NUREG/CR-3380

Current Methodologies for Assessing Seismically Induced Settlements in Soil

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Prepared for
U.S. Nuclear Regulatory Commission

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NUREG/CR-3380
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Manuscript Completed: June 1983
Date Published: August 1983

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Prepared for
Division of Health, Siting and Waste Management
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U.S. Nuclear Regulatory Commission
Washington, D.C. 20555
NRC FIN B7002
Under Agreement No. RES-79-115

ABSTRACT

Earthquake-induced surface settlements have ranged from 0.7 to 10 percent of layer thickness for the relatively few incidences where reliable estimates have been made of settlement magnitudes and soil conditions. Standard penetration test results obtained for pre-earthquake and postearthquake conditions in Japan show that relative densities have changed from 188 percent increase to 44 percent decrease. At present, there are no verified methods of seismic settlement analysis. However, there are current methods of analysis ranging from empirical to fully theoretical, which take into account a few to all of the major variables affecting seismically induced settlement behavior. This report reviews pertinent current knowledge and methodologies related to this subject.

PREFACE

The study covered by this report was performed by the U. S. Army Engineer Waterways Experiment Station (WES) for the Nuclear Regulatory Commission (NRC) under Inter-Agency Agreement RES-79-115, during the period March 1980 to July 1982. The project monitors for the NRC were Dr. Jerry Harbour, and Mr. Leon Beratan, Chief of the Earth Sciences Branch, Office of Research.

The report was prepared by Mr. Richard H. Ledbetter of the Earthquake Engineering and Geophysics Division (EE&GD), Geotechnical Laboratory (GL). General supervision was provided by Dr. W. F. Marcuson III, Acting Chief, EE&GD, and Chief, GL; Dr. Don C. Banks and Mr. C. L. McAnear, Acting Chiefs, GL; and Dr. Paul F. Hadala, Assistant Chief, GL.

Commanders and Directors of WES were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. Fred R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
miles (U. S. statute)	1.609347	metres
pounds (mass) per cubic foot	0.01601846	grams per cubic centimetre
pounds (mass) per square foot	0.04214011	kilograms per square metre

CURRENT METHODOLOGIES FOR ASSESSING SEISMICALLY INDUCED SETTLEMENTS IN SOIL

PART I: INTRODUCTION

1. Earthquake-induced settlements in soils can range from no movement to several feet depending on soil type and conditions and on earthquake characteristics. Differential settlements which are the result of nonhomogeneous soil conditions and/or nonhomogeneous shaking can damage structures and connections between structures such as pipelines and conduits. For critical structures, such as nuclear reactor buildings and related pipe and power lines, such settlements could affect safety. The earthquake-induced shear strains cause volume changes which produce immediate settlements in dry or moist soils. In saturated nonfree-draining soils, settlements take place as a function of time with pore water pressure dissipation. Also, the fact should be recognized that liquefaction and large differential settlements almost always occur together.

2. Seismically induced settlements are influenced by many nonlinear variables such as a continuously changing shear modulus with shear strain, irregular earthquake shear stress loading histories, and continuously changing effective stresses. At present, there are no verified methods of seismic settlement analysis. However, there are current methods of analysis ranging from empirical to fully theoretical, which take into account a few to all of the major variables affecting seismically induced settlement behavior.

3. The purposes of this state-of-the-art report are to (a) define the major factors influencing seismically induced settlements, (b) describe and evaluate current methodologies having the potential for seismic settlement analysis, and (c) recommend the best method or methods for use in analysis and continued research on seismically induced settlements.

PART II: BACKGROUND

Observed Settlements

4. Only a relatively few incidences of ground subsidence due to earthquakes have been documented with information concerning reliable estimates of surface settlement magnitudes, soil types, conditions, etc. The following paragraphs summarize documented settlement occurrences which are more completely described in the cited references.

Field blasting

5. Earthquake-like ground motion can be produced by controlled explosions (Bruce, Lindberg, and Abrahamson 1979). The Russian experiences with the use of explosives for densifying loose saturated sands below the water table are summarized in Ivanov (1967). Sequential charges were detonated and the induced settlements measured. Ivanov reports that settlements occurred in the order of 2 to 3 percent of the layer thickness during the first blast in certain areas that were susceptible to liquefaction.

El Centro earthquake, California, 1940

6. A dike 7 ft* high along a canal bank in Mexico sank completely into the underlying soft foundation soil (Wiegel 1970). The foundation soil may have liquefied, but this fact is not known.

Tohnankai earthquake, Japan, 1944

7. In an area about 100 miles from the epicenter, there was considerable subsidence and damage due to liquefaction. Surface settlement beneath a Buddhist temple was in the order of 15 in. The site consisted of 26 to 52 ft of loose saturated sand, and the water table in 1969 was located at a depth of 6.4 ft below the surface (Kishida 1969).

Fukui earthquake, Japan, 1948

8. A temple settled about 12 in. in an area some 3 miles from the

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is given on page 3.

epicenter. The foundation soil was 20 to 26 ft of loose granular material, and the water table in 1969 was located at a depth of 4 ft below the surface (Kishida 1969).

Chilean earthquake, 1960

9. Flooding and inundation of land due to settlement of the soil occurred near Valdivia. Near Puerto Montt, a highway embankement 4 ft high sank into the unstable foundation material in a swampy area (Duke and Leeds 1963).

Niigata earthquake, Japan, 1964

10. In Niigata, about 35 miles from the epicenter, many structures settled more than 3 ft and were accompanied by severe tilting (Japanese Society of Soil Mechanics and Foundation Engineering 1966). The Niigata airport building settled 3 ft, and differential settlement of several feet occurred between bridge abutments and backfills. Areas of severe damage and settlement are underlain by 40 to 50 ft of saturated sand with the water table close to the surface.

Alaska earthquake, 1964

11. Measured settlement at a cased well at the Homer Split, 160 miles from the epicenter, showed surface settlement within a 468-ft-thick alluvium layer of about 2.5 ft (Grantz, Plafker, and Kachadoorian 1964). Other effects in the same general area were vertical offsets as much as 8 in. In the Portage area, about 4 ft of settlement occurred in the underlying soil. Highway and bridge embankments experienced severe settlements into weak foundations of silts and sands.

Tokachioki earthquake, Japan, 1968

12. In an area about 110 miles from the epicenter, at a plant site in Hachinohe, settlements occurred that varied from 4 to 16 in. The site consists of loose sands to a depth of 66 ft, with the water table at a depth less than 5 ft below the surface (Ohasaki 1970). In an adjacent area, a 16-ft excavation that had been backfilled with loose sand had a surface settlement of about 20 in.

San Fernando earthquake, California, 1971

13. Measurements at a plant located about 5 miles from the epicenter showed surface settlements between 3 and 4 in. The area is

underlain by 0 to 50 ft of silty, sandy, clayey fill over 0 to 10 ft of loose to medium dense granular alluvium, with the water table at a depth about 50 ft below the surface (Lee and Albaisa 1974).

Changes in Relative Density

14. Earthquake-induced settlements will cause a density change in soils, and some pre-earthquake and postearthquake Standard Penetration Test (SPT) results that reflect a change exist in the Japanese literature. The Tokachioki earthquake of 1968 had a main shock Richter magnitude of 7.9 and occurred off the Pacific coast of Northern Japan. At the previously mentioned plant site in the city of Hachinohe, 110 miles from the epicenter, the seismic intensity according to the Japan Meteorological Agency (JMA) was V. Pre-earthquake SPT results existed and postearthquake SPT tests were conducted. These are reported by Ohasaki (1970). Prior to the Tohachioki 1968 earthquake, SPT tests had been conducted for an oil tank site in the sands of Nanachama Beach near Hakodate, which is 170 miles from the epicenter and in the JMA intensity V area. Postearthquake SPT tests were conducted and these data are reported by Kishida (1970).

15. The Niigata earthquake of 1964 had a main shock Gutenberg-Richter magnitude of 7.5 and occurred off the Japan seacoast about 35 miles from Niigata, Japan. In the Niigata area where the JMA intensity was V, some pre-earthquake and postearthquake SPT results for the sandy soils exist. These SPT results are reported in the journal of the Japanese Society of Soil Mechanics and Foundation Engineering (1966) by: (a) Kanakami and Asada for the Ivibune Primary School, (b) Kishida for an area near Benten Cho and at the Saiseikai Hospital, and (c) Watanabe for the Showa Oil Company refinery plant.

16. For the sites described above, the water tables were near the surface. No in situ soil density data were reported, only SPT results which showed that the sands ranged from loose to dense. By assuming a wet unit weight of 110 pcf for all the sites and by using the empirical relationships between SPT and effective overburden pressures from Gibbs

and Holtz (1957), the author estimated the relative densities of the sands at the above-mentioned sites. Figure 1 presents pre-earthquake and postearthquake effects on density at these sites, in terms of initial relative density versus change in relative density.

Nuclear Power Plants

17. The potential for seismically induced settlements was considered to have existed for at least six nuclear power plants in the United States. A brief discussion of the foundation conditions and the solutions to the potential settlement problems are presented for each plant in the following paragraphs.

Midland Power Plant

18. At the Midland plant, a layer of loose sand existed above the water table. The Seed and Silver (1972) analysis method was used to analyze the potential seismically induced settlements. Because of insufficient knowledge concerning seismically induced settlements, the predicted settlements were multiplied by a factor of 4 to ensure safety. In order to mitigate the settlement potential, the water table was raised 25 ft and the area preloaded to densify the material. After densification, the water table was lowered to its original elevation.

Shoreham Power Plant

19. The potential for seismically induced settlements existed in loose to medium dense sand and silt below the water table. Vibroflotation was used to densify the materials. Volume change behavior of the densified material was tested under laboratory cyclic triaxial loading and was shown to be insignificant.

Beaver Valley, Unit 2, Power Plant

20. A loose granular soil layer below the water table existed beneath the containment area. The Pressure Injected Footing technique was used to densify the material. Based on SPT results and the data presented by Lee and Albaisa (1974) and Seed, Arango, and Chan (1975), the potential seismically induced settlements were predicted to be insignificant.

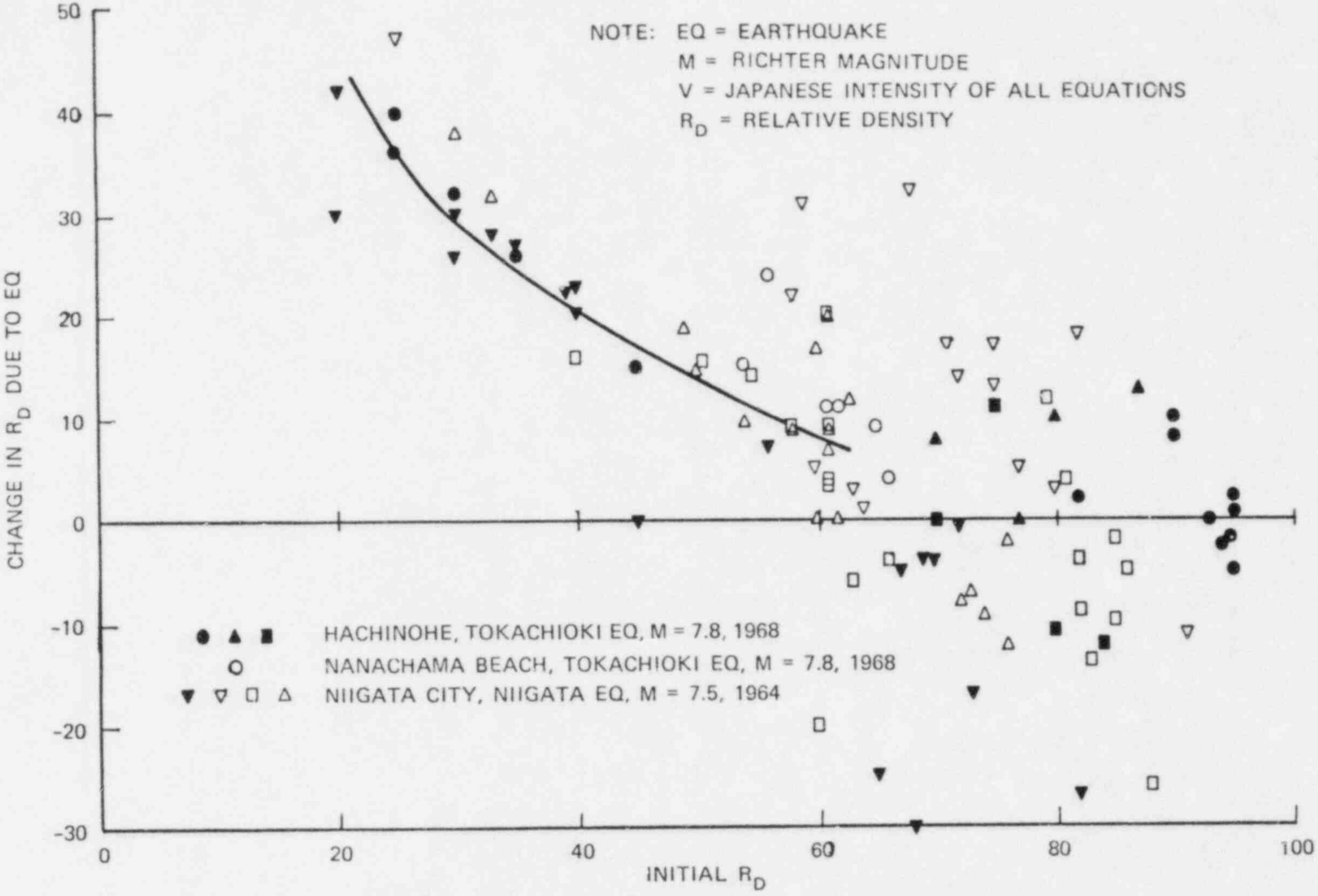

Figure 1. Pre-earthquake and postearthquake effects on density of sands (data from Japanese literature)

Beaver Valley, Unit 1, Power Plant

21. A medium dense granular soil layer below the water table exists beneath the turbine building. Based on SPT results and the data presented by Lee and Albasia (1974) and Seed, Arango, and Chan (1975), the potential seismically induced settlements were predicted to be insignificant. (This study was conducted in the review of operating reactor plants.)

Zimmer Power Plant

22. A loose granular material below the water table existed. The material was excavated and backfilled with sand compacted to about 85 percent relative density. A 2-ft-thick clay blanket was placed first, followed by 20 to 30 ft of sand, and topped with a layer of clay. Dewatering of the foundation soil is maintained in order to prevent the potential for liquefaction and settlements.

Susquehanna Power Plant

23. The pump house and spray pond are sited on stratified sands and gravels. Seismically induced settlements were evaluated by laboratory cyclic triaxial tests on reconstituted material and the methodologies of Seed and Silver (1972) and Lee and Albasia (1974). Potential settlements were predicted to be insignificant, and the water table is maintained at a depth 5 ft below the spray pond elevation.

PART III: DYNAMICALLY INDUCED SETTLEMENT BEHAVIOR

24. The current simplified assumption of earthquake loading in a soil mass is that the earthquake induces oscillating shear stresses and strains from the horizontal components of motion, with the primary effect of inertia loading on the vertical component of motion. Soil responses to earthquake loading are complex with many variables affecting the behavior. Depending on soil type and conditions, soils may experience liquefaction and/or settlements. Methodologies for analysis or prediction of settlements must take into consideration the major variables influencing the behavior. The following paragraphs describe the more significant variables identified. Discussions center on the behavior of cohesionless material, because the major interest and research of the geotechnical community in earthquake engineering has been concerned with seismic loading and liquefaction of sands.

Soil Type

25. The amount of settlement that a soil experiences as a result of repeated application of cyclic loading is a function of such things as (a) soil type, i.e. cohesive or cohesionless, (b) grain angularity for granular cohesionless soil, (c) grain-size distribution in granular cohesionless soil, (d) water content, (e) initial density, and (f) loading characteristics (amplitude and frequency).

26. Silver and Seed (1971) reported on settlements of a bed of dry silica sand (angular grains) under unidirectional loading in a shake table test and in simple shear. The settlement behavior from simple shear tests of a nonsaturated clayey sand and a uniform dry sand (Monterey #0, subrounded) is reported in Pyke, Seed, and Chan (1975). Comparisons of the reported data show that angularity and clay content modify the amount of settlement that may be expected. The clayey sand experienced the smallest amount of settlement. At low relative densities, in the neighborhood of 45 percent, the angular sand suffered significantly more settlement than the subrounded material; for example at

10 cycles of loading, with 0.20 percent shear strain, the angular sand had 0.76 percent settlement and the subrounded sand had 0.42 percent settlement. However, at high relative densities, in the neighborhood of 80 percent, the subrounded sand experienced larger settlements than the angular material; for example, at 10 cycles of loading with 0.20 percent shear strain, the subrounded sand had 0.25 percent settlement and the angular sand had 0.16 percent settlement.

27. Lee and Albaisa (1974) reported results from cyclic triaxial tests on six different uniformly graded saturated sands with mean grain sizes ranging from 0.1 to 3.0 mm. These data indicate that grain size has a significant effect, as large as a factor of 3, on volumetric strains. The coarser grained sands had larger volumetric strains than the finer grained ones. At a peak pore water pressure equal to the confining pressure, the volumetric strain upon drainage of the excess pore water pressure varied between about 0.35 percent for the 0.1-mm sand to about 1.1 percent for the 3.0-mm sand for a relative density of 50 percent.

28. The amount of research on the behavior of cohesive soils subjected to seismic loading conditions has been minor as compared with the amount of research devoted to cohesionless soil behavior. The problem of liquefaction in cohesionless soils is of a more critical nature when considering disastrous failures and has diverted attention from the behavior of cohesive soils which do not liquefy. However, saturated or near saturated cohesive soils have much lower shear strengths than cohesionless soils, and the ratio of seismic shear stress to failure strength can be much greater in cohesive soils.

29. Thiers and Seed (1968) reported on cyclic simple shear and cyclic triaxial tests on saturated San Francisco Bay mud and saturated silty clay samples from Anchorage, Alaska. They concluded that strength loss during earthquake pulses depends on the stress level, the number of loading cycles, and the strain at which the soil fails in a conventional undrained test. The shear strength of the San Francisco Bay mud was about 400 psf, and the Anchorage silty clays had shear strengths ranging from 200 to 1800 psf. These strengths are in the range of

earthquake-induced shear stresses. At 10 cycles of loading, all the materials failed at loads between 30 and 60 percent of the static strengths.

Density Effects in Cohesionless Soils

30. The relative density of cohesionless soils greatly affects the amount of volume change under cyclic loading. Effects of relative density on compaction or volume change have been studied and reported by a number of researchers including (a) Seed and Silver (1972), (b) Youd (1972), (c) Youd and Craven (1972), (d) Yoshimi (1967), (e) Whitman (1970), (f) Lee and Albaisa (1974), (g) Finn and Byrne (1976), (h) Pyke, Seed, and Chan (1975), and (i) Silver and Seed (1971). Typical of reported results on dry sand is that compaction volume change resulting from 10 cycles of laboratory simple shear loading can increase by a factor of 4 (such as 0.20 to 0.80 percent) as relative density decreases from 80 to 45 percent. Data on saturated sand from Lee and Albaisa (1974) indicate that, after dissipation of the pore water pressure in cyclic triaxial load tests (not taken to liquefaction), volume change increases by a factor of about 2 (such as 0.30 percent to about 0.60 percent) with relative density decreasing from 85 to 30 percent. Finn and Byrne (1976) showed that for a hypothetical problem in dry sand, surface settlements would increase by a factor of about 6.5 (such as 0.05 to 0.33 percent) with a decrease of relative density from 80 to 45 percent.

Moisture Effects in Cohesionless Soils

31. In saturated soil, pore water pressure builds up, and volume change is delayed and rate controlled by the pore water pressure dissipation. Research work reported by Martin, Finn, and Seed (1975) and Finn, Lee, and Martin (1977) has shown that volume change in dry sandy soil is the same as that which occurs in saturated sandy soils after pore water pressure dissipates. The primary effect of saturation of

the soil is to delay the volume changes.

32. In partially saturated soil, capillary forces between the individual soil particles affect the volume changes under loadings. Data presented in Broms and Forssblad (1969) showed that the vibratory compaction of sands varied with water content. The test data indicate that the vibratory compacted dry unit weight for partial saturation, as a percentage of that for dry or fully saturated conditions, varied from 73 percent for fine sand to 90 percent for coarse sand. For the dry and saturated conditions, the maximum vibratory compacted dry unit weights were equal. Broms and Forssblad (1969) also state that the results from vibratory vane shear tests indicated that the shear strength obtained for gravel, sand, gravelly silt, or crushed stone was the lowest value when the soil was either dry or saturated and was up to 6 percent higher for partially saturated conditions.

33. If a soil experiences liquefaction, volume change (settlement) occurs when the excess pore water pressure dissipates and the settlements associated with liquefaction are often large. As noted by (a) Finn, et al. (1978), (b) Seed and Lee (1969), (c) Seed, et al. (1973), and (d) Finn and Byrne (1976), unless stress reversals occur in a triaxial or simple shear cyclically loaded soil specimen, pore water pressure buildup may never rise to the level of confining pressure (initial liquefaction). However, a large accumulation of vertical strain may occur. This suggests that substantial settlements in percent of layer thickness can occur in saturated soils in which liquefaction does not occur but in which significant pore water pressures build up and then dissipate. Lee and Albaisa (1974) reported laboratory test results in which volume changes after liquefaction were more than 3 times (such as 1 percent to 3 percent volumetric strain) those when the tests were stopped prior to initial liquefaction and allowed to drain.

34. In dry or moist soil, the total stress acting is the effective grain-to-grain stress. However, in saturated soils, increasing pore water pressures due to cyclic loading cause decreasing effective grain-to-grain stresses which decrease the soil strength.

Soil Response to Earthquake Loading

35. Earthquake loading is three-dimensional and can be described using components of motion in three directions, two horizontal and one vertical components. The seismic loading is very irregular and induces irregular shear stress and strain as a function of time duration. The form, frequency, amplitude, and number of cycles of the input motion effect soil response.

36. Earthquake loading generates dynamic shear stresses and strains in a soil which cause slip at grain-to-grain contacts. For dry or moist sand, the intergranular slip leads to simultaneous volumetric compaction (Martin, Finn, and Seed 1975 and Silver and Seed 1971). In the case of saturated sands, the volume change is retarded because the water cannot drain instantaneously to accommodate the desired volume change. As a result, the relaxing sand skeleton transfers some of its intergranular or effective stresses to the pore water, and the pore water pressure increases. Therefore, pore water pressure and the tendency for volume change are directly related. In cohesive soils, the earthquake-induced shear strains can cause significant strength loss as shown by Thiers and Seed (1968).

37. During laboratory cyclic loading of sand, the volume changes in dry sand and the pore water pressures in saturated sand that develop depend on the dynamic shear strains (Martin, Finn, and Seed 1975 and Silver and Seed 1971), which in turn depend on the sand's stiffness and damping characteristics and the severity of the cyclic loading. Seed and Idriss (1970) and Hardin and Drnevich (1972) showed that for sand, the shear modulus is a function of the mean normal effective stress and the shear strain. Therefore, in saturated sands, the progressive development of pore water pressure during cyclic loading is continuously diminishing the level of effective stress and hence the shear modulus and the resistance to deformation. Depending on the permeability and pore water pressure distribution, there is simultaneous generation and dissipation of pore water pressure that is also affecting the effective stress and shear modulus. Cyclic loading produces slips at grain

contacts which lead to increased resistance of deformation in dry and moist sands, and in saturated sands after pore water pressure dissipation which is referred to as hardening (Martin, Finn, and Seed 1975, Silver and Seed 1971, and Finn, Brandsby, and Pickering 1970).

38. During an earthquake, a soil deposit is subjected to an irregular loading pattern which consists of intervals of loading, unloading, and reloading. As shown by Tatsuoka and Ishihara (1974), Ishihara, Tatsuoka, and Yasuda (1975), Ishihara and Okada (1978), Finn, Brandsby, and Pickering (1970), and Finn, Lee, and Martin (1975, 1976), sand has different behavioral characteristics of deformation, volume change, and pore water pressure generation in each of the different loading phases. In general, laboratory cyclic simple shear tests have shown that most of the volume changes in dry sands and the increases in pore water pressure in undrained saturated sands occur during the unloading portion of the load cycle (Finn, Lee, and Martin 1977). A current widespread procedure for dealing with random stress history is to convert it into an equivalent uniform cyclic stress (Seed 1976, Seed et al. 1975, Donovan 1971, and Lee and Chan 1972). However, research studies have shown that the magnitude of load pulses and the order in which they are applied in the stress and strain history of irregular loading are important to the volume change and pore pressure generation behavior of sands (Martin, Finn, and Seed 1975, Finn, Brandsby, and Pickering 1970, Seed, Mori, and Chan 1977, Shen, et al. 1978, Ishihara and Yasuda 1973, 1975, and Blázquez, Krizek, and Bazant 1980). Typical results of these above-mentioned laboratory triaxial and simple shear studies show the following:

- a. The influence of stress path and stress history are important at high material densities.
- b. For medium dense to dense soil samples, the cumulative damage (Donovan 1971) and equivalent uniform cycle (Lee and Chan 1972) procedures underestimate the conversion of irregular load patterns on the cyclic strength by 34 to 72 percent.
- c. Small stress peaks produce a disproportionately large strain after significant pore water pressure rise has occurred. However, prior to significant pore water

pressure rise, the small stress peaks may have a strengthening (hardening) effect.

d. The time rates of pore water pressure increases are intricately related to the position of the peak stress within the loading sequence. In an example by Blázquez, Krizek, and Bazant (1980), the time of increase of pore water pressure varied by 49 percent with the variation of position of the peak stress.

39. Maximum acceleration and frequency of input motion have significant effects on soil behavior and have been studied by Prakash and Gupta (1966), Finn and Byrne (1976), and Blázquez, Krizek, and Bazant (1980). Typical results are that doubling the acceleration of motion can increase the rate of pore water pressure rise by as much as 70 percent and increase the settlement by more than a factor of 3. Analytical computations by Blázquez, Krizek, and Bazant (1980), using their endochronic liquefaction model for typical field situations, indicate that pore water pressure can be greatly affected by the frequency of excitation below 1 to 2 Hz; excess pore water pressure can vary from negligible to initial liquefaction. They indicate that above a frequency of 1 to 2 Hz the soil response is quite insensitive to frequency.

40. The existence of a structure founded on the soil will affect the settlements of that soil from earthquake-induced loading. Two counteracting phenomona are present. The mean normal effective stress in the soil is increased by the weight of the structure; therefore, the shear modulus of the soil is increased. Increased shear moduli increase the resistance to shearing strain, and hence tend to reduce volume change (settlement). However, the opposite effect is produced by the mass of the structure, as the base shear produced by inertia forces generates additional shear stresses and strains and tends to increase settlement. In a theoretical study by Finn and Byrne (1976), the effect of the existence of a structure on sand was to significantly increase the settlement in all of the cases investigated. Depending on the structure weight and the sand density, the settlements under a structure were calculated to be 2 to 3.3 times the free-field settlements. Finn and Byrne (1976) concluded that the free-field settlements of a sand stratum provide a lower bound to the settlements to be expected under a structure during an earthquake.

41. As stated previously, earthquakes induce three-dimensional motions that are described by two horizontal and one vertical components. An extensive laboratory study of the settlement of dry sands under combined horizontal and vertical motions was conducted by Pyke, Seed, and Chan (1975). Their tests showed that the settlements caused by the combined horizontal components of earthquake motions are about equal to the sum of the settlements caused by the horizontal components acting separately. Settlements due to vertical accelerations were found to depend on the level of excitation and amounted to about 50 percent of the settlement caused by horizontal motions alone. An estimate of total settlement can be obtained by multiplying the response under a single horizontal earthquake component by a factor of about 3.0.

PART IV: DYNAMIC SETTLEMENT AND EFFECTIVE STRESS ANALYSES

42. In assessing methods for prediction and analysis of seismically induced settlements, the reader should recognize that despite more than 40 years of research on settlement analysis under static loads the current state-of-the-art procedures are only reliable to about ±25 to 50 percent (Lee and Albaisa 1974). Present earthquake analysis methods can deal with only unidirectional shaking response and cannot treat all three components of acceleration. Also, most of the dynamic analyses presently in use for the one-dimensional response of horizontal deposit soils are based on the total stress principle. However, a fundamental principle of soil mechanics is that deformations are controlled by effective stresses, and consequently effective stress methods of stability analysis are widely used in static problems. Until recently, no dynamic effective stress analysis techniques existed for lack of a model to predict the pore water pressures developed by cyclic or seismic loading. Analysis of total stress does not directly yield information on pore water pressures and the effect of strain softening that results from the reduced effective stress decreasing the shear modulus. A nonlinear effective stress model that adequately couples pore water pressure generation, material softening, and pore water pressure dissipation is the only type which can rationally treat these important aspects of the problem of seismically induced settlements.

43. In summary, the major factors which must be considered when computing the deformation response of a soil to earthquake motions are:

- a. Initial shear modulus in situ.
- b. Nonlinear variation of shear modulus with shear strain as effected by hardening and softening.
- c. Simultaneous generation and dissipation of pore water pressures at different rates.
- d. Effective stress changes.
- e. Damping.
- f. Irregular shear stress loading history.
- g. Volume change.

44. The following sections describe current models that could be used for analysis of horizontal one-directional base motion induced deformations of a soil column. Some of the models directly compute volume change and pore water pressure while others could be indirectly used to compute volume change. Currently, no method of analysis has been formulated for computing the settlements under the three-dimensional earthquake motions; therefore, a multiplication factor of 2 to 3 is recommended with settlement from one horizontal component as discussed previously.

Seed and Silver Model

45. Seed and Silver (1972) proposed a method for calculating the settlement of dry or moist sand. The procedures are as follows:

a. Use an equivalent linear method of dynamic analysis to determine the shear strain history in various layers of the sand.

b. Convert the shear strain history for each layer into an equivalent number of uniform cycles.

c. In a simple shear test, apply the uniform cycles of shear strain and determine the resulting volumetric or vertical strains for each sand layer.

d. Integrate the maximum vertical strains over the depth of sand stratum and obtain the surface settlement.

46. The disadvantages of this method are as follows:

a. The use of equivalent uniform cycles on response is questionable when, as discussed previously, an irregular shear strain history affects the soil volumetric strain history by both magnitude of the pulses and particularly by the order in which they occur.

b. An equivalent linear elastic procedure is used for determining strains in a nonlinear hysteretic medium subjected to irregular, random loading. In the analysis of an idealized soil profile by Finn, Martin, and Lee (1978), an equivalent linear elastic procedure produced a maximum shear stress distribution about 50 percent larger than those produced by two nonlinear procedures using the same soil characteristics. Therefore, the calculated shear strains should also be larger for the linear elastic procedure.

c. The equivalent linear methods cause a phenomenon of pseudoresonance and a false increase in the maximum amplification of acceleration. As discussed by Finn, Martin, and Lee (1978), an equivalent linear elastic method produced about 50 percent increase in the amplification of acceleration than did two nonlinear methods for an idealized soil profile.

Lee and Albaisa Model

47. Lee and Albaisa (1974) proposed a method for calculating settlements in saturated sands. The procedures are as follows:

a. At every depth, obtain the number of equivalent uniform cycles corresponding to the average seismic shear stress, as determined from an equivalent linear elastic analysis.

b. Obtain the pore water pressure for the equivalent uniform cyclic loading caused by the earthquake from laboratory cyclic triaxial test pore water pressure response data.

c. Create the same levels of pore water pressures in triaxial samples at in situ stress conditions by either cyclic loading or by back pressure.

d. When the pore water pressures are allowed to drain, the volumetric strains are computed from the volume of drained water.

e. Assume that the triaxial sample volumetric strains are equal to the field vertical strains, because in a horizontal layer with no lateral movement, they are equal.

f. Multiply layer thickness by volumetric strains and compute surface settlements.

48. This method has all the disadvantages of the Seed and Silver method. In addition, this method uses laboratory pore water pressure response data as determined from the application of uniform loading cycles. This procedure does not produce the same pore water pressure response histories as irregular loading. If the pore water pressures are different in uniform and irregular loadings, the volume change histories that occur on drainage must also be different. Shen, et al. (1978) showed triaxial test results that varied by a factor of 2 in time of significant pore water pressure increase, and by as much as a factor of 2 in magnitude of pore water pressure increase at a given

point in time for two irregular load patterns with the same significant number of equivalent uniform cycles and maximum deviator stress. Additionally, triaxial tests are used which are not as representative as simple shear tests of field earthquake-induced shear stress conditions.

Ishihara Model

49. A stress path effective stress model for soil dynamic response to earthquakes was proposed by Ishihara, Tatsuoka, and Yasuda (1975) and expanded in more recent work by Ishihara and Towhata (1980). The stress path is a line in which stress space is defined by p' and q axes where p' equals the effective isotropic confining pressure and q equals the deviator stress (maximum principal stress difference). This model permits the calculation of pore water pressures in response to irregular loading from stress path behavior in triaxial tests. The model does not concentrate on volume change or settlement, but rather on pore water pressure response and liquefaction potential. However, Ishihara did formulate a volume change relation dependent on the stress path determined pore water pressure in undrained loading, in order to account for time-dependent effects caused by softening and the simultaneous generation and dissipation of pore water pressure. The model also accounts for the nonlinear hysteretic strain behavior of soil.

50. The basic assumptions of the stress path model for predicting pore water pressure response are as follows:

- a. Pore water pressures are generated only by plastic deformation whenever the loading path penetrates the current yield surface established by previous levels of loading.
- b. The yield loci in the low stress range associated with earthquake loadings are represented by straight lines of constant shear strain which originate at the origin of the stress space.
- c. Yielding in either the compression or extension half of cyclic triaxial test load cycle is independent of any previous stress history in the other mode of deformation.

d. Unloading from a point in p'-q space results in purely elastic deformations and causes no change in pore water pressures or effective mean normal stress.

e. In the q-p' plane, loading below the current yield surface is represented by a vertical straight line with no plastic deformation occurring (no pore water pressure change). When the current yield surface is crossed, the stress path has a simple geometrical shape, such as a parabolic path, during plastic deformation (pore water pressure change). Upon unloading from a point, the stress path is a vertical straight line (parallel to the q-axis) with no plastic deformation occurring (no pore water pressure change).

51. Disadvantages of this method and assumptions dealing with the central theory of the model (pore water pressure generation) are as follows:

a. Experimental data do not support the assumptions that unloading and reloading within previously established yield loci are elastic and do not cause changes in pore water pressures. Data from stress controlled cyclic loading undrained triaxial tests (Ishihara, Tatsuoka, and Yasuda 1975) show clearly that within the assumed elastic region the stress paths are not vertical but sloped and increases in pore water pressures, in addition to those caused by plastic deformation, are associated with changes in stress state inside the current yield locus.

b. Experimental data (Ishihara, Tatsuoka, and Yasuda 1975 and Martin, Finn, and Seed 1975) do not support the assumed straight lines of yield loci and constant shear strain through the origin in stress space. These assumed lines imply that all stress paths in constant strain controlled cyclic loading tests are elastic and represented by a single vertical line except for the first half-cycle in each direction, which establishes the two limits to yield. In constant strain tests, the loading sequence is therefore caught in a trap of elastic response where pore water pressure cannot be generated. Constant shear strain controlled cyclic test data show that plastic deformations and pore water pressures continue to occur with cyclic loading and no elastic trap effect occurs.

Ghaboussi Model

52. Ghaboussi and Dikmen (1978) adapted a slightly modified version of the Ishihara stress path model into a method for dynamic effective

stress analysis of soils. The plastic deformation and pore water pressure generation region of the Ishihara model is assumed to be a quarter ellipse in the p'-q plane, with the remaining Ishihara model behavior intact. Ghaboussi and Dikmen (1978) do not directly relate volume change and pore water pressure generation and do not define settlement behavior. Once the pore water pressure behavior is computed by their approach, it could be related to settlement through an appropriate consolidation theory.

53. During excitation, the pore water pressures are simultaneously generating and dissipating, and pore water pressure response histories are computed. Ghaboussi and Dikmen (1978) model soil behavior as a two-phase medium. The stress-strain relations follow a hyperbolic stress-strain law with unloading assumed to be purely elastic, and a shear modulus equal to the initial loading modulus. As with the Ishihara model, this model can follow irregular loading.

54. Disadvantages of this method are the same as previously described for the Ishihara model. From Ghaboussi and Dikmen (1978), it is not clear whether or not the soil properties are changed (nonlinear moduli behavior and hardening and softening behavior) with the changing effective stresses during earthquake excitation.

Finn Model

55. Finn, Lee, and Martin (1977) proposed a model for one-dimensional dynamic effective stress analysis including settlement behavior. One of the resultant predictions is the settlement history of each layer and the surface total settlement. The model is applicable for both dry and saturated material and can be used in total or effective stress modes. A true nonlinear method of analysis composes the model that includes nonlinear stress-strain response, separate loading and unloading responses, hysteretic and viscous damping, and simultaneous generation and dissipation of pore water pressures. Hardening is accounted for during cyclic loading in dry sand, and the softening effects of increasing pore water pressure on saturated sand properties (changing

effective stress effects) are included. The model also operates from the irregular shear stress history of an earthquake.

56. The settlement-shear strain-pore water pressure analysis of the model is a volumetric strain formulation based on the coupling between volume changes in drained cyclic simple shear loading tests and the pore water pressures that develop in corresponding undrained tests. This coupling relation was developed and proven for the case of dry and saturated sand by Martin, Finn, and Seed (1975) and requires no assumptions about the behavior of sand under load other than that such behavior is uniquely dependent on the effective stresses. The volume change (settlement) relation is a function of pore water pressure generation and dissipation, magnitude of dynamic shear strain, and the accumulated volumetric strain (strain history).

57. The basic assumptions of the model are as follows:

a. The shear stress-strain law is hyperbolic.

b. The shear stress-strain curves during unloading and reloading, which behave differently, are defined by the Masing (1926) criterion.

c. The shear modulus and maximum shear stress are changing continually due to pore water pressure behavior in saturated soil which causes the effective stress regime to change, and they are changing continually in dry soil due to hardening.

d. Damping is both hysteretic and viscous in saturated soil.

e. The pore water is assumed an order of magnitude stiffer than the soil skeleton which results in negligible net volumetric strain during undrained cyclic loading. This means that the unrecoverable and elastic volumetric strains that occur must be equal and of opposite sense.

f. During an undrained load cycle, volumetric strain caused by nonrecoverable slip deformation at grain contacts results in the transfer of intergranular stress to the more incompressible water. There is a corresponding reduction in effective stress which results in release of recoverable volumetric strain stored by elastic deformation at grain contacts in the soil skeleton and constant volume is preserved.

g. Volume change in drained behavior is uniquely related to pore water pressure behavior in undrained conditions. The plastic volumetric strain (absorbed by rebound) which develops during a cycle of uniform shear strain in an undrained simple shear test is the same as the volumetric strain in a drained simple shear test.

h. Incremental volumetric strain is a function of the accumulated volumetric strain (volumetric strain history) and shear strain amplitude.

i. The pore water pressure and volumetric strain behavior relationships are valid for uniform cyclic loading, as well as for irregular shear strain histories and previous loading histories.

j. The distribution of pore water pressure, including simultaneous generation and dissipation, can be modeled by a form identical with that for the conduction of heat in a metal bar with distributed heat sources.

58. This method of analysis consists of a procedure for dynamic analysis, a specific stress-strain law, and a method for computing volume changes and pore water pressures concurrently with the dynamic response. The model predicts the phenomenological features of the dynamic response of saturated sand deposits (Finn, Lee, and Martin 1975), and has been used to predict the laboratory behavior of sands in both the deformation mode and pore water pressure mode. Good quality laboratory simple shear test data for volume change are required for successful implementation. The model is founded on volume change behavior characteristics which are dependent on grain shape, gradation, and structure and are less sensitive to sample disturbance than are models based on soil moduli or stress path behavior. Thus, this model is the least dependent on quality of soil sampling.

59. Disadvantages of this model are as follows:

a. At the present time, the constants describing the volume change characteristics and the associated pore water pressures must be measured for each sand under investigation. No generalizations for the model parameters exist.

b. The laboratory-determined parameters require cyclic simple shear tests. Cyclic simple shear equipment is not as widely available, nor as familiar to soil laboratory personnel, resulting in more expensive tests than use of cyclic triaxial tests.

Bazant Model

60. Bazant and Krizek (1976) and Blázquez, Krizek, and Bazant (1980) used the endochronic theory principles of Valanis (1971) to derive a densification-pore water pressure relationship for undrained sand that is similar to and is used in the same way as the one (Martin, Finn, and Seed 1975) used in the Finn model. This model is an alternative mathematical description and formulation for dealing with volume change and pore water pressures similar to Finn's model.

61. The disadvantages of this model are the same as those listed for the Finn model. Not as much laboratory or hypothetical field problem verification has been conducted with the Bazant model as with the previously described models.

Zienkiewicz Model

62. Zienkiewicz, Chang, and Hinton (1978) proposed that the single endochronic variable describing the accumulated volumetric plastic strains in the Bazant model (called a damage parameter) was related to the length of the total accumulated strain path in cyclic simple shear tests. They further incorporated the endochronic volumetric strain-pore water pressure model into an elastic-plastic method of dynamic effective stress analysis. The volume changes are obtained explicitly by a correlation with the damage parameter. A nonassociative isotropic hardening theory of plasticity is used which combines a Mohr-Coulomb yield surface with a Tresca-type potential surface with zero dilatancy. The effects of increasing pore water pressures on the yield criterion are accounted for but those on the elastic moduli are not.

63. Disadvantages of the model are:

- a. The elastic modulus is not adjusted as the effective stress changes.
- b. Isotropic hardening behavior for dynamic cyclic loading of soils is not supported by experimental data for unloading and reloading within previously established yield loci (Ishihara, Tatsuoka, and Yasuda 1975 and Martin, Finn, and Seed 1975).

Liou Model

64. Liou, Streeter, and Richart (1977) proposed a model for dealing with effective stress (pore water pressure) and liquefaction. The soil is treated as a two-phase medium and the model is only applicable to soils in a loose state. In the vertical direction, the soil deformation is considered to be constrained compression or expansion. The earthquake-induced motion component in the horizontal direction is treated in a shear wave submodel, while the vertical component of the earthquake-induced motion is treated in a pressure-wave submodel. Dependence of soil properties upon shearing strain amplitude and upon effective stress provides a coupling between the two submodels. Constrained compressibility is related to shear modulus and bulk compressibility of the skeleton by the theory of elasticity. Shear stress and strain computed from the shear wave submodel cause reductions in the shear modulus which causes a change in the constrained compressibility. However, the bulk compressibility remains unchanged. Pressure-wave motions generated by changes in the constrained compressibility caused by reductions in the shear modulus are calculated from the pressure-wave submodel. Because the constrained compressibility is increased by shear strain, the soil tends to settle. However, the pore water cannot be freely drained which causes pore water pressure rises and decreases in effective stress. Due to the lower effective stress, new shear modulus and maximum shear stress values are calculated. This completes one cycle of computation. The next cycle begins with the shear wave submodel and newly defined shearing characteristics of the soil. Volume change (settlement) can be calculated from the effective stresses and constrained modulus relationship.

65. Disadvantages of this model are:

- a. Shear modulus and constrained modulus vary during dynamic loading with the bulk compressibility remaining constant. Upon load reversal, the constrained modulus is assumed to remain constant with the changes being permanent and cumulative. The permanent cumulative changes in constrained modulus are inconsistent with the elasticity assumption relating it to shear modulus and bulk compressibility.

b. Experimental verification of the underlying assumptions is needed.

c. The inconsistency concerning the constrained modulus behavior makes any settlement computation questionable.

Baladi Model

66. Baladi and Rohani (1977) and Baladi (1978, 1979) developed a three-dimensional, elastic-plastic, work-hardening, constitutive cap-model based on the theory of effective stress for simulating the response of isotropic three-phase (partially saturated) soils. Strain softening with effective stress decrease is accounted for in the model. The model is a marriage of two single-phase elastic-plastic models which simulate, respectively, the drained and undrained stress-strain behavior of soils. Model coefficients for both drained and undrained behavior are determined experimentally with conventional laboratory triaxial tests, and the model is fitted to the laboratory response data. The model operates in stress space computing effective and total stresses from which pore water pressures are calculated. Volume change and deformation are related to the total and effective stress behavior. The model was developed considering only monotonic-type loading in compression and deviatoric shear stresses that do not completely reverse.

67. Disadvantages of the model are:

a. As discussed for previous models, work-hardening yield surface behavior for dynamic cyclic loading of soils is not supported by experimental data for unloading and reloading within the assumed purely elastic regions (Ishihara, Tatsuoka, and Yasuda 1975 and Martin, Finn, and Seed 1975).

b. Pore water pressure dissipation and volume change response histories are not calculated.

c. Cyclic loading with complete shear stress reversals cannot be presently considered.

d. The model has not been evaluated experimentally for dynamic cyclic loading, either with or without complete shear stress reversal.

e. Model response to irregular loading is not known.

f. With continuous loading (such as in cyclic loading), changes in parameters and elastic- and plastic-moduli, as deformations and effective stresses continue to change, are not forecast or accounted for.

g. Shear modulus is assumed to remain constant; however, experimental data show that a considerable reduction in the shear modulus occurs in cyclic tests with significant increases in pore water pressure. Shear modulus is a function of mean effective stress and will not remain constant if pore water pressures develop.

Prevost Model

68. Prevost (1977, 1978, 1979) and Prevost and Höeg (1976, 1977) proposed a multiyield surface analytical model which describes the anisotropic, elastoplastic, path-dependent, stress-strain-strength behavior of soils under undrained loading conditions. The model combines properties of isotropic and kinematic plasticity by using a field of plastic shear moduli which is defined in stress space by the relative configuration of nested yield surfaces. This combined isotropic and kinematic plasticity removes the previously discussed isotropic hardening behavior problems. For any loading history, the instantaneous configuration of the yield surfaces is determined by calculating the translation and contraction or expansion of each yield surface. Complex loading paths can be followed, and the locations of the nested yield surfaces are a direct expression of the material's memory of its past loading history. Elastic and plastic behavior in unloading and reloading is defined, and the behaviors in compression and extension (complete shear stress reversal) are different. The model parameters are derived from conventional laboratory tests and the model fitted to the behavior.

69. This model defines effective stress behavior in stress space from which pore water pressure behavior can be determined when combined with the total stress. Volume change is not directly predicted, but vertical and shear strain behavior is calculated. Therefore, volume change can be calculated from the strain behavior and can be related to the pore water pressure behavior.

70. Disadvantages of this model are:

a. The model assumes that the shear modulus remains constant. Shear modulus is a function of mean effective pressure and will not remain constant if pore water pressures develop. Considerable reduction in the shear modulus occurs with significant increases in pore water pressure.

b. Plastic deformations are assumed to occur at constant volume, which appears to be contrary to the successful prediction of pore water pressure increase and settlement (volume change) behavior.

c. Pore water pressure dissipation and volume change response histories are not calculated.

d. Changes in parameters and plastic moduli as loadings and deformations continue (such as in cyclic loading) are not forecast or accounted for.

e. Verification of the model has been limited. Studies on laboratory triaxial cyclic loading of lightly overconsolidated clays have been conducted; however, cyclic loading of sands or weak clays has not been performed.

Mroz Model

71. Mroz, Norris, and Zienkiewicz (1978, 1979) proposed a two-surface anisotropic theory of plasticity. This model, as for the Prevost model, combines properties of isotropic and kinematic plasticity which removes the isotropic hardening difficulties of previous models. The basic assumptions of the model are as follows:

a. A bounding yield surface exists in stress space (the Roscoe and Burland 1968 surface) which represents the consolidation history of a soil.

b. A separate yield surface exists, within the bounding surface, which defines an elastic domain. This yield surface may undergo contraction, expansion, and translation in the stress space during loading, unloading, and reloading but cannot intersect the existing bounding surface. An associated flow rule governs the plastic flow, and a hardening rule describes the variation of the two yield surfaces along any stress path.

c. The plastic modulus varies along any stress path between the inner yield surface and the bounding yield surface.

72. The model defines effective stress behavior, in stress space, which when combined with the total stress behavior allows pore water pressure determination. Volume change behavior can be calculated and could be related to the pore water pressure behavior. The model parameters are determined by and the model fitted to conventional laboratory triaxial tests.

73. Disadvantages of this model are:

a. Shear modulus remains constant, and the Prevost model discussion is applicable for this model.

b. Pore water pressure dissipation and volume change response histories are not calculated.

c. Rapidly changing parameters and plastic moduli in cyclic loading tests may be difficult to model.

d. For anisotropic soils and general loadings, the computations would be difficult because they would be carried out in a four-dimensional stress space.

e. The model has not been verified for cyclic loading.

PART V: SUGGESTED APPROACH

Finn Model

74. The method suggested for further research and development studies concerned with dynamically induced settlements is the Finn model (Finn, Lee, and Martin 1977). Of the previously presented models, the Finn model is the most completely formulated methodology with the least serious disadvantages. It is the only model concerned with and which directly computes settlement and pore water pressure response histories. The Finn model is a nonlinear method of effective and/or total stress analysis and applies to horizontally layered deposits shaken by horizontal shear waves propagating vertically. (The other models discussed are also restricted to one-dimensional conditions.) The major factors that affect dynamic response are dealt with and include: (a) initial in situ shear modulus, (b) continuous variation of shear modulus with shear strain, (c) continuous simultaneous generation and dissipation of pore water pressures, (d) changes in effective mean stress, (e) damping, (f) hardening and softening, and (g) irregular loading patterns. The Finn model was developed from and verified with laboratory simple shear tests of sand behavior data; however, it should be applicable for other soil materials.

75. Finn's model is in the form of two computer programs, DESRA-1 and DESRA-2 (Dynamic Effective Stress Response Analysis). DESRA-1 assumes an infinite rigid base and DESRA-2 assumes an elastic base with an energy-transmitting boundary which allows evaluation of the shear stresses transmitted across the boundary. Both boundaries are subjected to input motions selected by the user. Formulations of the Finn model are briefly described in the following paragraphs.

Earthquake-Loading Characteristics

76. A soil deposit is subjected to earthquake irregular loading patterns which consist of intervals of loading, unloading, and reloading.

For initial loading up to the first load reversal, the soil response is defined by the hyperbolic stress-strain relationship of Kondner and Zelasko (1963) which is

$$\tau = \frac{G_{mo}\gamma}{1 + (G_{mo}/\tau_{mo})\gamma} \tag{1}$$

where

τ = the shear stress at strain amplitude γ

G_{mo} = the initial maximum tangent modulus

τ_{mo} = the maximum shear stress that can be applied to the material

The quantities G_{mo} and τ_{mo} are derived from the equations proposed by Hardin and Drnevich (1972) and are expressed in terms of void ratio, vertical effective stress, coefficient of earth pressure at rest, and effective angle of shearing resistance.

77. During unloading and reloading, seismic motions cause continual changes in the maximum shear modulus G_{mt} and maximum shear stress τ_{mt} because of pore water pressure development in saturated soil and strain hardening in dry soil. Hysteretic shear stress-strain curves for unloading and reloading are defined by Masing (1926) criteria as

$$\frac{\tau - \tau_r}{2} = \frac{\dfrac{G_{mt}(\gamma - \gamma_r)}{2}}{1 + \dfrac{G_{mt}(\gamma - \gamma_r)}{2\tau_{mt}}} \tag{2}$$

where γ_r and τ_r define the point in shear stress-strain space where load reversal begins.

Pore Water Pressure and Strain Hardening

78. At the end of a motion cycle, the values of G_{mt} and τ_{mt} are updated for the effects of pore water pressure in saturated soil and strain hardening in dry soil. Volumetric strain increment $\Delta\varepsilon_{vd}$ is

related to the total accumulated volumetric strain ε_{vd} and the amplitude of shear strain γ by Martin, Finn, and Seed (1975) through the equation

$$\Delta\varepsilon_{vd} = \frac{C_1(\gamma - C_2\varepsilon_{vd}) + C_3\varepsilon_{vd}^2}{(\gamma + C_4\varepsilon_{vd})} \tag{3}$$

where C_1 to C_4 are constants that depend on the material properties. Pore water pressure increment Δu is related to the volumetric strain increment (Martin, Finn, and Seed 1975) by

$$\Delta u = \bar{E}_r \cdot \Delta\varepsilon_{vd} \tag{4}$$

where $\bar{E}_r$ is the one-dimensional rebound modulus, which is defined by

$$\bar{E}_r = \frac{(\sigma'_v)^{1-m}}{mK_2(\sigma'_{vo})^{n-m}} \tag{5}$$

where

σ'_v = the effective stress level
σ'_{vo} = the initial value of effective stress
K_2 , m , and n = experimental constants for a given material

79. Due to grain slips in dry soil during seismic loading and after pore water dissipation in saturated soils, the soil structure is hardening, and the amount of hardening is a function of the accumulated volumetric strain ε_{vd} . In saturated undrained material under cyclic loading, the desired volumetric strain is accommodated by an expansion of the structure due to the decreasing effective stresses caused by pore water pressure increases in such a way that the total volume change is zero. The shear modulus G_{mt} and maximum shear stress τ_{mt} applicable to the next cycle of loading are changed by the effective stress regime in saturated soil and by hardening in dry soil or after pore water

dissipation in saturated soil. They are defined by Finn, Lee, and Martin (1975, 1976) as

$$G_{mt} = G_{mo} \left(1 + \frac{\varepsilon_{vd}}{H_1 + H_3\varepsilon_{vd}}\right)\left(\frac{\sigma'_v}{\sigma'_{vo}}\right)^{1/2} \tag{6}$$

and

$$\tau_{mt} = \tau_{mo} \left(1 + \frac{\varepsilon_{vd}}{H_3 + H_4\varepsilon_{vd}}\right) \frac{\sigma'_v}{\sigma'_{vo}} \tag{7}$$

where σ'_v is the current vertical effective stress and H_1 to H_4 are experimental constants. Therefore, Equation 2 is continuously updated, and the current shear modulus and maximum shear stress are determined at any time for use in dynamic analysis.

80. If pore water can drain during the seismic motions there will be simultaneous generation and dissipation of pore water pressure, and the maximum pore water pressure will be less than that for undrained conditions. The distribution of pore water pressure at time t throughout depth z for the case of a specified boundary drainage condition or for distribution within an undrained layer is given by

$$\frac{\partial u}{\partial t} = \bar{E}_r \left[\frac{\partial}{\partial z}\left(\frac{K}{\gamma w} \cdot \frac{\partial u}{\partial z}\right) + \frac{\partial \varepsilon_{vd}}{\partial t}\right] \tag{8}$$

where K is permeability and γw is the unit weight of water. This equation is the same form as given by Sneddon (1957) for the conduction of heat in a metal bar with distributed heat sources. Postearthquake pore water pressure behavior is also determined with Equation 8.

Results Obtained from Finn Model

81. For a given soil site and earthquake acceleration history, the soil profile is divided into horizontal layers and converted to a lumped mass system. The masses are connected by nonlinear springs with stress-strain properties given by Equation 1 for initial loading and by Equation 2 for subsequent unloading and reloading. These equations

describe the nonlinear, strain-dependent, hysteretic behavior of the material. In addition to the hysteretic damping, viscous damping can be added for saturated soil to account for the resistance to the flow of water through the pore channels of the soil skeleton. The differential equations of motion are solved with each increment of integration including the effects of shear strains, hardening, and pore water pressures taken into account by Equations 2-7. For the effects of contemporaneous generation and dissipation of pore water pressure, Equation 8 is solved simultaneously with the equations of motion. Computed results from the Finn model are as follows:

- a. Acceleration history for each layer.
- b. Shear stress and shear strain histories for each layer.
- c. Pore water pressure history for each layer.
- d. Vertical settlement history for each layer.
- e. Surface total vertical settlement history for the site.

Verification of Finn Model

82. More laboratory experimental verification of the Finn model has been conducted than for any of the other models presented. The most important part of Finn's model, as far as settlement calculations are concerned, is Equation 3 which relates increments in volumetric strain with current shear strain and the accumulated volumetric strain. Equation 3 is actually a settlement-shear strain model describing the effects of shear strain and the history of shear strain loading. Equation 4 is of less importance in settlement calculations, but indirectly reflects the behavior of the volumetric strain increments (Equation 3). Therefore, Equation 4 can be used with laboratory test results to imply verification of the volume change equation.

83. An example of comparison between experimental and theoretical data for shear stress versus shear strain using Equations 1, 2, 6, and 7 is shown for Crystal Silica Sand in Figure 2 (from Finn, Lee, and Vaid 1976). Another example of the behavior of Equations 1, 2, 6, and 7 is shown in Figure 3 for liquefaction data for Crystal

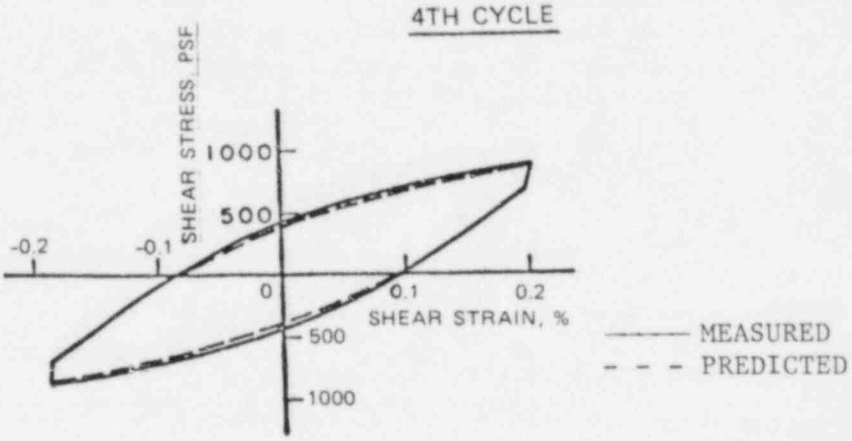

Figure 2. Computed and experimental loops. (Finn, Lee, and Vaid 1976)

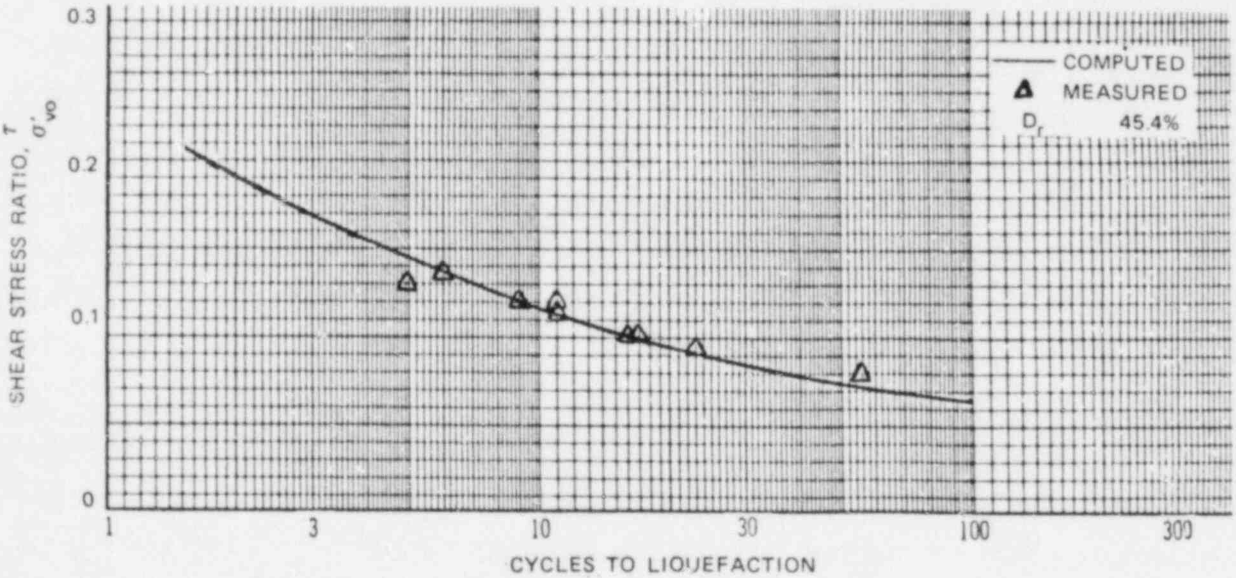

Figure 3. Comparison of computed and measured liquefaction resistance. (Data from Finn, Lee, and Martin 1977)

Silica Sand (data from Finn, Lee, and Martin 1977, ASCE, Vol 103, No. GT6).

84. Figure 4 (from Martin, Finn, and Seed 1975 and a contract report by Finn for WES) shows measured and predicted volumetric strains, from Equation 3, for a series of irregular shear strain histories applied to Crystal Silica Sand at $D_r = 45$ percent. In Figure 4, the shear strain histories were specified; therefore, the complete dynamic analysis of Finn's model was not used. Figure 5 shows predicted and measured volumetric strains for constant stress amplitude ($\tau/\sigma'_{vo} = 0.196$, 0.146, and 0.1195) cyclic simple shear tests on dry Ottawa sand at $D_r = 47$ percent, which were computed by the complete analysis of Finn's model. (This work was conducted by Finn under contract with WES.)

85. Figure 6 (from Finn and Bhatia 1980) shows an example of the unique relationship between volumetric strain and pore water pressures, which is a basic assumption of Finn's model. The data in Figure 6 are measured from drained and undrained constant strain cyclic simple shear tests on Ottawa sand at $D_r = 45$ percent. Similar patterns occurred for $D_r = 60$ percent. Figure 7 (from Finn and Bhatia 1980) shows a typical comparison for measured and calculated pore water pressure response using Equation 4 for constant stress cyclic simple shear tests. The calculations in Figure 7 were made using the measured shear strains with Equation 3.

86. Figure 8 (from Finn and Bhatia 1980) compares measured and calculated pore water pressure response to an irregular strain history in cyclic simple shear in the laboratory. The calculated response in Figure 8 was made using Equations 3, 4, and 5. As can be seen, the volumetric strain and pore water pressure models are operating quite satisfactorily. Finn attributes the differences in Figure 8 to the fact that the irregular strain pattern was manually applied and the peaks were not as uniform as implied in the pattern.

87. A comparison of predicted and measured pore water pressure behavior under the effects of previous loading history is shown in Figure 9 (from Finn and Bhatia 1980). The sand sample was subjected to a loading-drainage-loading sequence in cyclic simple shear. Initial

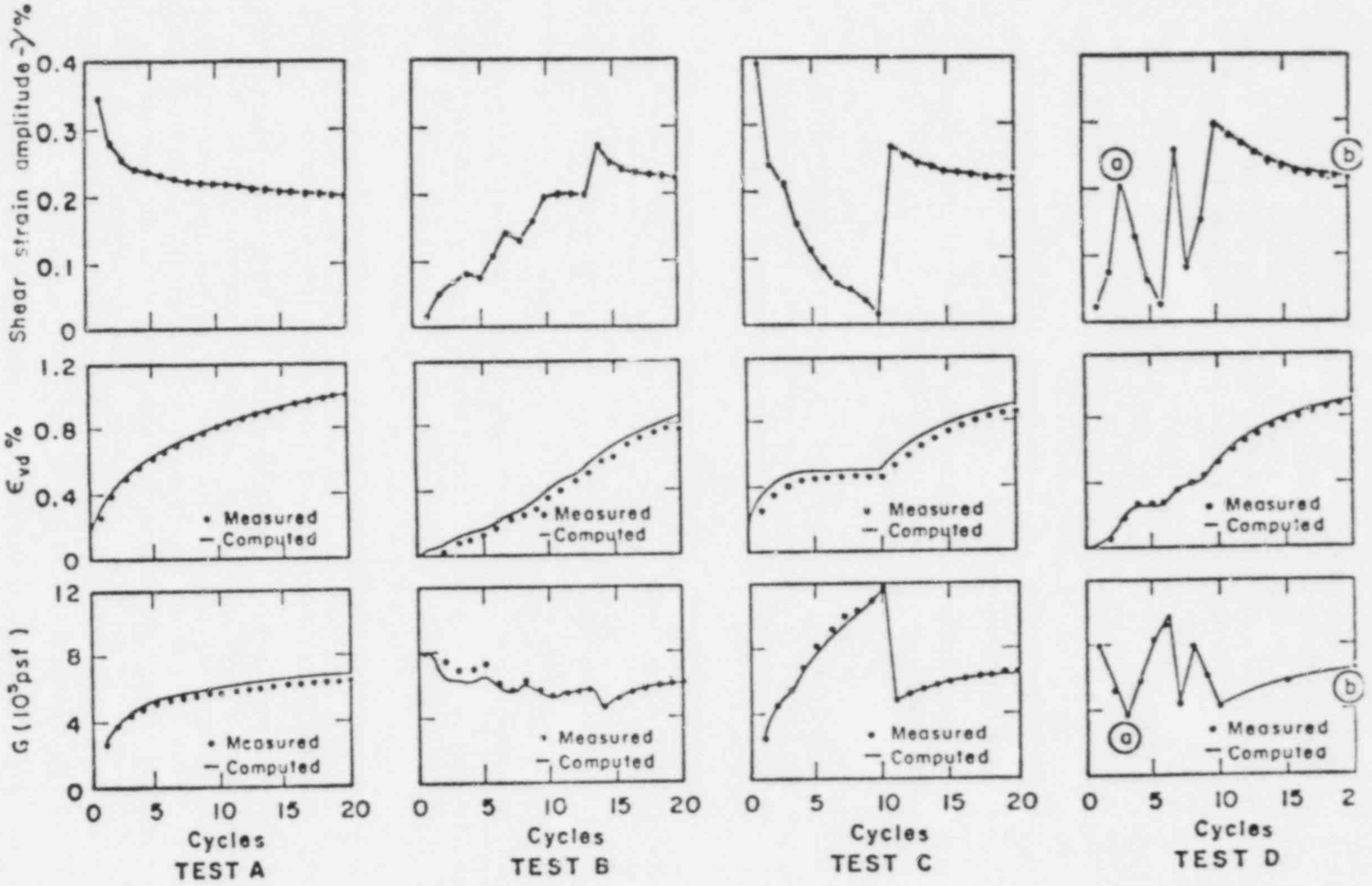

Figure 4. Effect of cyclic strain history on volumetric strain and shear modulus ($1 \text{ psf} = 47.9 \text{ N/m}^2$) (Martin, Finn, and Seed 1975 and a contract report by Finn for WES)

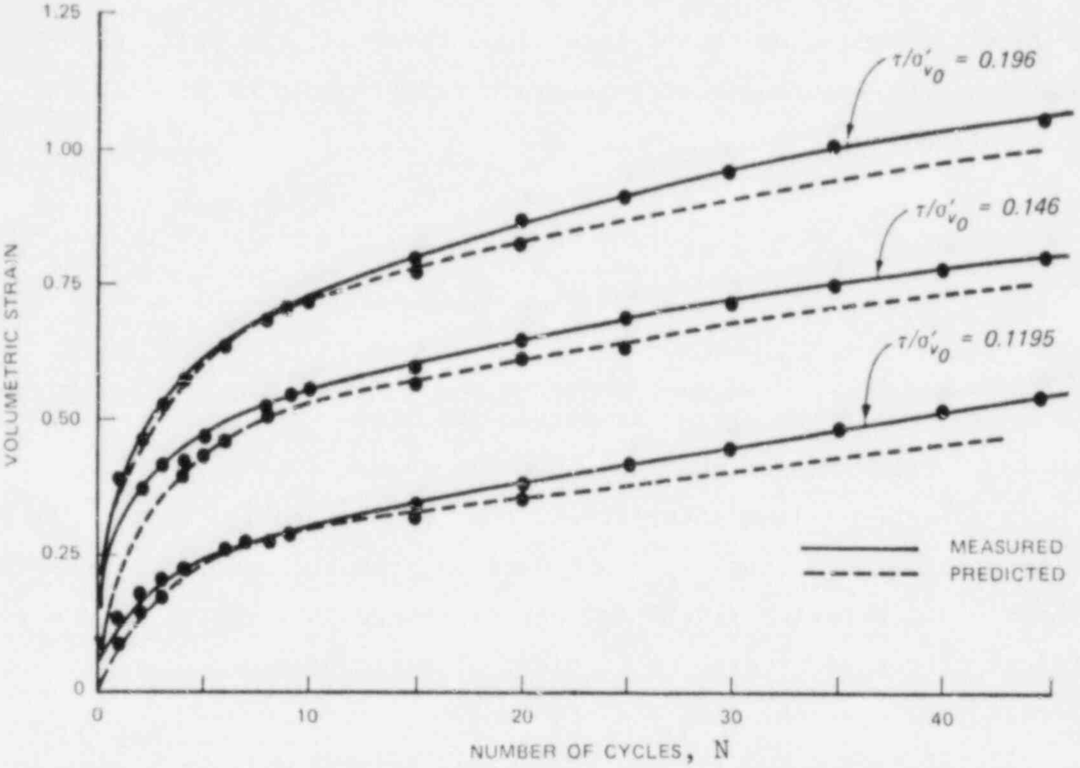

Figure 5. Comparison of predicted and measured volumetric strain

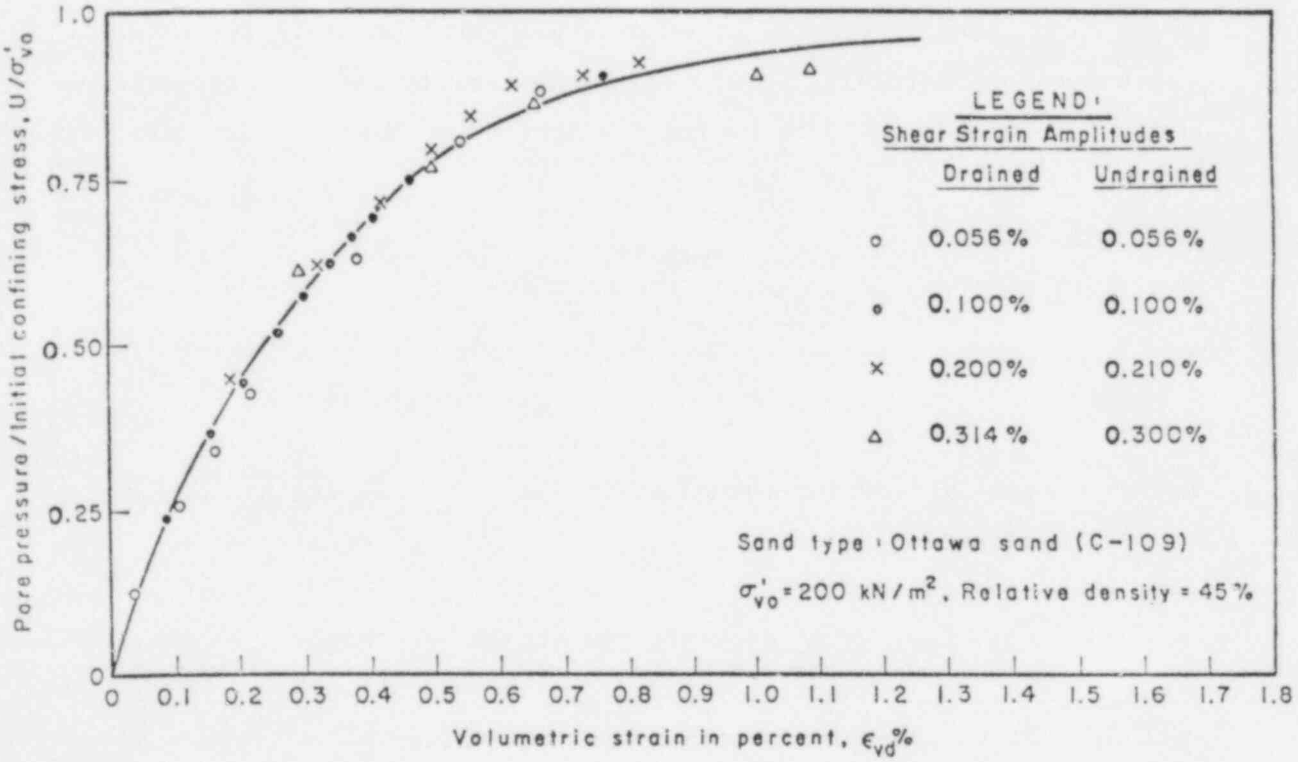

Figure 6. Relationship between volumetric strains in drained and pore water pressures in undrained tests of the same type in constant strain cyclic simple shear tests. (Finn and Bhatia 1980)

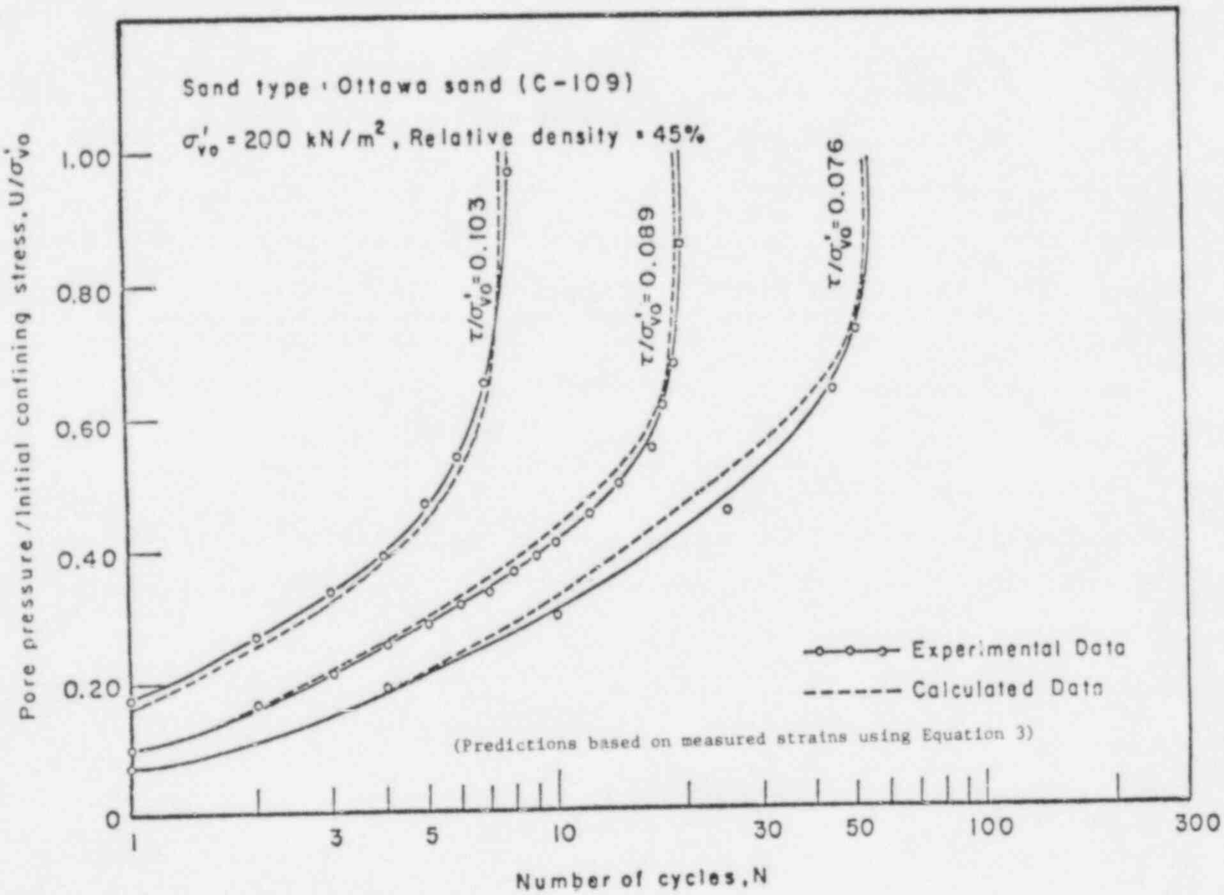

Figure 7. Predicted and measured pore water pressures in constant stress cyclic simple shear tests D_r = 45 percent. (Finn and Bhatia 1980)

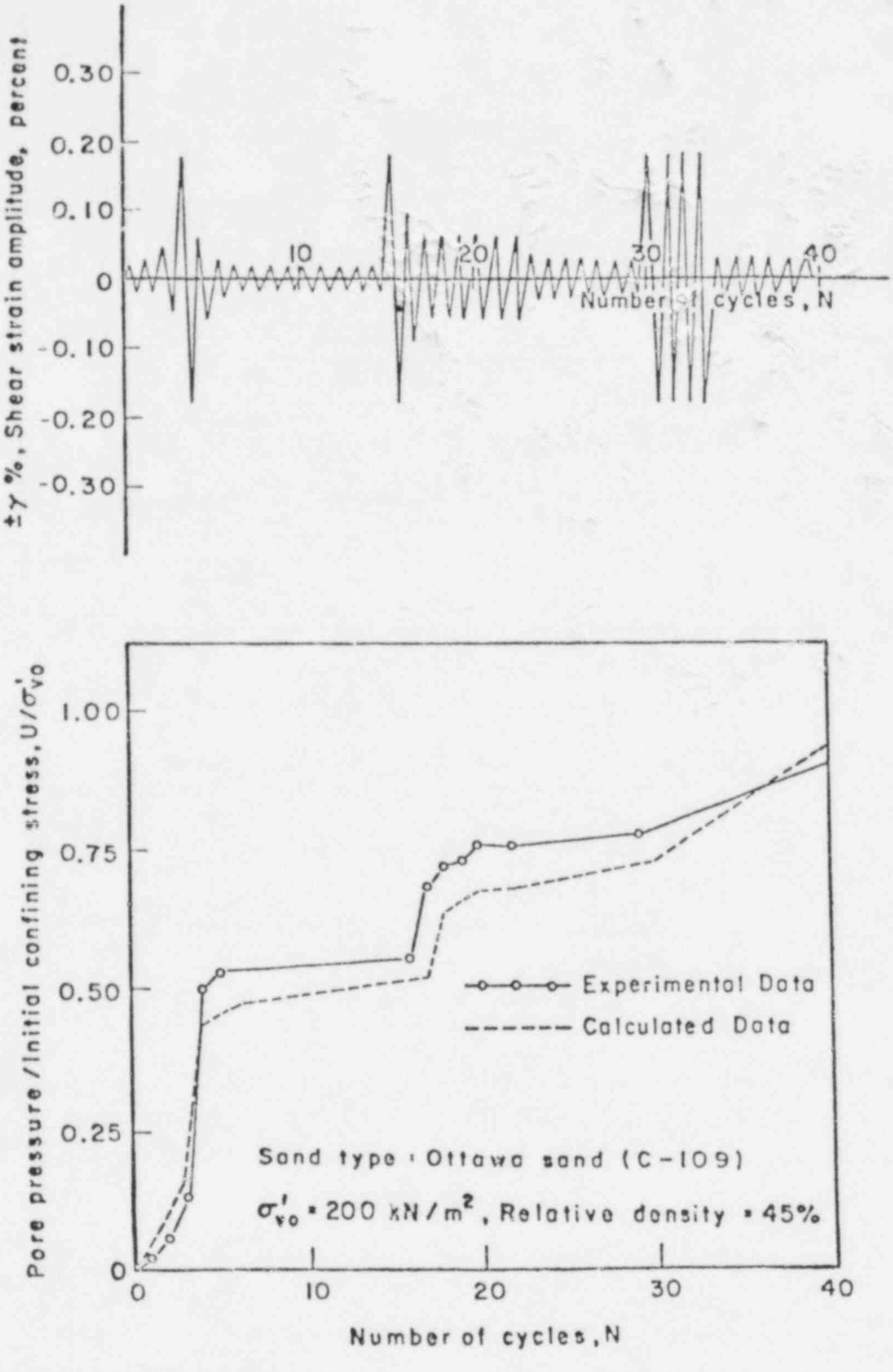

Figure 8. Predicted and measured pore water pressures under irregular cyclic loading. (Finn and Bhatia 1980)

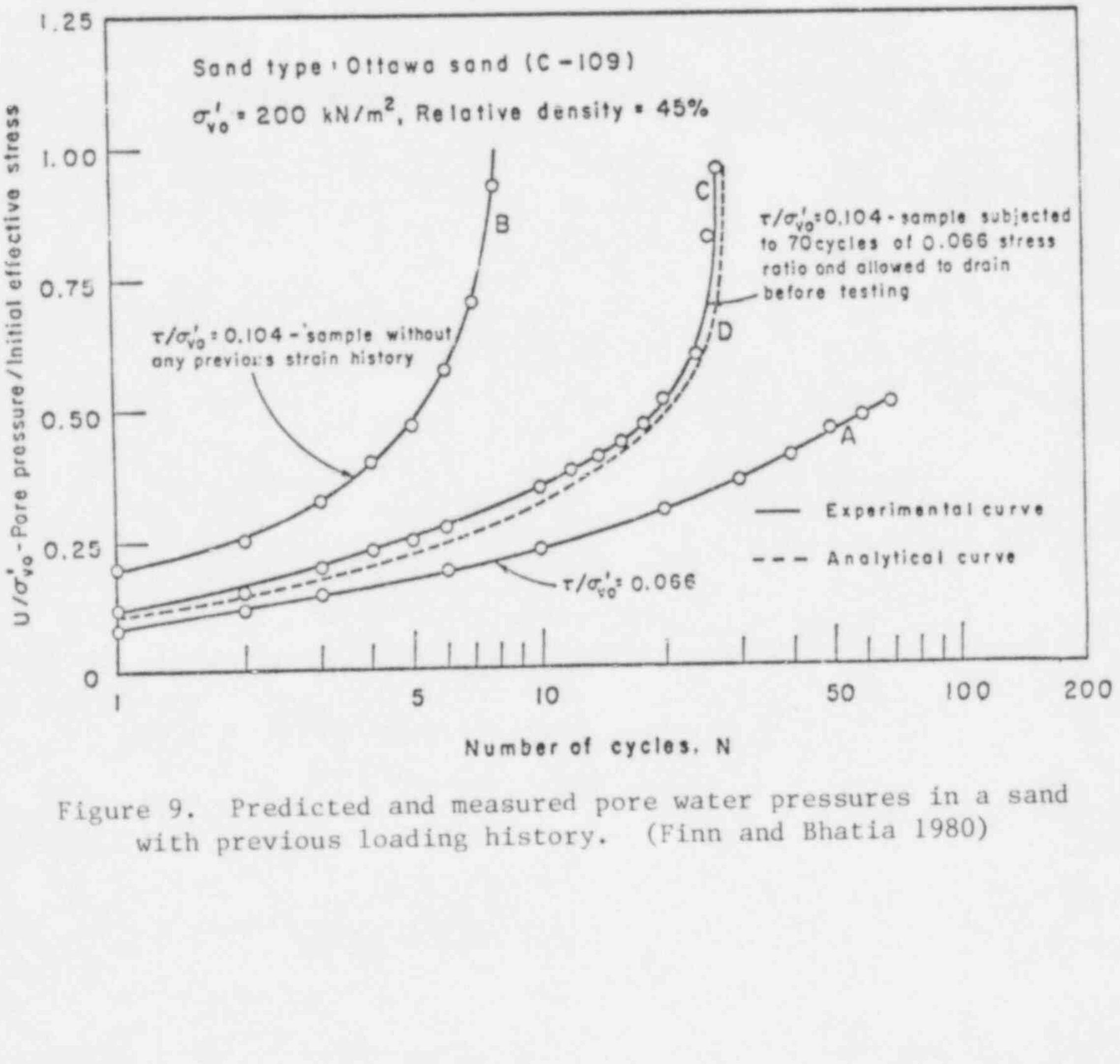

Figure 9. Predicted and measured pore water pressures in a sand with previous loading history. (Finn and Bhatia 1980)

loading was at a stress ratio of 0.066 and the second loading, after drainage, was at a stress ratio of 0.104. Strain hardening and effective stress effects after the first loading were calculated by Equations 6 and 7. As seen, the comparison is quite good.

PART VI: CONCLUSIONS

88. Based on this study, the following conclusions are drawn:

- a. Earthquake-induced surface settlements have ranged from 0.7 to 10 percent of layer thickness for the relatively few incidences where reliable estimates have been made of settlement magnitudes and soil conditions.
- b. Standard penetration test results obtained for pre-earthquake and postearthquake conditions in Japan show that relative densities have changed between 188 percent increase to 44 percent decrease. (Some of this difference may be due to energy input effects.)
- c. The potential for seismically induced settlements has been investigated for at least six nuclear power plants in the United States.
- d. The major factors that control seismically induced settlements in soil and that must be properly accounted for in an analysis methodology are:
  - (1) Initial in situ shear modulus.
  - (2) Nonlinear continuous variation of shear modulus with shear strain history as affected by hardening and softening.
  - (3) Continuous and simultaneous generation and dissipation of pore water pressure.
  - (4) Continuous effective stress changes.
  - (5) Damping.
  - (6) Irregular shear stress loading history.
  - (7) Volume change.
- e. The Finn model is presently superior to the other methods available for seismic settlement analysis. Finn's model is:
  - (1) The most complete formulation.
  - (2) Fully computer packaged.
  - (3) Accounts for the major factors.
  - (4) The only model directly concerned with settlement behavior.
  - (5) Has the least serious disadvantages.
  - (6) Has the most complete laboratory verification.

f. The majority of research that is applicable to seismic settlement analysis has been concerned with cohesionless sand behavior. However, the Finn model should also be applicable to cohesive soils.

PART VII: RECOMMENDATIONS

89. The Finn model is recommended for use in analytical studies and continued research of seismically induced settlements in soil. A correlation or an equivalence between laboratory triaxial and simple shear tests should be developed in the laboratory with respect to the generation of volumetric strains during cyclic loading. The triaxial test is more widely available and much easier to conduct than the simple shear test and would facilitate the use of Finn's model by the geotechnical community. Laboratory research and verification should be conducted with regard to the application of Finn's model to the behavior of cohesive soils.

90. In a rational sense, Finn's model fulfills the requirements for a general consistent theory for seismically induced settlement analysis and prediction, based on the present one-dimensional vertically propagating shear wave simplification of earthquake motion. Final verification of the model capabilities is needed. Ideally, final verification should come from predicting settlements at a site in which settlements will be induced at a known point in time or less ideally for a site where settlement has occurred. Unfortunately, a fully documented site in terms of exact data on settlements, pore water pressures, ground accelerations, and preearthquake soil conditions is not presently known to exist. Neither is a site known where, in the future, an earthquake will occur at an approximate point in time.

91. A potential site should be selected where earthquakes occur frequently (such as the Imperial Valley, California), detailed soils analyses conducted, and settlement predictions made for comparison with postearthquake conditions. Additionally, the site should be instrumented to measure accelerations, deformations, settlements, and pore water pressure behavior. Such a site could yield valuable information on earthquake response for a number of years, and could be used for field verification of earthquake dynamic analysis methodologies in addition to verification of settlement analysis techniques.

92. Another possibility for field verification of Finn's model

would be to locate a sand site where explosives could be used to induce earthquake-like shear stresses and strains (Bruce, Lindberg, and Abrahamson 1979). Detailed soils analyses could be conducted and settlement predictions made. The test site should be adequately instrumented as described above. Prerequisite careful studies of this approach would be necessary, because recent studies reported by Zienkiewicz, Chang, and Bettess (1980) indicate that pore water pressures generated in the near field in blasting are controlled by different variables and behavior mechanisms than by those generated by earthquakes.

93. In addition to the above-mentioned field test site investigations, Finn's model should be used to study and predict behavior under laboratory-controlled conditions, including varying input earthquake histories provided by large-scale shaking table tests of a sand layer. The test specimen and mass would have to be large enough for boundary effects to be negligible and for the adequate development of a vertical distribution of shear stress and strain during the test that is typical of a sand deposit in the field. Measurements should be made of accelerations, deformations, settlements, and pore water pressures.

94. Prerequisite studies to meaningful shake table tests should be conducted. The dynamic properties of the sand to be tested at very low confining pressures need to be determined, because, as pointed out by Seed and Silver (1972), data on moduli and damping of sands under low confining pressures are scarce and may not be very reliable.

95. Another prerequisite study should be for adequate mass size, as described previously. Confining pressures may be increased to the range for which available dynamic properties are applicable by enclosing the test sample in a rubber membrane, applying a vacuum, and/or by applying a large mass to the surface of the sample. However, the enclosed sample will be stiffened by the confining stress such that significant shear strains will probably not develop, and an added large mass will cause uniform shear stress and strain rather than the typical field distribution. Therefore, the best solution would seem to be having a large enough sample for development of distributed stresses and strains and characterized for low confining pressures. A large sample, of course, requires a large shaking table.

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NRC FORM 335
(11-81)

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BIBLIOGRAPHIC DATA SHEET

1. REPORT NUMBER *(Assigned by DDC)*
NUREG/CR-3380

4. TITLE AND SUBTITLE *(Add Volume No., if appropriate)*
Current Methodologies for Assessing Seismically Induced Settlements in Soil

2. *(Leave blank)*

3. RECIPIENT'S ACCESSION NO.

7. AUTHOR(S)
Richard Ledbetter

5. DATE REPORT COMPLETED
MONTH June | YEAR 1983

9. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS *(Include Zip Code)*
Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
P.O. Box 631
Vicksburg, Mississippi 39180

DATE REPORT ISSUED
MONTH August | YEAR 1983

6. *(Leave blank)*

8. *(Leave blank)*

12. SPONSORING ORGANIZATION NAME AND MAILING ADDRESS *(Include Zip Code)*
Division of Health, Siting and Waste Management
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

10. PROJECT/TASK/WORK UNIT NO.

11. FIN NO.
B7002

13. TYPE OF REPORT
Technical

PERIOD COVERED *(Inclusive dates)*

15. SUPPLEMENTARY NOTES

14. *(Leave blank)*

16. ABSTRACT *(200 words or less)*

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17. KEY WORDS AND DOCUMENT ANALYSIS

17a. DESCRIPTORS

Earthquake-induced settlements
liquefaction

17b. IDENTIFIERS/OPEN-ENDED TERMS

18. AVAILABILITY STATEMENT
Unlimited

19. SECURITY CLASS *(This report)*
Unclassified

21. NO. OF PAGES

20. SECURITY CLASS *(This page)*
Unclassified

22. PRICE
$

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555

OFFICIAL BUSINESS
PENALTY FOR PRIVATE USE, $300

NUREG/CR-3380 CURRENT METHODOLOGIES FOR ASSESSING SEISMICALLY INDUCED SETTLEMENTS IN SOIL AUGUST 1983

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