usefulness of this study is the insight gained as to the behavior of this geometry. It was found that the deformations always start at the center of the studied area, far from the limiting boundary effects, and move outward towards the fixed boundaries. The up-slope surface settles while the down-slope surface rises commensurably. If the top surface was held completely flat at the start, the deformation was only at the very

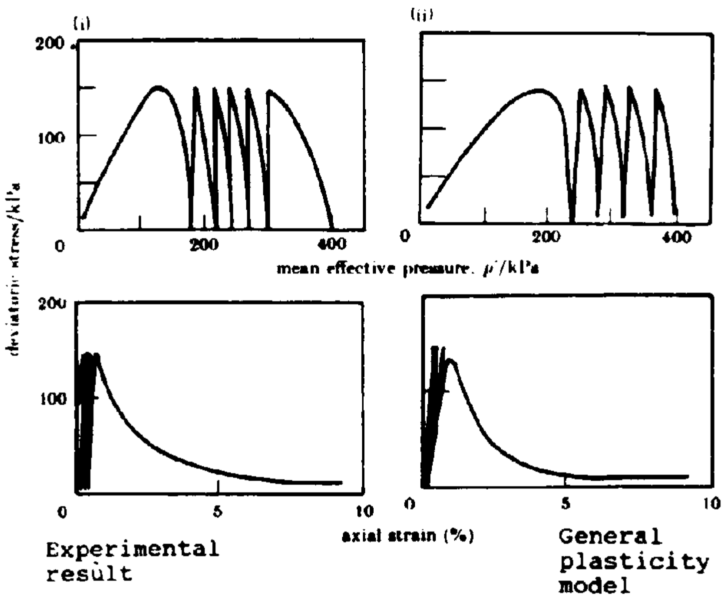

Fig. 3.13 Comparison between experimental strain and pore pressure data and calculated results for a dense Banding sand (from Zienkiewicz et al., 1991).

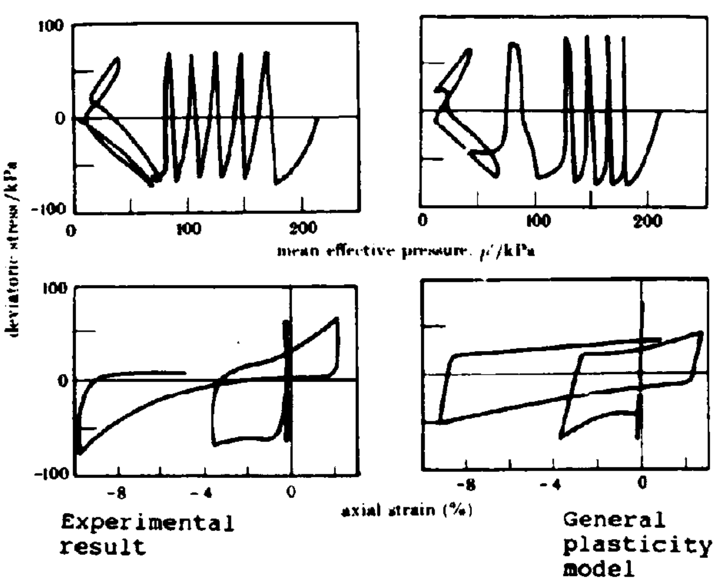

Fig. 3.14 Comparison between experimental strain and pore pressure data and calculated results for a loose Niigata sand (from Zienkiewicz et al., 1991).

against actual or theoretical displacements. It is not possible to be certain whether the results are meaningful or not. In addition, the author notes that the results are very dependant on the size of mesh and boundary conditions. It is hoped that Yoshida will prove his model in the future .

Succarieh et al, (1991) attempt to directly solve the same problem as Newmark (1965), that of a discrete block sliding on a weakened layer. The proposed approach takes into account the deformations of the discrete blocks in addition to their relative sliding. The earthen structure is divided into discrete zones which intimately interact. The behavior of the zone materials are modeled using finite elements, while an interaction relation insures geometric compatibility between zones. Input parameters include quantities such as the zone interface coefficient of friction, Poisson's ratio, viscous damping, and Young's modulus. Further details are found in Succarieh (1990).

The approach is tested against a simple block on a slope, a continuous slope, and the response of the earthen La Villita dam to the 1975 Mexico City earthquake. As would be expected, the two conceptual cases are well modeled. The displacement of the dam is estimated to within a factor of two. At this point, it is not clear what advantage this involved method offers over the much simpler Newmark approach.

3.5.3 Summary of Constitutive Modeling

After this brief review of some of the constitutive models for post-liquefaction deformations, it is seen that much work remains to be done. Only Zienkiewicz et al. propose a complete constitutive model of soil behavior, and this model remains to be validated on actual case histories. More seriously, the need for accurately and repetitively measuring seven uncommon material constants precludes widespread use of the method. Measuring the constants, of which very few people have any physical feel, in the laboratory is difficult enough; in the field it is impossible. The great accuracy offered by a good constitutive model is lost if results from necessarily disturbed field specimens are tested in the laboratory, or if rough correlations from methods such as the SPT or CPT provide the input parameters. These problems are true for all complicated constitutive models, even for problems well understood, and remains one of the major impediments to their wide-spread use.

The models offer a useful research tool since they lead to greater conceptual understanding, and can easily be used for parametric studies to gain insight into relative effects of various parameters. Much work remains to be done before direct calculation of displacements due to liquefaction of sands can be used as a reliable design tool.

3.6 Estimation of Displacements in Relation to Lifelines

3.6.1 Approaches to the Problem

A striking example of damage to a lifeline from large ground displacements is shown in Fig. 4.15, the result of the 1984 Alaskan earthquake. There have been several methods proposed to estimate the ground displacements as they relate specifically to lifelines. One problem unique to lifeline design is the large areal extent of the system, so that some idealization will always be needed to extend detailed, localized knowledge over the larger surrounding area. Because many lifelines, such as pipes, cables, and tunnels, are in the ground rather than on the ground, integrated approaches have been taken. Hamada (1991), for example, calculates a damage probability for buried pipes that incorporates a factor for the probability of damaging displacements.

Suzuki et al. (1989) analyzed the displacement field that occurred around the Niigata rail station during the 1964 Niigata earthquake. They point out that for design questions involving pipelines, the displacements should be broken into fields expressing the axial and flexural deformation of the pipeline in question. Continuation of this work includes the incorporation of vertical displacements into the axial and flexural displacement fields (Suzuki and Masuda, 1991). Regression equations, based on data from Noshiro City and Niigata, were derived to give a relationship between the length over which a displacement was distributed, and the amount of displacement.

M. O'Rourke (O'Rourke and Nordberg, 1991; O'Rourke, 1989) has incorporated permanent ground displacements into models for the behavior of different types and construction of pipelines. Again, the displacements are broken into longitudinal and transverse directions. It was noted that for most pipe geometries, the displacements of the pipe are the same as that of the soil, and that for flexible pipes the damaging amount of displacement is a function of the length over which it acts. These observations show why the reliable estimation of ground displacements and extent are so important to the safe design and operation of lifelines.

Fig. 3.15 Damage to a rail line in Prince William, Alaska due to lateral spreading caused by the 1984 Alaska earthquake.

CHAPTER 4 SUMMARY

4.1 Concluding Comments

4.1.1 State-of-the-Art

This report has shown that the current state-of-the-art of the geotechnical approach to the problem of liquefaction is empirical in nature. Whether estimating the potential for a soil to liquefy or estimating post-liquefaction soil deformations, the accepted and workable methods are completely empirical. This approach is exemplified by the wide acceptance of Seed's method for estimating liquefaction potential. The method was shown to have empirical rather than theoretical grounding, yet it works well. Simple formulae for estimating displacements, such as Hamada's method, are other examples of purely empirical solutions.

There are several reasons why this condition exists. Soils are a very complicated and non-linear material. Material properties can change by an order of magnitude over a very short distance. They were placed erratically by nature when deposited, and then transformed through time in a very non-uniform manner. In addition, it is virtually impossible to measure in situ material properties of a soil without the very act of measuring contaminating the results. All these factors serves to make firm constitutive modeling of these materials virtually impossible. An empirical approach is the alternative.

A second factor is historical. Civil engineering design has traditionally been characterized as "not an experimental science." There has traditionally been some resistance against applying first principles of physics to build a sound physical theory, often because the goal of the designer is not understanding material science but to build a safe, useful structure in a cost-effective manner.

The use of geophysical techniques, especially spectral analysis of surface waves, is an advance of the state-of-the-art. Dobry's strain-based theory is another, although it is limited by only explaining behavior up to the onset of excess pore pressure generation. Application of Kalman filters to the system identification problem is a large step to rationally dealing with non-stationary signals. While still empirical in nature, Bartlett's regression equation takes into account all the major areas of physical influence on post-liquefaction displacements. The people working on constitutive models are attempting to attack the problem of liquefaction from first principles, but often in a manner that can not be applied to the real world. The problem is acknowledged to be extremely difficult, but progress should be made if the profession, empiricists as well as conceptualists, see as worthwhile the establishment of a strong physical base, and jointly work towards that goal.

4.1.2 Conclusions and Recommendations

The SPT-based method developed by Seed and his associates is the state-of-the-practice around the world. All other SPT methods compare themselves to the results of Seed and his collaborators rather than attempting to match the real world data, so there is little reason to not use Seed, and avoid having to convince others that the correlation between blow count and liquefaction potential is "as good as" Seed. The CPT is a far better tool for delineating stratigraphy, but the correlation between tip resistance and liquefaction potential uses an intermediary step of correlation between the CPT results and the corresponding SPT value. The greatest potential is the use of the CPT to carry other test transducers to depth while yielding a detailed stratigraphy. The seismic CPT and the combination with a nuclear density gage are devices of great potential that should be put to practice.

The SASW method is considered to be the most promising approach, especially for lifeline design problems. The SASW, especially the automated improvement, is ideal for testing large areal expanses. Unlike the SPT and CPT, this method does not disturb the in situ state of the sand — the measuring method does not affect the quantity being measured. Further work must be done to apply the small strain velocity measurements from SASW to estimating large strain soil behavior. The strain based approach to liquefaction potential has the advantage of being solidly based on first principles of physics, although more work needs to be done to extend the results past the onset of excess pore water pressure buildup.

One possible system of the future is seen as an automated SASW array driven by a servo-controlled shaker, combined with a seismic CPT carrying a down-hole nuclear density gage. SPT data will also be taken, along with soils samples to actual determine soil texture. The SASW system itself can be mobilized for a net cost of about $30,000.

The dependance of liquefaction potential on soil stiffness and threshold strain implies a simple soil improvement method. Stiffness and γ_t can both be improved by inducing subtle cementation. This can be done by flooding sands with a very dilute cement or chemical mixture. Even muddy water might make a substantial improvement. This method would be very inexpensive and not disrupt the local flow regime to anywhere near the degree that grouting would. Also, electrical methods similar to sintering might be used to "weld" the grains together. The workability and utility of these methods in the field is unproven.

The obvious conclusion to be reached is that the only useful techniques currently available for estimating post-liquefaction deformations are the purely empirical correlations. This is true for both vertical and horizontal displacements, although the work done by Scott (1986) hopefully will lead to a predictive technique. It is difficult to choose one empirical technique over another, since most seemed to give results correct to within a factor of four. It is believed that the regression approach of Bartlett (1992) is more satisfying since it takes into account all the major physical areas that control displacement. The method proposed by Tokimatsu and Seed (1987) to estimate cyclic settlements is not based on theory but has been widely used and found to give results within an order of magnitude. Input data is inexpensive, and is this method is the most likely to be employed.

Problems that remain to be solved include the accurate measurement of in situ residual strength of the liquefied sand, and actual in situ relative density. It is disappointing that at present there is not enough understanding of the liquefaction problem to lead to a good theory based on first principles. It is hoped that study of the problem in the field will soon clear our clouded vision.

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