NUREG-0026

EVALUATION OF SOIL LIQUEFACTION POTENTIAL FOR LEVEL GROUND DURING EARTHQUAKES

A SUMMARY REPORT

Shannon & Wilson, Inc. and Agbabian Associates
for
U.S. Nuclear Regulatory Commission

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MASTER

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ABSTRACT

The report evaluates the results of a three-year research program conducted to investigate the settlement and liquefaction of sands under multi-directional shaking.

The investigation indicated that the behavior of a saturated sand under cyclic loading conditions is a function of its geologic and seismic history and grain structure as well as its placement density. It is concluded that the resistance to liquefaction of a sand deposit can best be estimated by laboratory testing on undisturbed samples.

It is shown that cyclic triaxial tests used in conjunction with appropriate correction factors to account for multi-directional shaking, simple shear loading conditions, and overconsolidation effects can provide valid data on cyclic loading characteristics.

The concepts of "limited strain potential" and acceptable value of the factor of safety against initial liquefaction are introduced in the report.

Finally, the two basic methods for evaluating liquefaction potential and the effects of liquefaction are reviewed and updated with the information obtained through this research effort.

FOREWORD

This report evaluates the results of a three-year research program conducted by the University of California to investigate the settlement and liquefaction of sands under multi-directional shaking. This work represents a part of continuing studies to evaluate free-field soil behavior under earthquake loading conditions. This and other related studies each provide important steps in the overall project for improving methods for evaluation and prediction of soil behavior at potential nuclear power plant sites under seismic loading conditions.

This work was conducted by the University of California, Berkeley, under subcontract to the joint venture of Shannon & Wilson, Inc. (SW) and Agbabian Associates (AA) as a part of Contract No. AT(04-3)-954 between the joint venture (SW-AA) and the United States Nuclear Regulatory Commission.

The authors of this report are Dr. H. Bolton Seed, Professor of Civil Engineering, University of California, Berkeley; Dr. Ignacio Arango, Staff Consultant-Dynamics, Shannon & Wilson, Inc.; and Clarence K. Chan, Research Engineer, University of California, Berkeley. For the joint venture, Dr. I. Arango served as the Project Monitor for the entire investigation, and Dr. R.P. Miller was Project Manager for the joint venture. Mr. S.D. Wilson provided a critical review of the report.

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LIST OF SYMBOLS

a_{max}	Maximum ground acceleration
C_N	Correction factor for penetration resistance
c_r	Stress correction factor for cyclic triaxial tests
C_{ssr}	Stress correction factor for cyclic simple shear tests
D_r	Relative density
e_o	Initial void ratio
g	Acceleration of gravity
K_o	Coefficient of earth pressure at rest
M	Earthquake magnitude, Richter scale
N_c	Number of cyclic stress applications
N_ℓ, $N_{eq.}$	Number of cyclic stress applications required for liquefaction
N	Standard Penetration Resistance (blows per foot)
N_1	Standard Penetration Resistance corrected to an overburden pressure equal to 1 tsf
OCR	Overconsolidation ratio
r_d	Stress reduction factor
u	Pore water pressure
u/σ'_o	Pore water pressure ratio
α	Ratio between the stress ratio causing liquefaction in the field and the stress ratio determined by cyclic triaxial tests
γ	Cyclic shear strain
γ_T	Total unit weight of soil
Δ_e	Change in void ratio
Δ_u	Change in pore water pressure
w	Water content
σ_o	Total overburden pressure
σ'_o, σ'_f	Effective overburden pressure (initial and final stages)

σ_{dc}	Cyclic deviator stress in triaxial tests
σ'_1	One ton per square foot
σ_3	Chamber pressure in triaxial tests
σ'_{3c}	Effective chamber pressure in triaxial tests
τ, τ_H, τ_{HV}, τ_{MAX}, τ_M	Maximum cyclic shear stress
$\tau_{eff.}$	0.65τ
τ/σ'_o	Stress ratio

EVALUATION OF SOIL LIQUEFACTION POTENTIAL FOR LEVEL GROUND DURING EARTHQUAKES

A SUMMARY REPORT

By H. Bolton Seed[1], Ignacio Arango[2], and Clarence K. Chan[3]

CHAPTER 1
GENERAL STATE OF KNOWLEDGE

1.1 INTRODUCTION

Since the catastrophic failures due to soil liquefaction in the Alaska (1964) and Niigata (1964) earthquakes, great interest has developed in this phenomenon in seismically active regions of the world. Major landslides (Seed, 1968), lateral movements of bridge supports (Ross, Seed, and Migliaccio, 1969), settling and tilting of buildings (Ohsaki, 1969), and failure of waterfront structures have all been observed in recent years as a result of this phenomenon, and efforts have been increasingly directed to the development of methods for evaluating the liquefaction potential of soil deposits. It is the purpose of this report to review recent developments in procedures available for this purpose and suggest the most appropriate methods for use in engineering design at the present time.

It should be noted at the outset that the term "liquefaction" as used in this report describes a phenomenon in which a cohesionless soil loses strength during an earthquake and acquires a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet. When the term was originally introduced, it was intended to describe a phenomenon in which a soil could undergo large movements, as in flow slides, with an essentially constant and very low residual resistance to deformation resulting from the development of high pore water pressures. However, damages resulting from limited movements of

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only several feet in recent earthquakes have been attributed to "liquefaction." While the term "cyclic mobility" has been suggested as a more appropriate term to describe this latter type of soil behavior, the broader use of the term "liquefaction" is adopted in the following pages.

However, in an effort to clarify the sometimes misleading impression that differences in terminology reflect wide differences of opinion concerning the nature of the phenomena involved, the following qualifications of the term "liquefaction" will also be used:

a. "Initial Liquefaction": denotes a condition where, during the course of cyclic stress applications, the residual pore water pressure on completion of any full stress cycle becomes equal to the applied confining pressure; the development of initial liquefaction has no implications concerning the magnitude of the deformations which the soil might subsequently undergo; however, it defines a condition which is a useful basis for assessing various possible forms of subsequent soil behavior.

b. "Initial Liquefaction with Limited Strain Potential" or "Cyclic Mobility": denotes a condition in which cyclic stress applications develop a condition of initial liquefaction and subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil dilates, the pore pressure drops, and the soil stabilizes under the applied loads.

c. "Liquefaction": denotes a condition where a soil will undergo continued deformation at a constant low residual stress or with no residual resistance, due to the build-up and maintenance of high pore water pressures which reduce the effective confining pressure to a very low value; pore pressure build-up may be due either to static or cyclic stress applications.

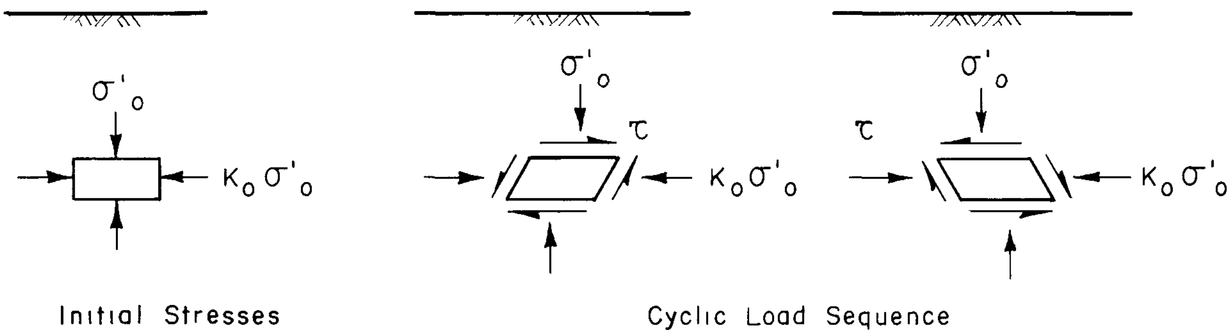
Hopefully, within this framework of terms, it will be possible to adequately describe the various phenomena involved when pore pressures are generated in soils by earthquake motions with resulting deformations of tolerable or intolerable magnitudes for engineering purposes.

For the purposes of this evaluation, considerations of liquefaction potential are limited to cases of relatively level ground where the response to stresses induced by an earthquake is not further complicated by the presence of initial horizontal shear stresses due to the proximity of ground surface irregularities or loads.

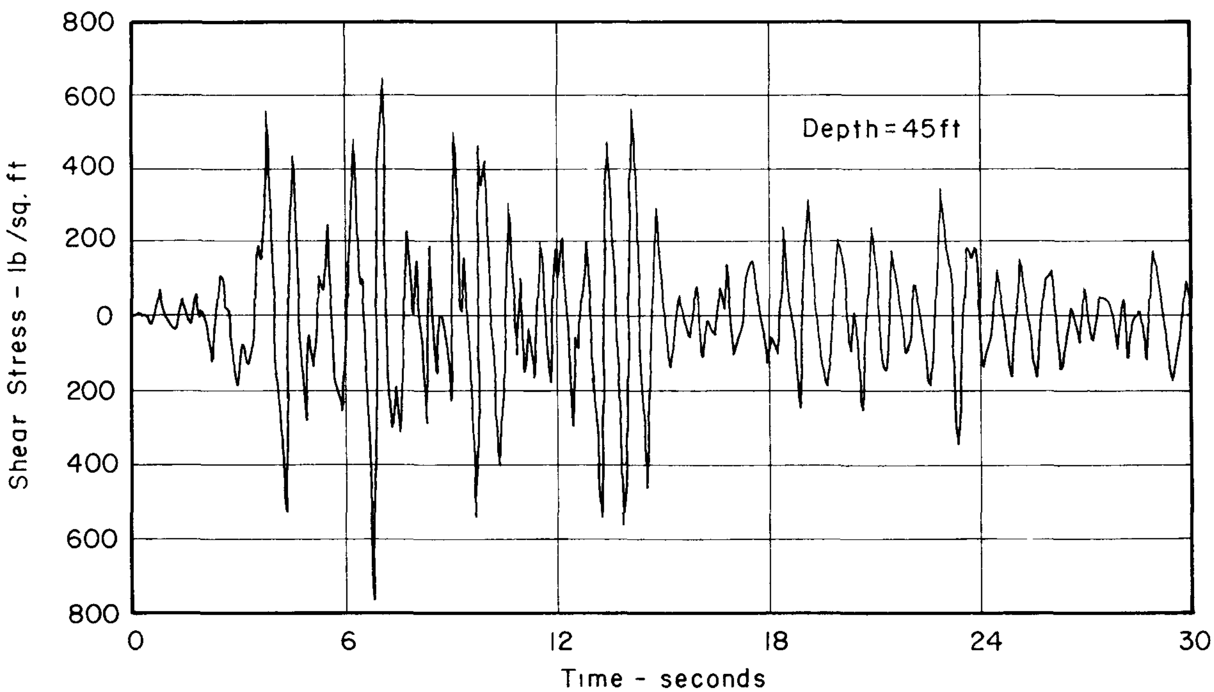
1.2 CAUSES OF LIQUEFACTION

It is now generally recognized that the basic cause of liquefaction of saturated cohesionless soils during earthquakes is the build-up of excess hydrostatic pressures due to the application of cyclic shear stresses induced by the ground motions. These stresses are generally considered to be due primarily to upward propagation of shear waves in a soil deposit, although other forms of wave motions are also expected to occur. Thus, soil elements can be considered to undergo the series of cyclic stress conditions illustrated in Fig. 1-1a, the stress series being somewhat random in pattern but nevertheless cyclic in nature as shown in Fig. 1-1b.

As a consequence of the applied cyclic stresses, the structure of the cohesionless soil tends to become more compact with a resulting transfer of stress to the pore water and a reduction in stress on the soil grains. As a result, the soil grain structure rebounds



a) IDEALIZED FIELD LOADING CONDITIONS



b) SHEAR STRESS VARIATION DETERMINED BY RESPONSE ANALYSIS

FIG. I-1 CYCLIC SHEAR STRESSES ON A SOIL ELEMENT DURING GROUND SHAKING

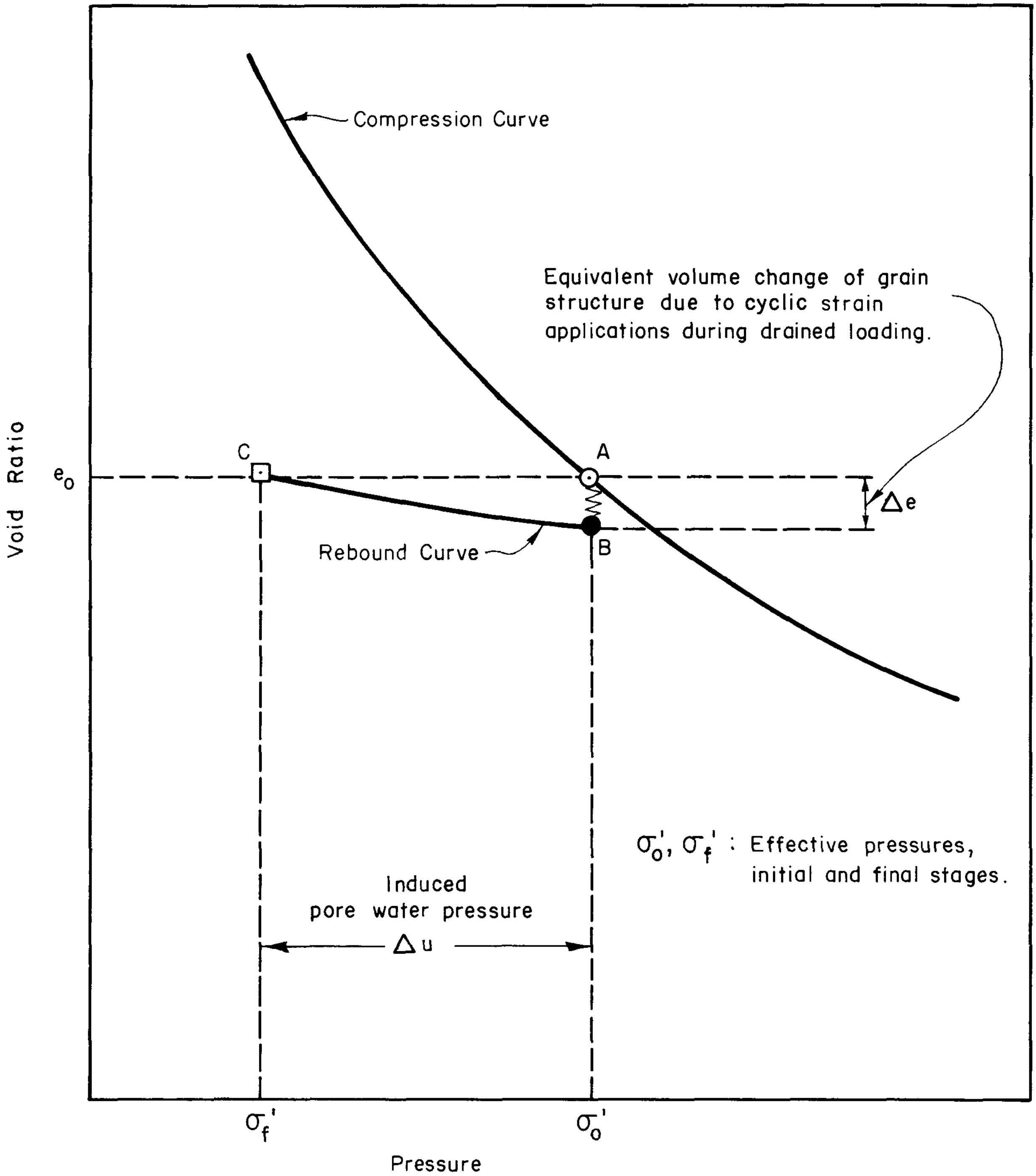


FIG. I-2 SCHEMATIC ILLUSTRATION OF MECHANISM OF PORE PRESSURE GENERATION DURING CYCLIC LOADING to the extent required to keep the volume constant, and this interplay of volume reduction and soil structure rebound determines the magnitude of the increase in pore water pressure in the soil. The basic phenomenon is illustrated schematically in Fig. 1-2. The mechanism can be quantified so that the pore pressure increases due to any given sequence of stress applications can be computed from a knowledge of the stress-strain characteristics, the volume change characteristics of the dry sand subjected to cyclic strain conditions, and the rebound characteristics of the sand due to stress reduction (Martin, Finn, and Seed, 1975; Seed and Pyke, 1975).

As the pore water pressure approaches a value equal to the applied confining pressure, the sand begins to undergo deformations. If the sand is loose, the pore pressure will increase suddenly to a value equal to the applied confining pressure, and the sand will rapidly begin to undergo large deformations with shear strains which may exceed $\pm$ 20 percent or more. If the sand will undergo unlimited deformations without mobilizing significant resistance to deformation, it can be said to be liquefied. If, on the other hand, the sand is dense, it may develop a residual pore water pressure, on completion of a full stress cycle, which is equal to the confining pressure (a condition of initial liquefaction), but when the cyclic stress is reapplied on the next stress cycle, or if the sand is subjected to monotonic loading, the soil will tend to dilate, the pore pressure will drop if the sand is undrained, and the soil will ultimately develop enough resistance to withstand the applied stress. However, it will have to undergo some degree of deformation to develop the resistance, and as the cyclic loading continues, the amount of deformation required to produce a stable condition may increase. Ultimately, however, for any cyclic loading condition, there appears to be a cyclic strain level at which the soil will be able to withstand any number of cycles of a given stress without further deformation. This type of behavior is called "cyclic mobility" or "initial liquefaction with a limited strain potential". It should be noted, however, that once the cyclic stress applications stop, if they return to a zero stress condition, there will be a residual pore water pressure on the soil equal to the overburden pressure, and this will inevitably lead to an upward flow of water in the soil which could have deleterious consequences for overlying layers.

In fact, the upward flow of water to the ground surface from an underlying layer in which a condition of initial liquefaction has been produced by the earthquake ground motions may well be the cause of the surface manifestations of liquefaction, such as sand boils, a "quick" condition or a general condition of water seepage causing inundation, which can cause major damage to structures supported on the near-surface soils (Ambraseys and Sarma, 1969; Yoshimi and Kuwarbara, 1973; Seed, Martin, and Lysmer, 1975). This is illustrated by the analytical results shown in Figs. 1-3a and 1-3b for a soil profile closely simulating the conditions in Niigata, Japan, during the earthquake of 1964 (after Seed, Martin, and Lysmer, 1975). Initial liquefaction is indicated in Fig. 1-3b to have developed between depths of 15 to 40 feet during the 50-second duration of earthquake shaking, with initial liquefaction at depths of ten, three, and one foot occurring at times of about 3, 4, and 13 minutes, respectively, after the ground motions had stopped. Such results are in general accord with observations of the sequence of sand boil development and water flow at the ground surface in this earthquake and illustrate the importance of tracing the time history of pore pressure changes in sand layers, both during and following an earthquake. In some cases, the earthquake-induced pore water pressures may dissipate so rapidly that a liquefied condition could not possibly develop, while in others, the high pore water pressures accompanying the development of initial liquefaction or cyclic mobility may themselves lead to a loss of strength in overlying soil deposits.

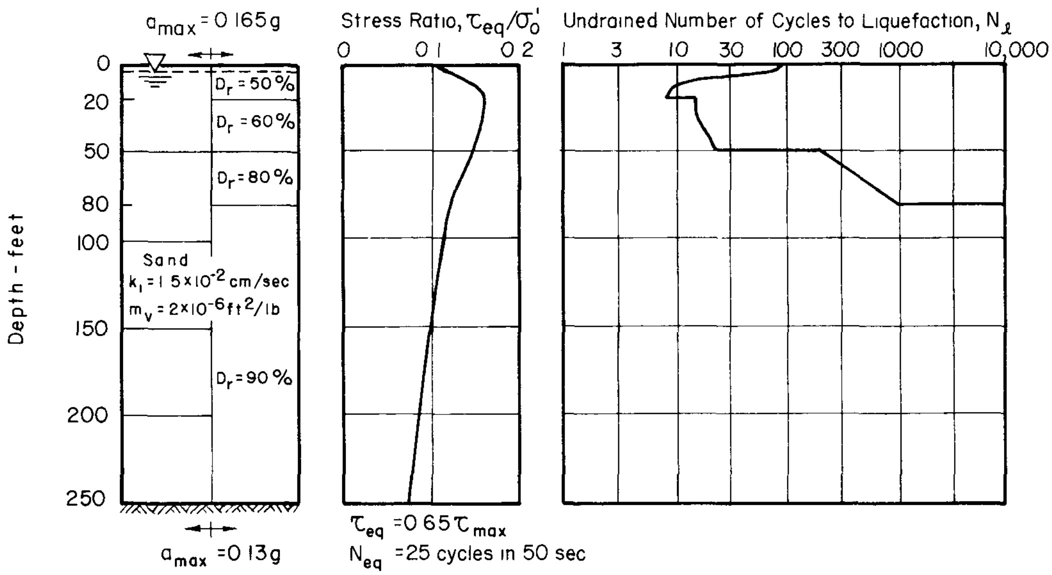


FIG. 1-3a SOIL PROFILE AND STRESS CONDITIONS USED FOR ANALYSIS

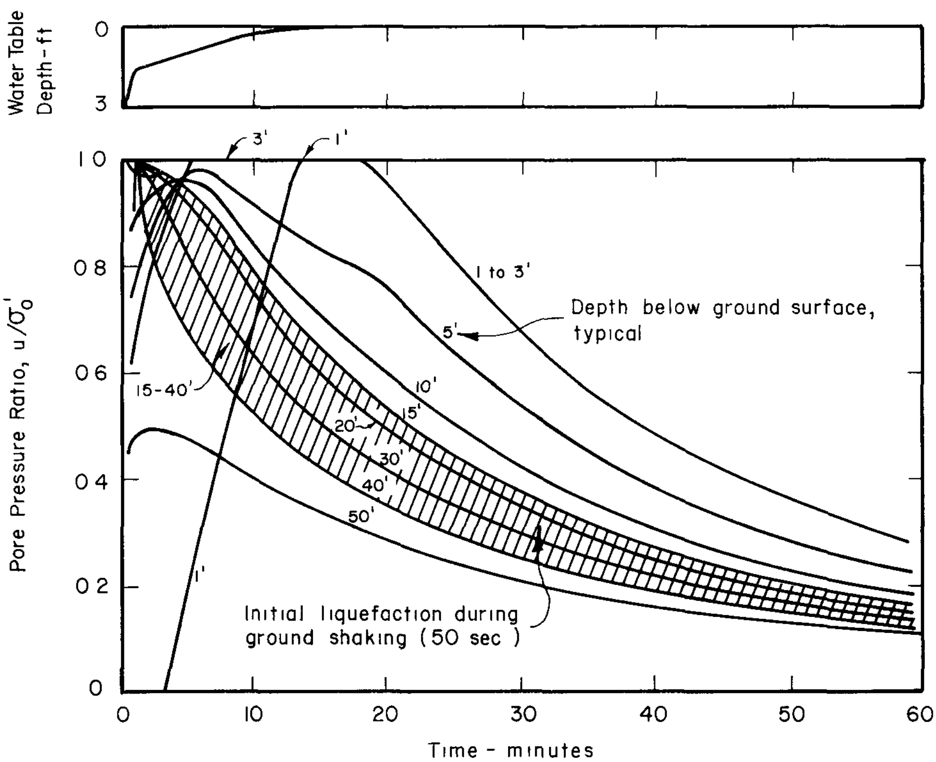


FIG. 1-3b COMPUTED VARIATION OF PORE WATER PRESSURE IN 60 MINUTE PERIOD FOLLOWING EARTHQUAKE FOR SOIL PROFILE

FIG. 1-3 Niigata Earthquake (1964): ANALYSIS INDICATING THE PROGRESSIVE ADVANCE OF INITIAL LIQUEFACTION FROM DEEP TO SHALLOW STRATA

(After Seed, Martin and Lysmer, 1975)

It is apparent that the deleterious effects of pore pressure dissipatation following initial liquefaction of underlying soil layers cannot occur if the underlying layers do not first develop high pore water pressures such as those accompanying initial liquefaction. Furthermore, if dissipation of water pressures in pervious soils will help to prevent the build-up of pore pressures sufficiently large to produce initial liquefaction in such materials, it is conservative to ignore this effect and assume that all sand layers are essentially undrained during earthquake shaking. Accordingly, these principles have been the basic premises for virtually all analyses of possible soil liquefaction effects at sites of critical structures. If, under undrained conditions, it can be shown that any soil layer in a profile has an adequate margin of safety against liquefaction, initial liquefaction, or cyclic mobility, then no further studies of pore pressure dissipation effects have been considered warranted. This approach is both reasonable from a safety point of view and, it has been necessary, from a practical point of view, since methods of evaluating the rate of pore pressure build-up and dissipation, both during and following earthquakes, have only recently become available (Seed, Martin, and Lysmer, 1975; Martin, Finn, and Seed, 1975). Thus, the present state of practice is to analyze the liquefaction potential of all soil layers in a profile and demonstrate an adequate margin of safety against any form of liquefaction or cyclic mobility. Methods of accomplishing this are discussed below.

1.3 METHODS FOR EVALUATING THE LIQUEFACTION POTENTIAL OF SAND DEPOSITS

There are basically two methods available for evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking.

1.3.1 Method Based on Observations of Performance of Sand Deposits in Previous Earthquakes

It was not until the Alaska and Niigata earthquakes of

1964 that geotechnical engineers took serious interest in the general phenomenon of earthquake-induced liquefaction or the conditions responsible for causing it to occur in the field. Following the Niigata earthquake, a number of Japanese engineers (Kishida, Koizumi, Ohsaki) studied the areas in Niigata where liquefaction had and had not occurred and developed criteria, based primarily on the Standard Penetration Resistance of the sand deposits, for differentiating between liquefiable and non-liquefiable conditions in that city. For example, Kishida presented data to show that liquefaction-induced settlement of foundations of buildings was invariably minor when the Standard Penetration Resistance, N, of the sand at the base of the foundation exceeded 20 blows per foot. Kishida (1966), Koizumi (1966), and Ohsaki (1966) presented the results shown in Fig. 1-4 separating liquefiable and non-liquefiable conditions.

Subsequently, a more comprehensive collection of site conditions at various locations where liquefaction or no liquefaction was known to have taken place was presented by Seed and Peacock (1971) and used as a basis to determine the relationship between field values of cyclic stress ratio, τ_h/σ'_o (where τ_h or simply τ is the average horizontal shear stress induced by an earthquake, and σ'_o is the effective overburden pressure on the soil layer involved) and the relative density of the sand, as determined from the Standard Penetration Resistance and its correlation with relative density proposed by Gibbs and Holtz (1957). This collection of field cases has subsequently been used by others, often supplemented by a few additional site studies (e.g., Castro, 1975) to determine other correlations between liquefaction-producing parameters and penetration resistance. The most recent form of this data collection is shown in Table 1-1 and Fig. 1-5 (after Seed, Mori, and Chan, 1975). Values of stress ratio known to be associated with liquefaction or no liquefaction in the field are plotted as a function of the corrected average penetration resistance N_1 of the sand deposit involved. In this form of presentation N_1 is the measured penetra-

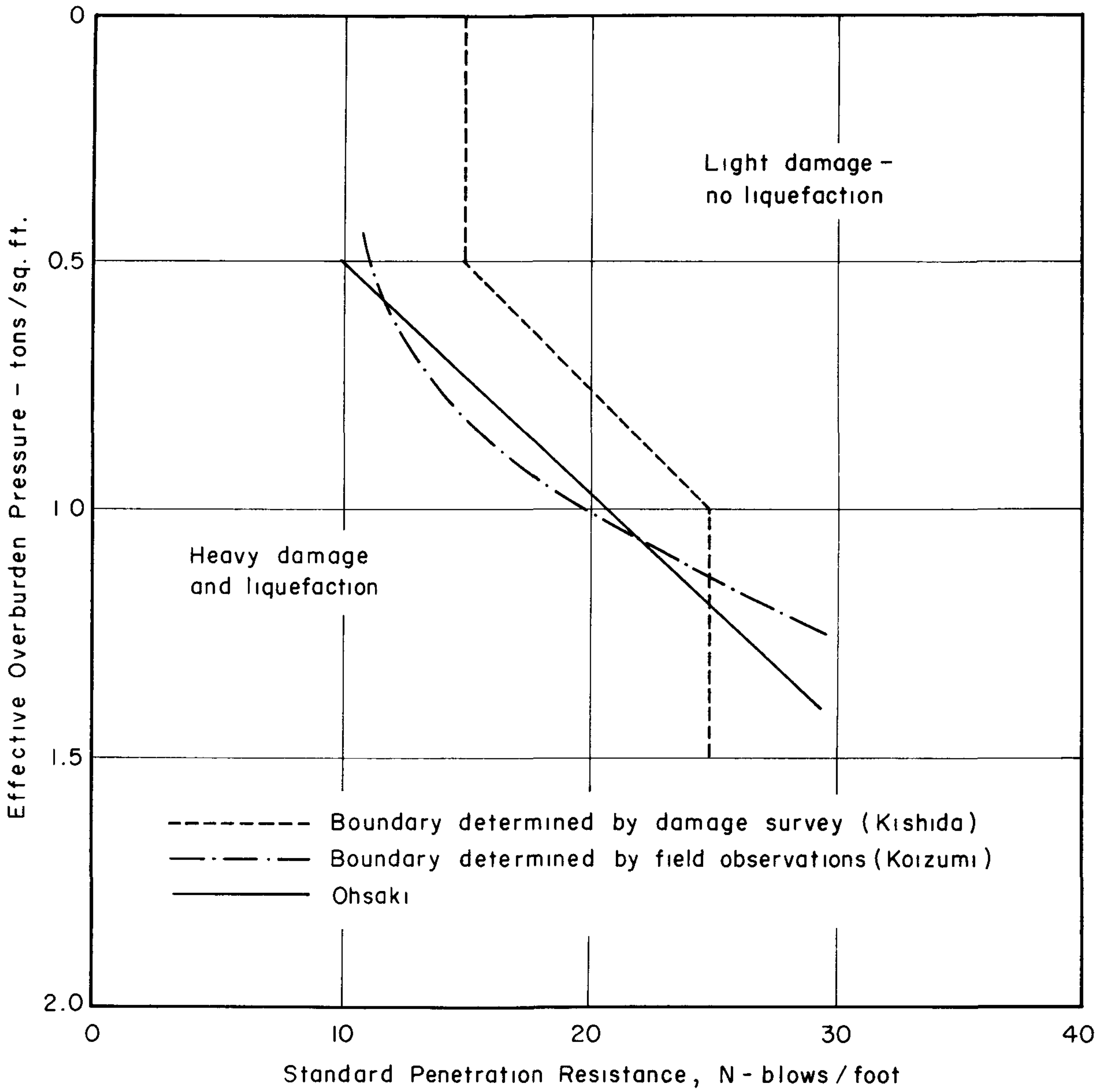


FIG. I-4 ANALYSIS OF LIQUEFACTION POTENTIAL AT NIIGATA FOR EARTHQUAKE OF JUNE 16, 1964

TABLE 1-1

Site Conditions and Earthquake Data for Known Cases of Liquefaction and Non-Liquefaction

Earthquake	Date	Magnitude	Site	Distance from source of energy release (miles)	Soil Type	Depth of water table (feet)	Critical depth (feet)	Effective overburden pressure, p.s.f. σ'_o	Correction factor for penetration resist., C_N	Av. penetration resist. at critical depth N	Corrected av. penetration resistance N_1	Relative density (%)	Max. ground surface acceleration (g)	$\frac{\tau}{\sigma'_o}$	Duration of shaking (secs)	Field Behavior	Reference
Niigata	1802	6.6	Niigata	24	Sand	3	20	1100	1.35	6	8	53	0.12	0.135	20	No liquefaction	Kawasumi (1968)
Niigata	1802	6.6	Niigata	24	Sand	3	20	1100	1.35	12	16	64	~0.12	0.135	~20	No liquefaction	Seed & Idriss (1967)
Niigata	1887	6.1	Niigata	29	Sand	3	20	1100	1.35	6	8	53	~0.08	0.09	~12	No liquefaction	Seed & Idriss (1967)
Niigata	1887	6.1	Niigata	29	Sand	3	20	1100	1.35	12	16	64	0.08	0.09	~12	No liquefaction	Seed & Idriss (1967)
Mino Owari	1891	8.4	Ogaki	20	Sand	3	45	2600	0 85	17	15	65	0.35	0.39	~75	Liquefaction	Kishida (1969)
Mino Owari	1891	8.4	Ginan West	20	Sand	6	30	2000	1.0	10	10	55	~0.35	0.37	~75	Liquefaction	Kishida (1969)
Mino Owari	1891	8.4	Unuma	20	Sand & Gravel	6	25	1700	1.1	19	21	75	0.35	0.35	~75	No liquefaction	Kishida (1969)
Mino Owari	1891	8.4	Ogase Pond	20	Sand	8	20	1500	1.17	16	19	72	~0.35	0.35	~75	Liquefaction	Kishida (1969)
Santa Barbara	1925	6.3	Sheffield Dam	7	Sand	~15	25			-		40	~0.2	0 16	15	Liquefaction	Seed et al (1959)
El Centro	1940	7.0	Brawley	5	Sand	15	~15			-		58	~0.25	0.155	30	Liquefaction	Ross (1968)
El Centro	1940	7.0	All-Am. Canal	5	Sand	~20	~25			-		43	~0.25	0 155	30	Liquefaction	Ross (1968)
El Centro	1940	7.0	Solfatara Canal	5	Sand	5	20			-		32	~0.25	0.26	30	Liquefaction	Ross (1968)
Tohnankai	1944	8.3	Komei	100	Sand	5	13	1000	1.4	4	6	40	~0.08	0.08	~70	Liquefaction	Kishida (1969)
Tohnankai	1944	8.3	Meiko St.	100	Silt & Sand	2	8	500	1.75	1	2	30	0.08	0.09	~70	Liquefaction	Kishida (1969)
Fukui	1948	7.2	Takaya	4	Sand	11	23	1900	1.02	18	18	72	~0.30	0.30	~30	Liquefaction	Kishida (1969)
Fukui	1948	7.2	Takaya	4	Sand	3	23	1400	1.2	28	34	90	~0.30	0.32	~30	No liquefaction	Kishida (1969)
Fukui	1948	7.2	Shonenji Temple	4	Sand	4	10	750	1.5	3	5	40	~0.30	0.29	≈30	Liquefaction	Kishida (1969)
Fukui	1948	7.2	Agr. Union	4	Sand & Silt	3	20	1200	1.3	5	7	50	0.30	0 33	≈30	Liquefaction	Kishida (1969)
San Francisco	1957	5.5	Lake Merced	4	Sand	8	10	1000	1 4	7	10	55	0.18	0.13	18	Liquefaction	Ross (1968)
Chile	1960	8.4	Puerto Montt	~70	Sand	12	15	1500	1.17	6	7	50	~0.15	0.15	~75	Liquefaction	Lee (1970)
Chile	1960	8.4	Puerto Montt	~70	Sand	12	15	1500	1.17	8	10	55	~0.15	0.15	~75	Liquefaction	Lee (1970)
Chile	1960	8.4	Puerto Montt	~70	Sand	12	20	1800	1.05	18	19	75	~0.15	0.15	≈75	No liquefaction	Lee (1970)
Niigata	1964	7.5	Niigata	32	Sand	3	20	1200	1.3	6	8	53	0.18	0.195	40	Liquefaction	Seed & Idriss (1967)
Niigata	1964	7.5	Niigata	32	Sand	3	25	1500	1.17	15	18	70	0.18	0.195	40	Liquefaction	Kishida (1966)
Niigata	1964	7.5	Niigata	32	Sand	3	20	1200	1.3	12	16	64	0.18	0.195	40	No liquefaction	Seed & Idriss (1967)
Niigata	1964	7.5	Niigata	32	Sand	12	25	2000	1 0	6	6	53	0.18	0.12	40	No liquefaction	Seed & Idriss (1967)
Alaska	1964	8.3	Snow River	60	Sand	0	20	1100	1.35	5	7	50	~0.15	0.18	180	Liquefaction	Ross et al (1969)
Alaska	1964	8.3	Snow River	60	Sand	8	20	1500	1.17	5	6	44	~0.15	0.15	180	Liquefaction	Ross et al (1969)
Alaska	1964	8.3	Quartz Creek	70	Sandy Gravel	0	~25	1500	1.17	40-80	46-95	100	≈0.12	0.145	180	No liquefaction	Ross et al (1969)
Alaska	1964	8.3	Scott Glacier	55	Sand	0	≈20	1100	1.35	10	14	65	~0.16	0.185	180	Liquefaction	Ross et al (1969)
Alaska	1964	8.3	Valdez	35	Sand & Gravel	5	~20	1300	1.25	13	16	68	≈0.25	0.25	180	Liquefaction	Coulter & Migliaccio (1966)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	3	12	800	1.5	14	21	78	0.21	0.23	45	No liquefaction	Ohsaki (1970)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	3	12	800	1.5	6	9	58	0.21	0.23	45	Liquefaction	Ohsaki (1970)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	5	10	800	1.5	15	23	80	0.21	0.185	45	No liquefaction	Ohsaki (1970)
Tokachioki	1968	7.8	Hakodate	100	Sand	3	15	1000	1.4	6	9	55	0.18	0.205	45	Liquefaction	Kishida (1970)
Caracas	1967	6.3	Caraballeda	35	Sand	3	3	330	1.8	3	5	60	0.13	0.085	15	Liquefaction	Cluff (1973)
San Fernando	1971	6.6	SFo.Juvenile Hall	5	Sandy Silt	15	20	2000	1.0	2	2	30	0.40	0.28	15	Liquefaction	Seed (1973)
San Fernando	1971	6.6	Jensen Plant	5	Silty Sand	55	55	6600	0.45	24	11	58	0.35	0.16	15	Liquefaction	Dixon & Burke (1973)

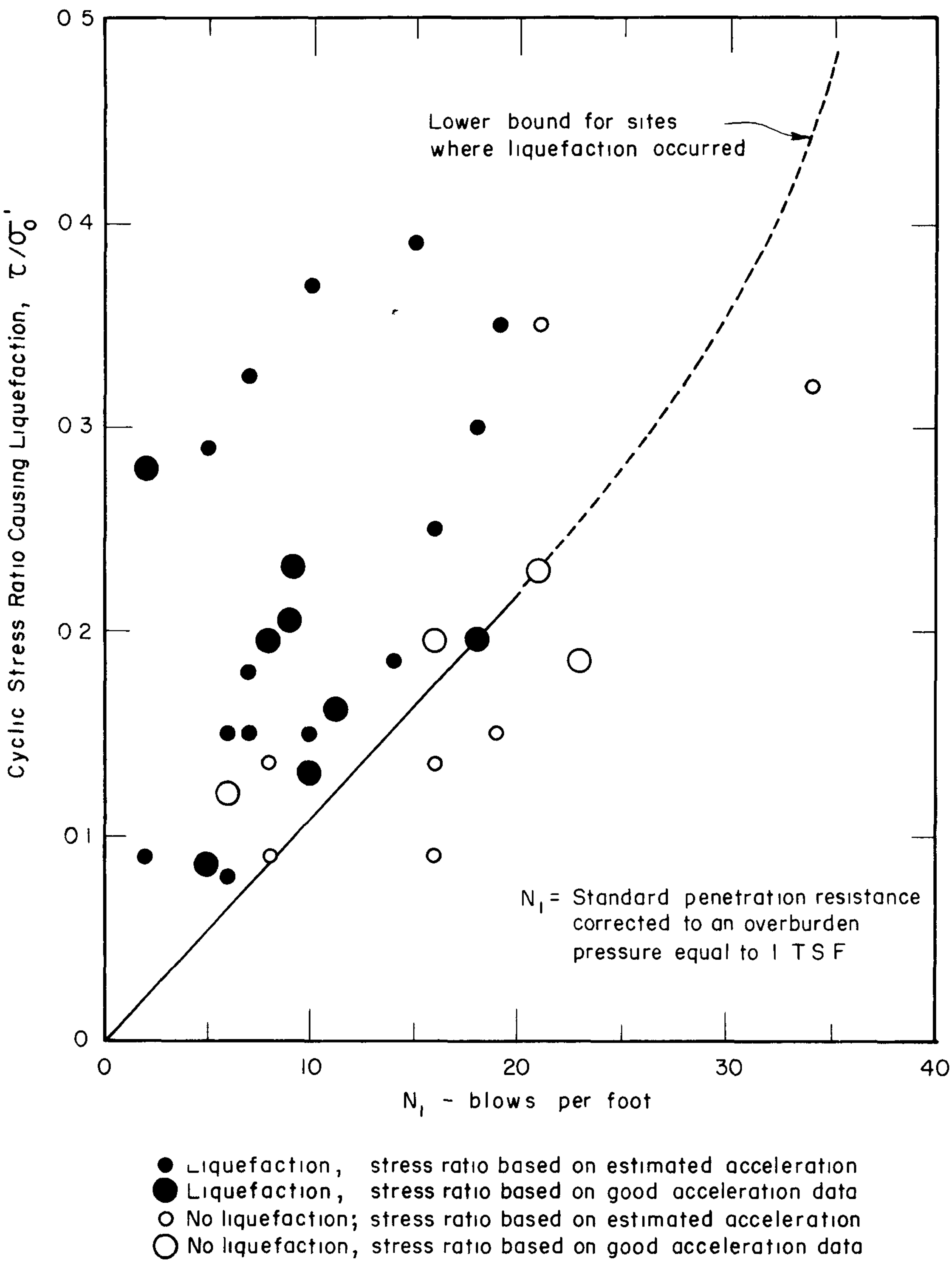


FIG. 1-5 CORRELATION BETWEEN STRESS RATIO CAUSING LIQUEFACTION IN THE FIELD AND PENETRATION RESISTANCE OF SAND

(After Seed, Mori and Chan, 1975)

tion resistance corrected to an effective overburden pressure of one ton per square foot, based on the results of Gibbs and Holtz, using the relationship:

$$N_1 = C_N \cdot N$$

where $C_N = 1 - 1.25 \log \frac{\sigma'_o}{\sigma'_1}$

σ'_o = effective overburden pressure in tons per square foot where the penetration resistance has the value N

and σ'_1 = one ton per square foot

The cyclic stress ratio causing liquefaction can be determined from the relationship:

$$\frac{\tau_{av}}{\sigma'_o} \approx 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma'_o} \cdot r_d$$

where a_{max} = maximum acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ'_o = effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of one at the ground surface to a value of 0.9 at a depth of 30 feet.

Thus for any given value of maximum ground surface acceleration, the possibility of liquefaction can readily be obtained on an empirical basis with the aid of this chart. It may be noted that there is a scarcity of reliable data at high values of τ/σ'_o, and there is a need for supplementary data to better define the lower bound of liquefaction conditions in this range.

Since empirical charts of this type take no account of other significant factors such as the duration of shaking or the possibility of drainage and depend upon the reliability of field measurements of penetration resistance, which in the opinion of many engineers is open to serious question, it appears to be the general belief among most engineers that while they can provide useful preliminary evaluations of liquefaction potential, they should be supplemented by detailed studies, based on ground response analyses and detailed soil testing programs, in order to arrive at a meaningful evaluation of the liquefaction potential of any particular site.

1.3.2 Method Based on Evaluation of Stress Conditions in the Field and Laboratory Determinations of the Stress Conditions Causing Liquefaction of Soils

Analytical procedures for evaluating the liquefaction potential of soil deposits were first proposed by Seed and Idriss (1967) and involve two independent determinations: 1) an evaluation of the cyclic stresses induced at different levels in the deposit by the earthquake shaking and 2) a laboratory investigation to determine the cyclic stresses, which for given confining pressures representative of specific depths in the deposit, will cause the soil to liquefy or undergo various degrees of cyclic strain. As shown in Fig. 1-6, the evaluation of liquefaction potential is then based on a comparison of the cyclic stresses induced in the field with the stresses required to cause liquefaction or an acceptable limit of cyclic strain in representative samples in the laboratory.

The cyclic stresses induced in the ground by an earthquake may be computed by a ground response analysis (Seed and Idriss, 1967), by a simplified procedure based on a knowledge of the maximum ground surface acceleration (Seed and Idriss, 1971), or by deconvolution of a known ground surface motion (Schnabel, Lysmer, and Seed, 1972; Roesset and Whitman, 1969). The computed irregular time history of stresses at any depth is then converted to an equivalent uniform cyclic stress series by an appropriate weighing procedure (Seed, Idriss, Makdisi, and Banerjee, 1975) for use in the analysis.

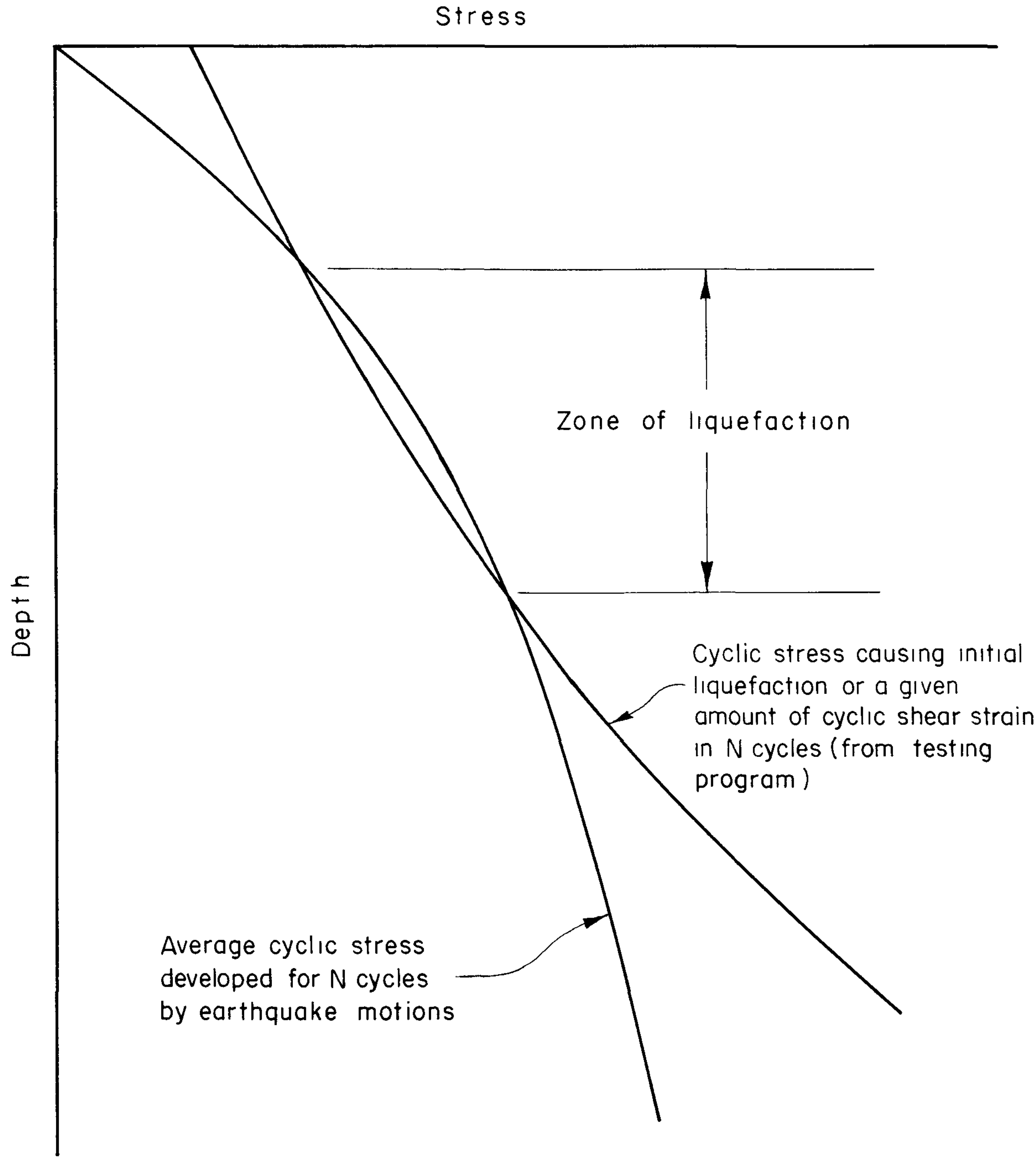


FIG. I-6 METHOD OF EVALUATING LIQUEFACTION POTENTIAL

Various types of laboratory test procedures have been used to investigate the cyclic stress conditions required to cause liquefaction or cyclic mobility of saturated sand. Since the object of the test is to reproduce the stresses acting on an element of sand subjected to horizontal shear stresses which reverse direction many times during an earthquake, some form of simple shear test provides the best representation of field conditions. Tests of this type used for this purpose include cyclic simple shear tests (Peacock and Seed, 1968; Finn, Bransby, and Pickering, 1970; Seed and Peacock, 1971) and cyclic torsional shear tests (Yoshimi and Oh-Oka, 1973; Ishihara and Li, 1972; Drnevich, 1972; Ishibashi and Sherif, 1974). The results of these tests can be expressed directly in terms of the relationship between the cyclic stress ratio τ_h/σ'_o and the number of stress cycles required to cause initial liquefaction or a given degree of cyclic strain. Attempts have also been made to use shaking table tests for this purpose (Yoshimi, 1967; Tanimoto, 1967; Whitman, 1970; Finn, Emery, and Gupta, 1970), but these have often been influenced by the confining effects of the sides of the box and have not reproduced the desired boundary conditions in many cases.

Equipment for conducting any type of simple shear test is somewhat complicated, and to provide a practical and convenient alternative, the cyclic loading triaxial test was developed by Seed and Lee (1966). This test does not reproduce the correct initial stress conditions in the ground; it must be performed with an initial ambient pressure condition to represent level ground conditions, and the stress ratio used to express the results ($\sigma_{dc}/2\sigma_3$) is the ratio of the maximum shear stress to the ambient pressure, rather than the shear stress on the horizontal plane to the effective overburden pressure (τ/σ'_o), as used in the cyclic simple shear test. For these reasons alone, the stress ratios causing initial liquefaction or given cyclic strains in the two types of test will necessarily be different, and they are usually related by the expression (Seed and Peacock, 1971):

$$\left(\frac{\tau}{\sigma'_o}\right)_{field} = \alpha \left(\frac{\sigma_{dc}}{2\sigma_3}\right)_{triaxial}$$

While some engineers apparently believe that cyclic triaxial test data are too low because of the stress conditions involved (Ambraseys, 1973), most investigators have concluded that the value of α is substantially less than one for normally consolidated sands. Thus, the following values have been proposed:

Peacock and Seed (1968)	$\alpha \approx 0.55$ for $k_o \approx 0.4$
Finn, Bransby, and Pickering (1970)	$\alpha = \dfrac{1 - 2K_o}{3} \approx 0.6$ for $K_o = 0.4$
Seed and Peacock (1971)	α = varies from 0.55 to 0.72 depending upon relative density, $K_o = 0.4$
Castro (1975)	$\alpha = \dfrac{2(1 + 2K_o)}{3\sqrt{3}} = 0.7$ for $K_o = 0.4$

and values of α ranging from about 0.55 to 0.7 have been used as correction factors for triaxial test data to obtain stress ratios representative of field simple shear conditions.

It has been generally recognized since the advent of cyclic load testing that virtually all types of cyclic load tests are subject to some degree of error due to equipment limitations (Seed and Peacock, 1971; Finn, Emery, and Gupta, 1970; Castro, 1969). However, with due allowances made for these effects, it has been considered that test data could be obtained from these various tests with an adequate degree of accuracy to provide a useful basis for liquefaction potential evaluations for most practical purposes.

Nevertheless, certain aspects of the test procedures have remained a matter of concern; these include:

a. The argument that stress concentrations in small-scale simple shear tests lead to inaccurate results (Castro, 1969).

b. The argument that simple shear tests or torsional shear tests produce deformations in only one direction and do not reproduce the effects of multi-directional straining such as occurs in the field (Casagrande, 1971; Shannon & Wilson, Inc., 1971, 1972).

c. The argument that the boundary conditions in shaking table tests, with vertical boundaries preventing or restricting the movement of test samples, do not reproduce the deformation conditions existing in the field (Shannon & Wilson, Inc., 1972).

d. The argument that many shaking table tests have been conducted without preventing drainage so that pore pressures can dissipate by flow to the surface of the sample, thereby preventing liquefaction from developing as rapidly as it would under undrained conditions (Shannon & Wilson, Inc., 1971, 1972).

e. The argument that cyclic triaxial tests, because of stress concentrations introduced by the cap and base and the possibility of necking in the extension stage of the stress cycle, develop non-uniformities of strain and a redistribution of water content which lead to an underestimate of the ability of medium dense to dense sands to withstand cyclic loading (Castro, 1975; Casagrande, 1971).

These are valid points of concern and will be addressed in detail in the following sections of this report.

Apart from possible limitations of test equipment and procedures, another important aspect of a cyclic load test program for use in design concerns the selection of representative samples for testing purposes. In the early stages of cyclic load testing, it was generally recognized that the liquefaction characteristics of any given sand varied greatly depending on its density or relative density, but the possible effects of other factors, such as geologic history, soil structure, or method of sample preparation, were not considered likely to affect the results significantly. Nevertheless, many engineers adopted a policy of testing undisturbed samples to ensure that such factors were properly considered in the results obtained. This inevitably raises the question of the ability of existing sampling procedures to obtain good quality undisturbed samples of sand and the possible errors introduced if samples are disturbed to some extent in the sampling and handling process. In fact, it seems likely that procedures vary widely in their adequacy in this respect, and the nature of the sampling process requires careful evaluation in assessing the quality of test data obtained from the resulting samples.

The importance of factors other than density on the liquefaction characteristics of sand was first noted by Finn, Bransby, and Pickering (1970), who showed, by means of simple shear tests on small-scale samples of saturated sand, that the liquefaction characteristics were influenced by the strain history to which they had been subjected and concluded: "The dependence of the resistance to liquefaction of a given sand on its previous strain history leads to the conclusion that the resistance of sand deposits in the field cannot be reliably determined by cyclic loading tests on sand samples prepared in the laboratory at the same void ratio as those in the field. It appears that the resistance to liquefaction can only be reliably determined on undisturbed samples."

This significant observation seems to have been overlooked in many subsequent studies, perhaps partly because the extraction of undisturbed samples from the ground is often extremely difficult, and reconstitution presents an attractive alternative, and partly because of the difficulty encountered by many soil engineers in visualizing that the same set of bulky particles at the same void ratio could have significantly different structures, at least sufficiently different to have significant effects on soil properties of primary interest. In recent years, however, this latter belief has been dispelled by studies showing that samples of a given sand prepared to the same density by different methods of compaction may have quite different settlement characteristics (Pyke, 1973), different liquefaction characteristics (Ladd, 1974; Marcuson and Townsend, 1974; Mulilis, Chan, and Seed, 1975), different structures (Oda, 1972; Mulilis, Chan, and Seed, 1975), and different penetration resistances (Mitchell and Durgunoglu, 1975). Thus, although none of these studies were directly concerned with the effects of strain history on liquefaction characteristics, the potential significance of this factor in producing changes in both structure and liquefaction characteristics has been given important support and clearly warrants consideration in any design study of liquefaction potential.

In view of the need to determine the reliability of current test procedures for evaluating liquefaction characteristics and the desirability of clarifying the influence of strain history, method of sample preparation, and soil structure on the liquefaction characteristics, no matter what test procedure is used for their measurement, detailed studies of these aspects of the problem have been undertaken during the past three years at the University of California, Berkeley, under the sponsorship of the U.S. Nuclear Regulatory Commission. The results of these studies and their significance in design are reviewed in detail in the following pages.

CHAPTER 2

INFLUENCE OF SEISMIC HISTORY AND SOIL STRUCTURE ON LIQUEFACTION CHARACTERISTICS

2.1 LARGE-SCALE TEST EQUIPMENT FOR LIQUEFACTION STUDIES

Since one of the objections raised against existing cyclic simple shear test data has been the possibility of effects of non-uniform stress conditions being introduced due to the small size of the test specimens involved, it was considered desirable to eliminate this possibility by constructing a shaking table facility capable of inducing cyclic stresses on large-scale samples under simple shear conditions. The details of the equipment and test results are described by De Alba (1975). A schematic drawing of the test specimen and apparatus arrangement is shown in Fig. 2-1. Samples are 90 inches long by 42 inches wide and 4 inches thick. These dimensions were selected to provide essentially free-field type conditions in the center portion of the specimen (Arango and Seed, 1974), and pore pressure measurements at different points in the sample showed this to be the case.

Shear stresses are applied by accelerating the base back and forth while a heavy reaction mass is resting on the top surface of the specimen. This mass is sufficiently flexible to provide a uniform pressure on the top surface of the specimen but rigid enough laterally to serve as an inertial reaction block. In a typical test, the stresses developed in the specimen can readily be controlled, and measurements can be made of the resulting pore water pressures and strains induced in the test specimen. By varying the confining pressure on the sample, conditions representative of different depths in the ground can be developed.

2.2 INVESTIGATION OF INFLUENCE OF SEISMIC HISTORY ON LIQUEFACTION POTENTIAL

By means of the large-scale test equipment, stresses repre-

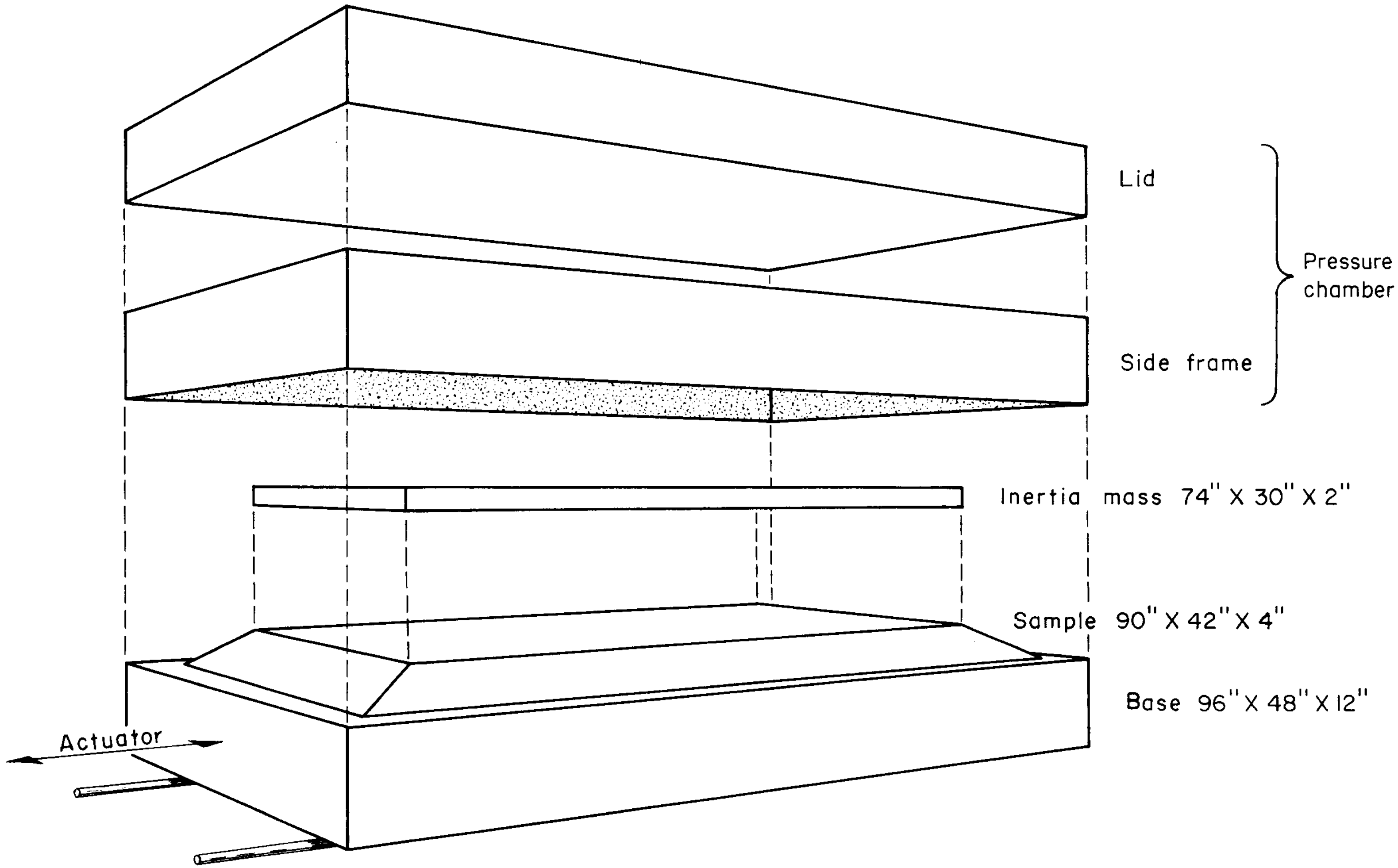


FIG. 2-1 SHAKING TABLE SYSTEM: SCHEMATIC VIEW

sentative of any given earthquake can be induced on a thin layer of sand loaded to represent an elemental layer at various depths in a soil deposit. Accordingly, several investigations were conducted to determine the effect of a series of small earthquakes on the subsequent liquefaction characteristics of a sand deposit. The sand sample was formed by pluvial deposition, which produces a structure and characteristics similar to those of a sedimented deposit. After being saturated, the sand layer was subjected to a series of small shocks designed to represent the effects of a series of small (magnitude $\approx$ 5) earthquakes occurring over a period of years. After each small earthquake, which built-up a small residual pore water pressure in the sand, the pore pressure was allowed to dissipate and the layer to reconsolidate under the initial effective overburden pressure. Finally after five or six such small events, the sand was subjected to a larger shock to determine the stress conditions required to cause it to liquefy. For comparative purposes, the liquefaction characteristics of a similar layer of sand, not previously subjected to the series of small shocks, was also determined.

The results of a typical test are shown in Fig. 2-2. The sample was deposited with a relative density of 54 percent and subjected to a confining pressure representative of that existing at a depth of 15 feet in the ground with a water table four feet below the ground surface. The sand was then subjected to five shocks representative of magnitude five earthquakes occurring at a distance of about five miles. The maximum ground surface acceleration in these shocks was considered to be about 0.18g and the duration to be consistent with the development of 2.5 to 3.0 cycles of motion at an average stress level of about 210 psf. Thus, the cyclic stress ratio for each of these shocks was 0.185.

Previous tests had shown that for the selected test condition, a stress ratio of 0.185 would have caused the sand to develop a condition of initial liquefaction in about four stress

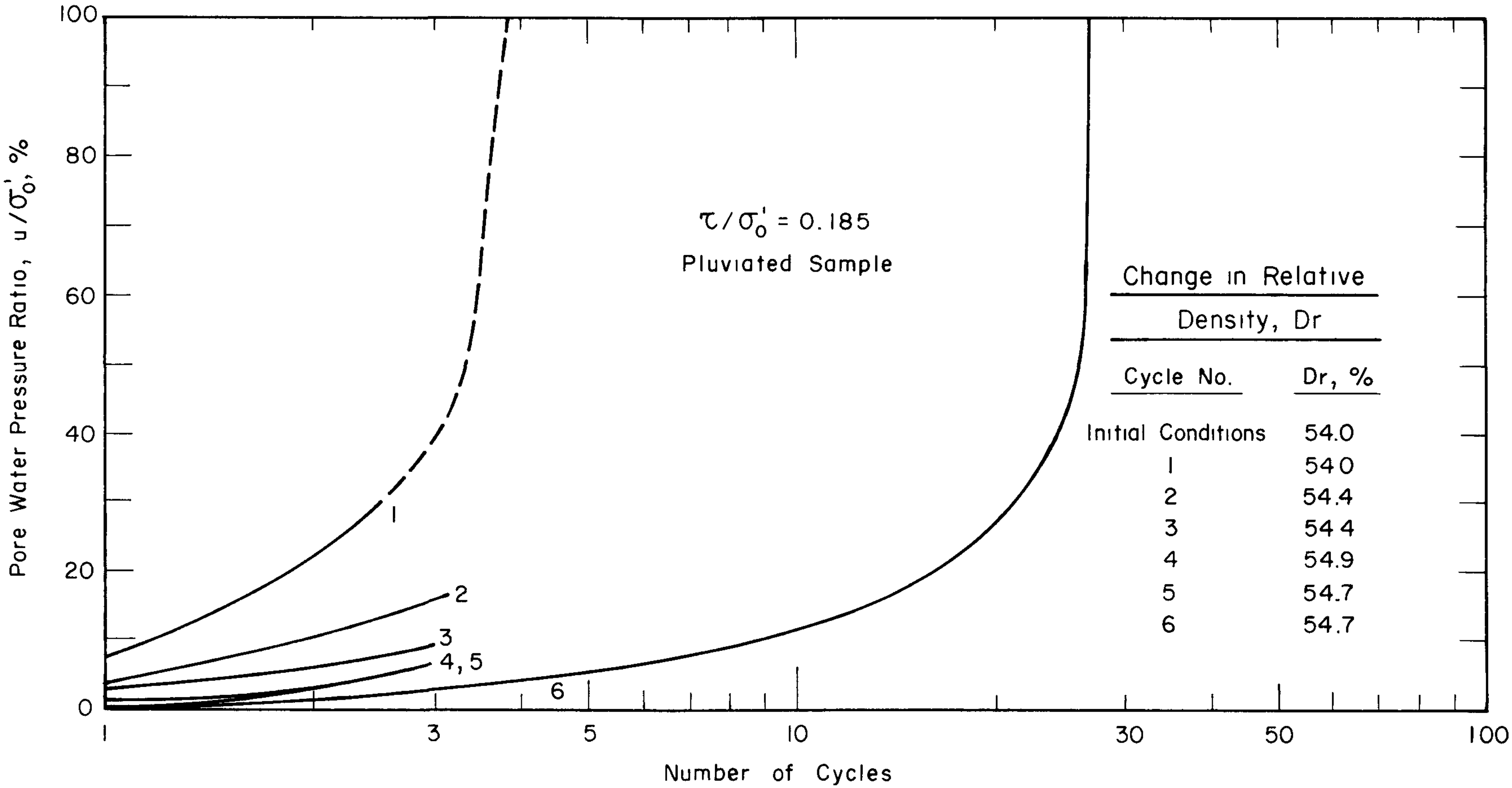


FIG. 2-2 APPLICATION OF MULTIPLE SHAKING AND SUBSEQUENT PORE WATER PRESSURE DISSIPATION TO A SHAKE TABLE SPECIMEN: TEST RESULTS

(After Seed, Mori and Chan, 1975)

cycles. However, the application of only 2.5 cycles simply built up an excess pore pressure ratio (u/σ'_o) of about 0.3 as shown in Fig. 2-2. Dissipation of this pore pressure caused almost no volume change of the sample.

Four subsequent repetitions of this small earthquake stress condition built up excess pore pressure ratios of only 0.16, 0.09, 0.06, and 0.05, respectively, and at the conclusion of this sequence of shocks, the relative density of the sand had increased from its initial value of 54 to 54.7 percent. At this point, the same stress ratio was applied as if it were representing the effects of a magnitude eight earthquake occurring at a distance of about 55 miles and, therefore, capable of producing up to about 30 stress cycles. As may be seen from the figure, the sand liquefied after 26 cycles, but even so, it was able to withstand eight times as many cycles as it could in its initial condition, even though there had been no significant change in relative density.

Two similar series of tests using lower stress ratios showed that the effect of five small earthquake shocks was to increase the number of stress cycles required to cause liquefaction by factors of about ten and eight, respectively.

The relationships between the applied cyclic stress ratio and the number of stress cycles required to cause liquefaction for samples having no previous seismic shaking and the samples subjected to low levels of seismic shocks are compared in Fig. 2-3. It may be seen that the effect of the seismic history to which the sand had been exposed was to increase the resistance to liquefaction considerably. In effect, the samples having a relative density of 54 percent and previous seismic shaking developed a resistance to liquefaction comparable to that of samples having a relative density of about 80 percent and no previous seismic history. In other words, for a given number of

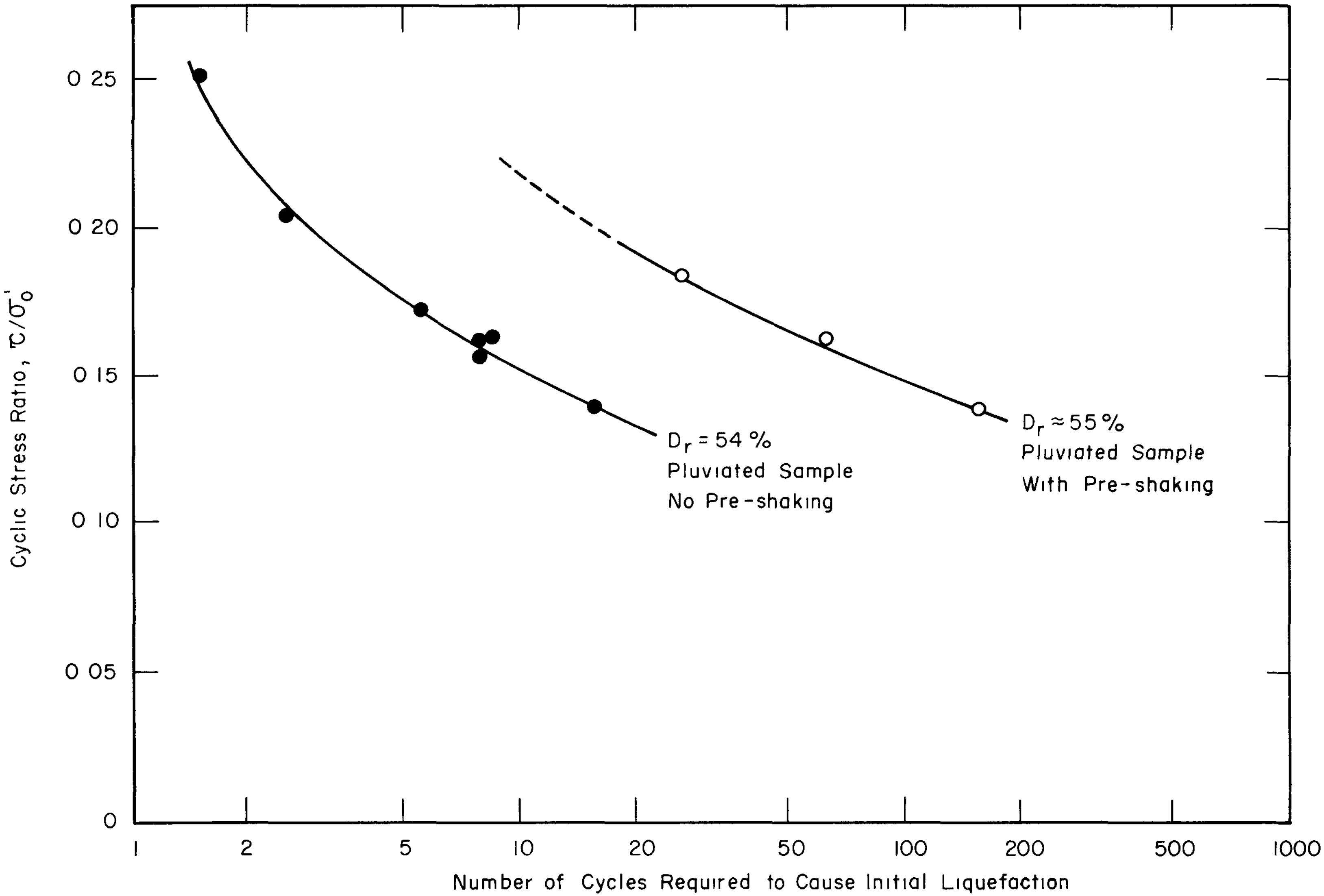


FIG. 2-3 EFFECT OF SEISMIC HISTORY ON LIQUEFACTION CHARACTERISTICS OF SAND

(After Seed, Mori and Chan, 1975)

cycles, the sand layers previously subjected to low levels of seismic shaking required stresses about 46 percent higher than those of samples with the same densities and no previous seismic history.

This is a substantial effect and confirms the conclusion of Finn, Bransby, and Pickering (1970) concerning: 1) the unreliability of relative density alone as a measure of the liquefaction potential of a sand deposit and 2) the need to retain the effects of any previous seismic history on the liquefaction characteristics of natural sand deposits in laboratory determinations of this soil property.

2.3 POSSIBLE CAUSES OF INCREASED RESISTANCE TO LIQUEFACTION RESULTING FROM SEISMIC HISTORY EFFECTS

Several reasons might be advanced for the observed increase in resistance to liquefaction induced by low intensity seismic histories. For example, it has been observed in tests by Youd and Craven (1975) and Pyke (1973) that during cyclic straining of dry sands in simple shear, there is a progressive build-up in the value of the lateral pressure coefficient, K_o. Such increases would lead to an increase in the stress ratio required to cause liquefaction as shown by Seed and Peacock (1971). However, at small strain levels, this effect does not seem likely to be sufficiently large to explain the significant increase in strength observed in the test program.

An alternative explanation is that during any period of cyclic straining, there is a progressive change in the soil structure with the result that the volume change occurring in any one cycle decreases progressively with increasing numbers of cycles. This effect may be observed in cyclic load test data for samples of dry sand. Thus, for example, in a cyclic load test on Monterey sand, the first stress cycle applied to a sample with a relative density of 50 percent causes a settlement of 0.01 percent, but the fifteenth application of the same stress cycle caused a settlement of only about 0.002 percent. Clearly the sand had acquired a more resistant structure in the course of the cyclic straining. This is in accord with the results of other studies which have shown structure to be a potentially significant factor influencing the liquefaction characteristics of sands. Accordingly, a detailed study of the influence of sample preparation method and soil structure on cyclic load test data was undertaken as described below.

2.4 EFFECTS OF METHOD OF SAMPLE PREPARATION ON LIQUEFACTION CHARACTERISTICS

To throw further light on the possible significance of soil structure on liquefaction characteristics and the manner in which both these properties of sands may be influenced by methods of sample preparation, a detailed investigation of the relationship between these factors was conducted by Mulilis, Chan, and Seed, (1975).

In the study, undrained stress-controlled cyclic triaxial tests were performed on saturated samples of sand compacted to the same density by 11 different procedures. A slightly modified form of the standard triaxial test equipment incorporating a pneumatic sinusoidal loading system (Chan, 1975) was used for all tests.

Except for a limited number of tests on soil dredged from the bottom of the Mississippi River described in a subsequent section, all tests were performed on Monterey No. 0 sand. This sand has been tested extensively for static stress-strain properties by Lade (1972), for static compressibility by Mahmood (1973), for settlement under multi-directional loading by Pyke (1973), and for liquefaction characteristics in a large-scale shaking table by De Alba (1975). It is a uniform medium sand, mostly passing the No. 30 sieve but retained on the No. 50 sieve, with a coefficient of uniformity of about 1.5 and a mean particle diameter of about 0.4 mm. The sand grains are predominantly quartz and feldspar with some mica; they have a specific gravity of 2.65 and are rounded to subrounded. The mean length to width ratio of the individual grains as determined by Mahmood (1973) is about 1.4. The maximum and minimum densities, determined in accordance with ASTM D 2049-69 and Kolbuszewski's method (1948), respectively, were 105.7 lbs/ft.^{3} and 89.3 lbs/ft.^{3}. Samples approximately 7.0 inches high by 2.8 inches in diameter were prepared at a relative density of 50 percent by the following 11 different compaction procedures.

a. Pluviation through air (i.e., raining dry sand through a predetermined opening).

b. Pluviation through water (i.e., pouring saturated sand into a water-filled forming mold and vibrating the mold until the desired density was achieved).

c. High frequency (120 Hz) vibrations applied horizontally to dry samples formed in one, seven-inch layer.

d. High frequency (120 Hz) vibrations applied horizontally to dry samples formed in seven, one-inch layers.

e. High frequency (120 Hz) vibrations applied vertically to dry samples formed in seven, one-inch layers.

f. High frequency (120 Hz) vibrations applied horizontally to moist samples (w = 8 percent) formed in seven, one-inch layers.

g. Low frequency (20 Hz) vibrations applied horizontally to dry samples formed in seven, one-inch layers.

h. Low frequency (20 Hz) vibrations applied vertically to dry samples formed in seven, one-inch layers.

i. Tamping moist soil (w = 8 percent) with a 1.4-inch compaction foot to form samples in seven, one-inch layers.

j. Rodding moist soil (w = 8 percent) with a 3/8-inch compaction foot to form samples in seven, one-inch layers.

k. Rodding dry soil with a 3/8-inch compaction foot to form samples in seven, one-inch layers.

Details of the sample preparation procedures have been described by Mulilis, Chan, and Seed (1975). Once a sample was formed by any method of compaction, it was saturated and then consolidated under an effective confining pressure of 8 psi. When the sample was fully consolidated (which required approximately 20 minutes), the drainage valves were closed and the sample was subjected to sinusoidal cyclic deviator stress applications of uniform magnitude. The axial deformations of the sample with increasing numbers of stress cycles were recorded, and the number of cycles required to cause initial liquefaction (pore pressure equal to initial effective confining pressure) and axial strains of ± 2.5, ± 5.0, and ± 10.0 percent were determined.

The results of these tests are presented in Figs. 2-4 through 2-6 which show the relationship between the cyclic stress ratio (cyclic deviator stress divided by twice the initial effective confining pressure) and the number of cycles required to cause liquefaction and ± 2.5 percent axial strain for the various compaction procedures. Figure 2-4 summarizes the results for different vibratory compaction procedures; Fig. 2-5 compares test data for samples prepared by moist tamping, moist rodding, and dry rodding, while the results for most of the different compaction procedures are summarized in Fig. 2-6.

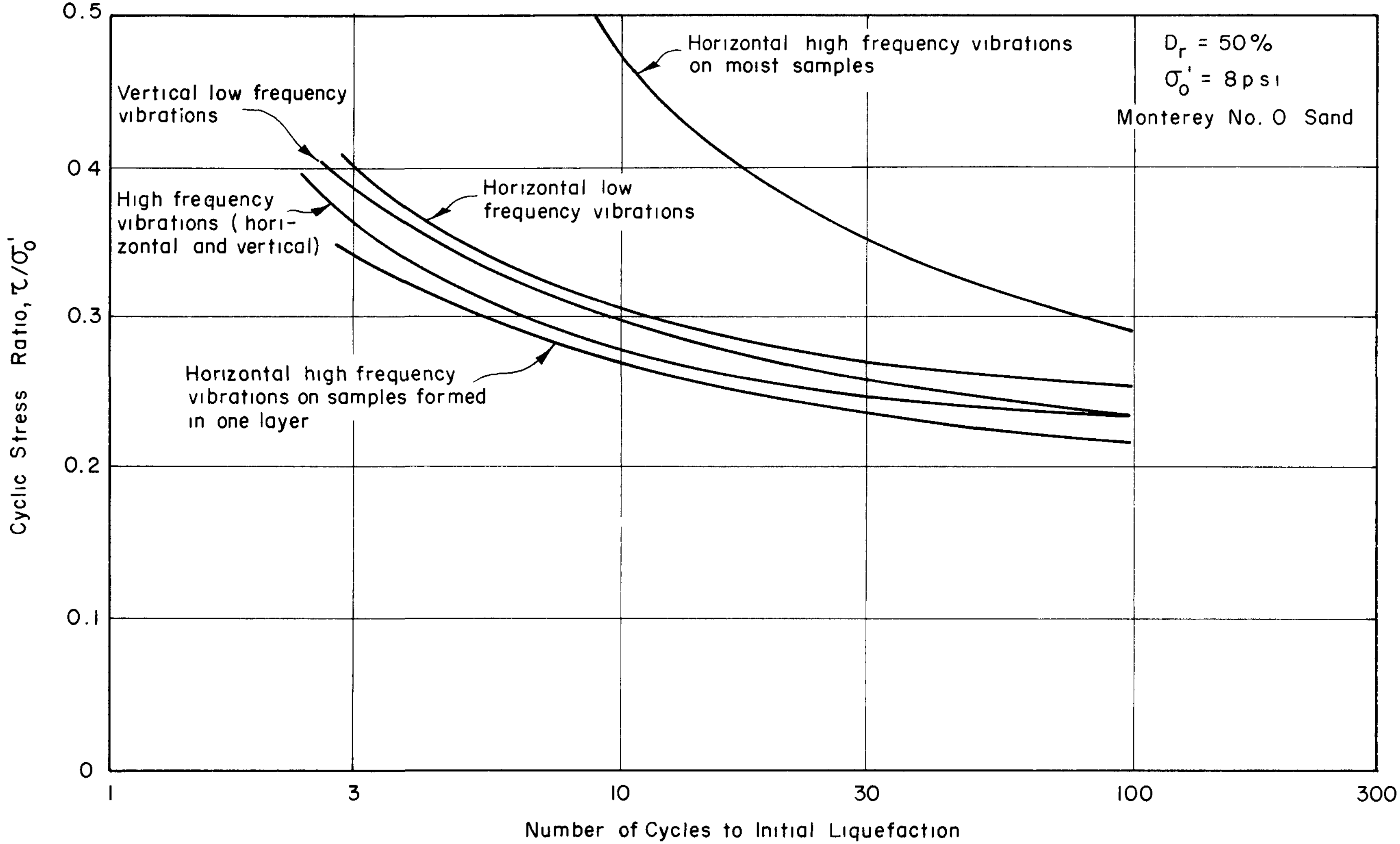


FIG. 2-4 CYCLIC STRESS RATIO VS NO. OF CYCLES FOR DIFFERENT VIBRATORY COMPACTION PROCEDURES

(After Mulilis, Chan and Seed, 1975)

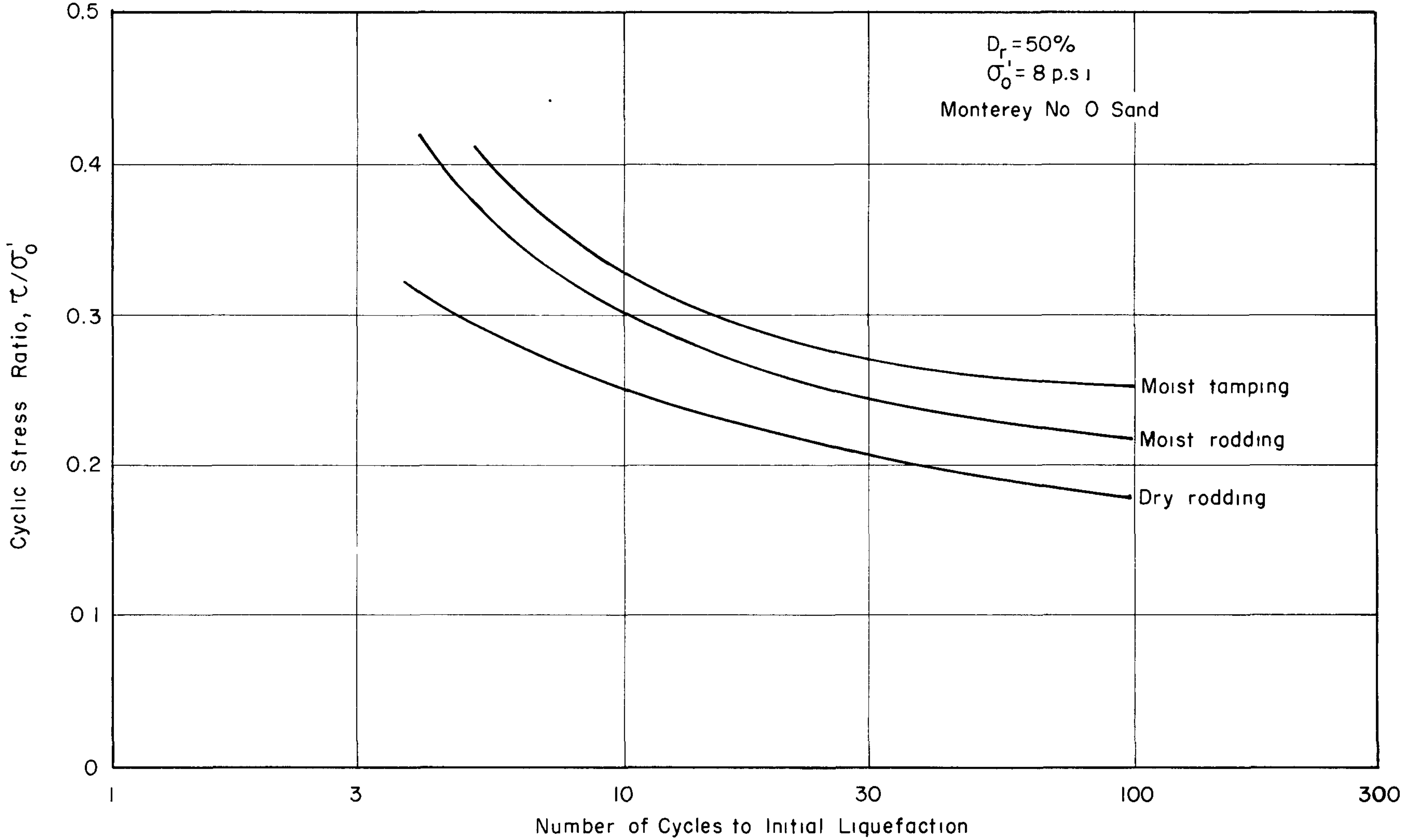


FIG. 2-5 CYCLIC STRESS RATIO VS NO. OF CYCLES FOR DIFFERENT VIBRATORY TAMPING PROCEDURES

(After Mulilis, Chan and Seed, 1975)

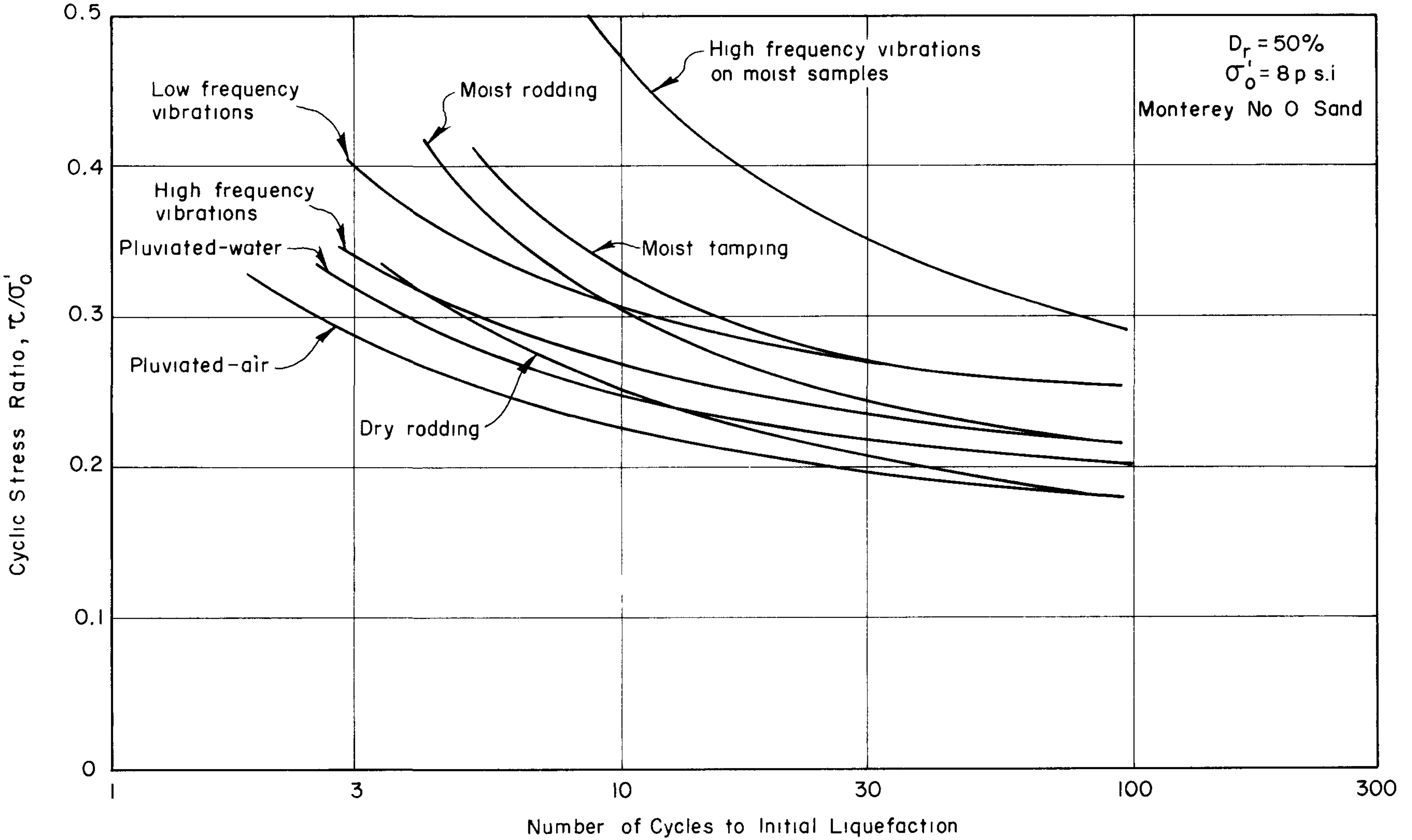


FIG. 2-6 CYCLIC STRESS RATIO VS NO. OF CYCLES FOR DIFFERENT COMPACTION PROCEDURES

(After Mulilis, Chan and Seed, 1975)

It is apparent that the method used to prepare samples of Monterey No. 0 sand can have a significant effect on the resulting cyclic stress characteristics; however, the differences in dynamic strength due to different methods of sample preparation are not the same for all types of soils, as evidenced from the results of tests performed on samples of a sand dredged from the bottom of the Mississippi River. This soil was a uniform, fine silty sand, with a mean particle diameter of about 0.2 mm, approximately one percent passing the No. 200 standard sieve, and containing shells, wood, coal, and various other kinds of debris; the soil was passed through a No. 20 standard sieve to remove the larger pieces of debris.

Stress-controlled cyclic triaxial tests were performed on samples of this soil, prepared to a relative density of 60 percent by moist vibrating and moist tamping at a moisture content of 15 percent, and the results are presented in Fig. 2-7 which shows the relationship between the cyclic stress ratio and the number of cycles required to cause initial liquefaction and ± 2.5 percent axial strain.

As seen in Fig. 2-7, the increase in the cyclic stress ratio causing initial liquefaction for samples prepared by moist vibrating over those prepared by moist tamping was only about 11 percent, as contrasted to a 42 percent increase in the cyclic stress ratio causing liquefaction for samples of Monterey No. 0 sand under similar conditions. The comparisons of strength increase were made at the cyclic stress ratio required to cause initial liquefaction and ± 2.5 percent axial strain in ten cycles. It may be noted that the samples of Montery No. 0 sand were prepared at a lower relative density (50 versus 60 percent) and a lower moisture content (8 versus 15 percent) and tested at a lower initial confining stress (8 versus 38.2 psi), and these factors, in addition to the type of soil, may have had an effect on the differences in the increase in dynamic strength due to sample preparation for the two soils.

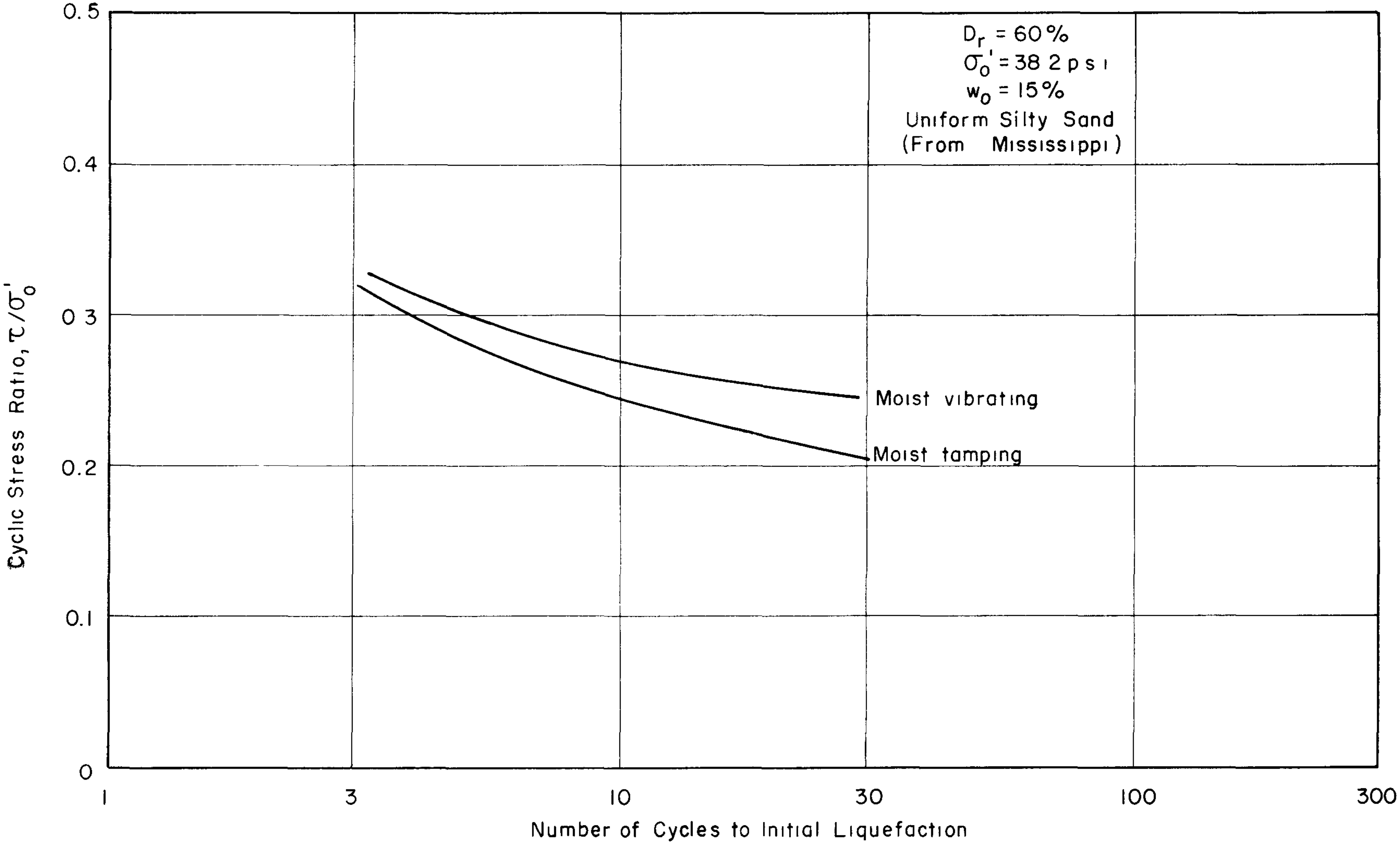


FIG. 2-7 CYCLIC STRESS RATIO VS NO. OF CYCLES FOR 2 DIFFERENT COMPACTION PROCEDURES

(After Mulilis, Chan and Seed, 1975)

To throw some light on this question, a series of tests were performed on samples of Monterey No. 0 sand which were compacted by 1) pluviation through air, 2) high frequency vibrations applied horizontally to moist specimens formed in layers, and 3) tamping moist specimens in layers and tested at different initial effective confining pressures. The samples formed by pluvial compaction were tested at confining pressures of 14.5 and 22 psi, while the samples formed by vibratory and tamping compaction were tested at 38.2 psi.

The results of these tests are summarized in Fig. 2-8 which shows the relationship between the cyclic stress ratio required to cause initial liquefaction and ± 2.5 percent strain in ten cycles and the initial effective confining pressure.

As shown in Fig. 2-8, an increase in the confining pressures at which tests were performed caused a reduction in the cyclic stress ratio required to cause initial liquefaction of the samples irrespective of the method of preparation used to form the samples. It may also be observed that a significant difference in the cyclic stress ratio causing initial liquefaction for samples of Monterey No. 0 sand formed by different compaction procedures is still apparent even at high confining pressures (i.e. about 38 psi); however, the cyclic stress ratio causing initial liquefaction of samples prepared by vibration of moist soil at a high confining pressure is only about 25 percent higher than that of samples prepared by moist tamping compared to a 42 percent difference for tests performed at a confining pressure of 8 psi. In spite of this reduction, it appears that differences in the liquefaction characteristics of samples due to the method of preparation are likely to vary to some extent with the type of soil.

2.5 DENSITY DISTRIBUTION WITHIN SAMPLES OF SAND FORMED BY DIFFERENT METHODS OF PREPARATION

To determine the reasons for the effects of sample prepara-

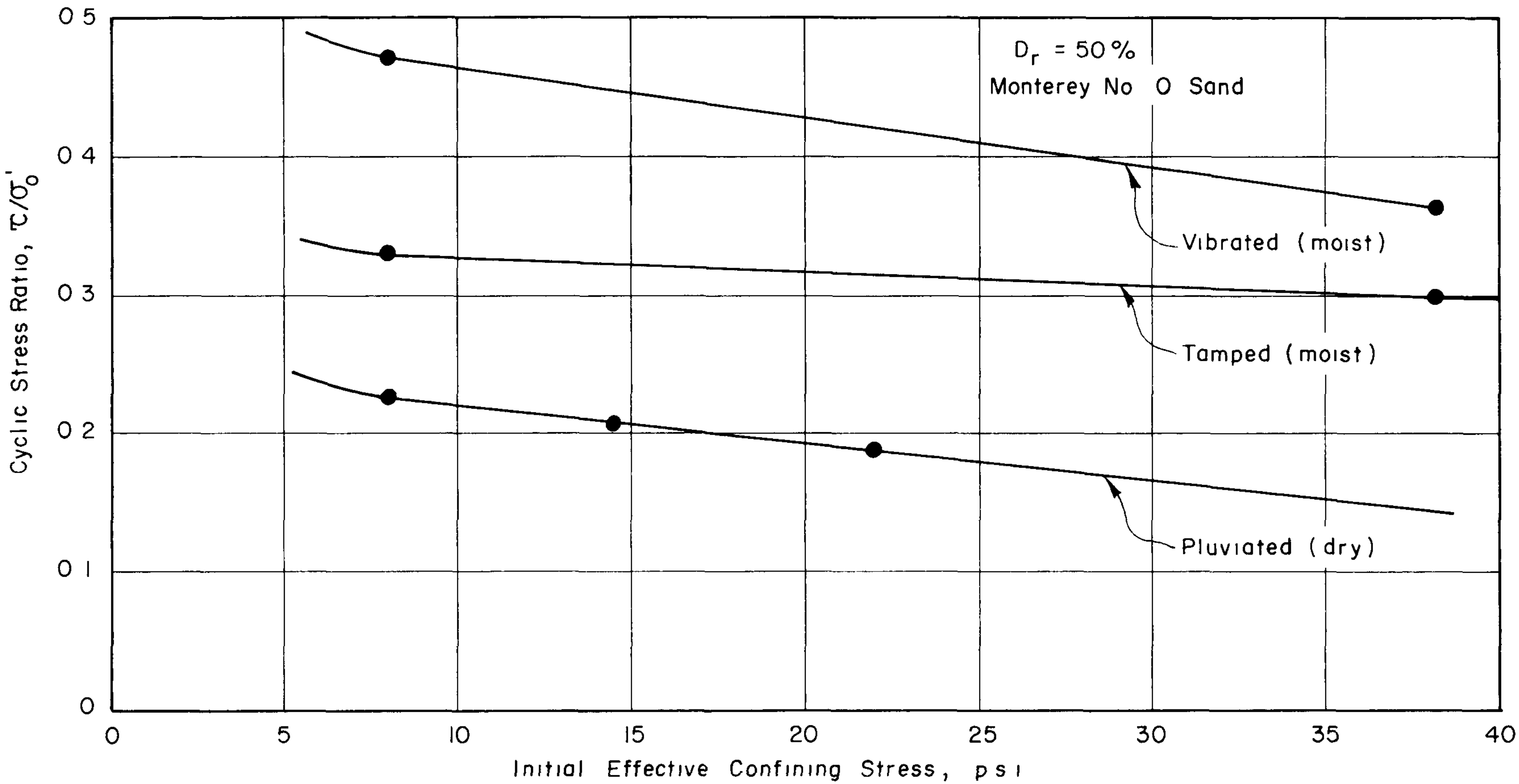


FIG. 2-8 CYCLIC STRESS RATIO AT 10 CYCLES FOR INITIAL LIQUEFACTION VS INITIAL EFFECTIVE CONFINING STRESS

(After Mulilis, Chan and Seed, 1975)

tion method on liquefaction characteristics, an investigation was performed to determine the density distribution as a function of height within samples formed by different methods of preparation. Once a sample was prepared by any given method of compaction, the initial height of the sample was determined by a dial gage. The sample was then trimmed back to a height of approximately five inches by drawing off about two inches of sand through a tube connected to a flask of known weight and to a vacuum source. A final height was determined by a dial gage. The volume of the sand which was excavated was computed, and by comparing the weight of the empty flask with that of the flask and soil, the weight of the excavated soil was computed; thus, the relative density of the top two inches of the sample could be determined. The remainder of the sample was excavated in the same manner in layers of approximately two, two, and one inch, and the relative density of each of the layers was determined.

The density distribution within samples prepared by four different methods of compaction (pluviation of dry sand through) air, low and high frequency vibrations applied horizontally to dry samples formed in seven layers, and high frequency vibrations applied horizontally to dry samples formed in one, seven-inch layer) were determined in the manner described. The values of the relative density for each layer are shown in Table 2-1 together with average values and the maximum variations.

Table 2-1

Relative Density Distribution

Layer	Pluviation (%)	Low Freq. Vibrations (7 Layers) (%)	High Freq. Vibrations (7 Layers) (%)	High Freq. Vibrations (one 7" Layer) (%)
1	55	49	50	64
2	56	51	49	46
3	53	50	46	37
4	55	52	55	48
Average	55	50	49	49
Maximum Variation	3	3	9	27

From the results shown in Table 2-1, it can be observed that compaction by pluviation and low frequency vibrations produced samples which are very uniform, compaction by high frequency vibrations produced samples which were slightly less uniform, and compaction by high frequency vibrations on one, seven-inch layer produced samples which were relatively non-uniform; the latter samples had a denser layer near the top of the sample (i.e., immediately below the surcharge) and a loose layer near the middle of the sample. This loose layer may have accounted for the fact that samples prepared in one, seven-inch layer had the lowest dynamic strength of any of those prepared by the three methods of forming samples in a dry condition by high frequency vibrations.

2.6 INFLUENCE OF STRUCTURE OF SAND ON LIQUEFACTION CHARACTERISTICS

In order to determine whether the observed differences in liquefaction characteristics due to differences in method of sample preparation shown in Figs. 2-4 through 2-6 were due to differences in structure of the samples, detailed studies were conducted to determine the structures of samples prepared by pluviation of dry sand, high frequency vibrations applied horizontally to dry samples formed in layers, and moist tamping in layers (Mulilis, Chan, and Seed, 1975). The liquefaction characteristics of samples prepared by these methods are reproduced in Fig. 2-9.

Measurements of soil structure were made by three methods:

a. By making X-radiographs of thin sections hardened with a polyester resin.

b. By determining the statistical orientations of interparticle contact planes by observing thin sections through a universal stage microscope.

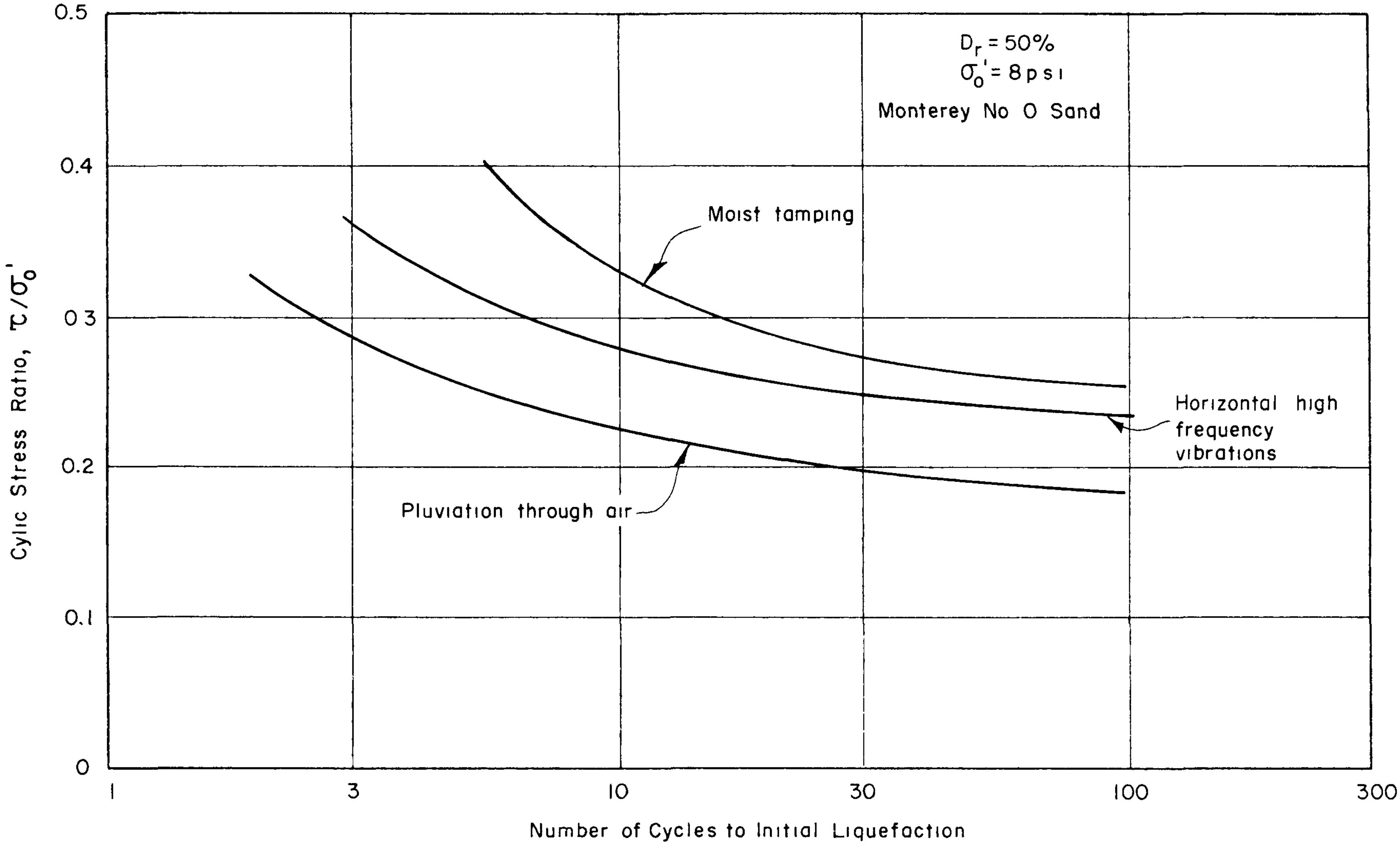


FIG. 2-9 CYCLIC STRESS RATIO VS NO. OF CYCLES FOR SAMPLES PREPARED BY 3 DIFFERENT METHODS

(After Mulilis, Chan and Seed, 1975)

c. By determining the electrical conductivity of the samples which may be used as a measure of the geometric grain arrangement, termed the "formation factor" (Archie, 1942: Arulanandan, 1975). The formation factor is defined as the ratio of the conductivity of the electrolyte to the conductivity of the sand saturated with the electrolyte.

Although the X-radiographs showed distinct differences in density distribution within the samples, there was no correlation between this characteristic and the cyclic stress ratios required to cause initial liquefaction of the different samples. However, as shown in Fig. 2-10 and Table 2-2, good correlations were observed between measurements of the grain structure determined by the orientations of contact planes between grains and formation factors. The sample preparation methods showing the lower angles between the maximum concentration of interparticle contact planes with the vertical axis or the lowest formation factors show the lowest resistance to liquefaction under cyclic loading.

Table 2-2

Statistical Orientation of Contacts Between Grains

Method of Preparation	Angle between Vertical Axis and Maximum Tangent Plane	Angle between Vertical Axis and Normal to Maximum Tangent Plane
Pluviated (dry)	11°	79°
Vibrated (dry)	24°	66°
Tamped (moist)	48°	42°

2.7 COMPARISON OF LIQUEFACTION CHARACTERISTICS OF NATURAL SAND DEPOSITS AND FRESHLY DEPOSITED LABORATORY SAMPLES

The studies described above would seem to leave little doubt concerning the facts that:

a. The behavior of a saturated sand under cyclic loading conditions is a function of its seismic history and grain structure as well as its placement density.

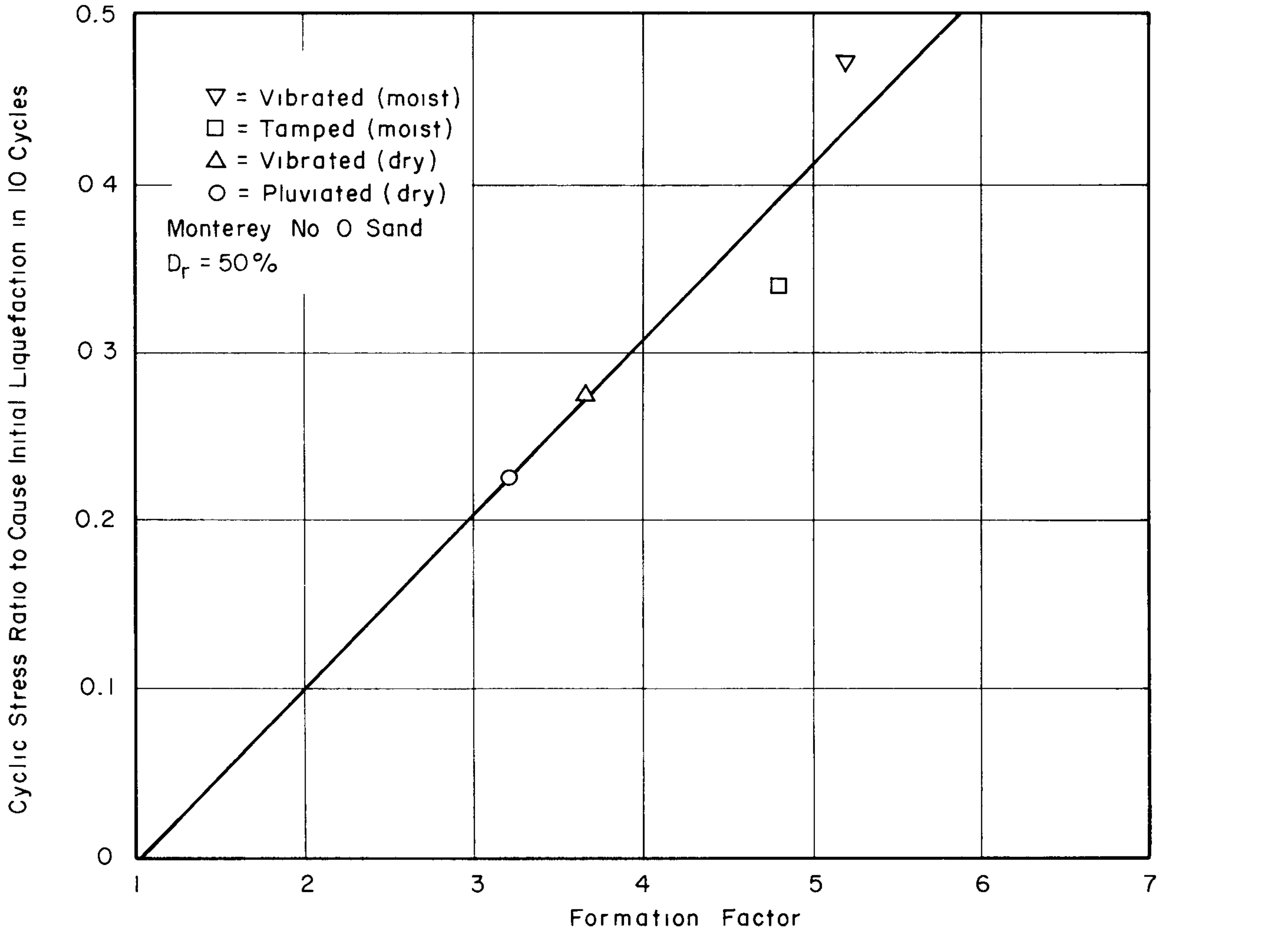


FIG. 2-10 RELATIONSHIP BETWEEN CYCLIC STRESS RATIO AND FORMATION FACTOR

(After Mulilis, Chan and Seed, 1975)

b. If samples prepared in the laboratory are to have the same characteristics as a soil deposit in the field, they must be prepared in a manner producing the same density and grain structure and tested in such a way that the in-situ value of K_o can be taken into account in assessing their field performance.

Clearly, the creation of a structure similar to that of the field deposit is only possible if the structure of the field deposit can be determined, and measurements of grain structure are by no means a standard procedure in soil mechanics laboratories. Since the measurement of the formation factor for a sand is a relatively rapid and inexpensive procedure, this method of measuring particle arrangement or fabric may provide a practical means for determining the structure of sands for a variety of purposes. However, further studies of the potential usefulness of this index of structure are required before it could be recommended for adoption as a practical tool for design studies.

Under these conditions, it would appear to be of interest to determine directly the relationship between the liquefaction characteristics of undisturbed samples of natural sand deposits and those of samples prepared by sedimentation procedures in the laboratory, which presumably reproduce structures similar to those of freshly deposited sands. While only a limited number of studies have been made to compare the liquefaction characteristics of undisturbed samples with those of laboratory prepared samples of the same density, it is significant to note that measured values of the resistance to liquefaction of natural sand deposits, obtained by tests on undisturbed samples, are invariably found to be higher than those of samples prepared to the same density by sedimentation processes in the laboratory. A comparison of such test data compiled by Mulilis, Chan, and Seed (1975) is shown in Table 2-3. It may be noted that the stress ratio causing liquefaction of the undisturbed samples was charac-

TABLE 2-3

COMPARISION OF LIQUEFACTION RESISTANCE CHARACTERISTICS OF UNDISTURBED AND RECONSTITUTED SAMPLES

Firm	Project	Ratio of Undisturbed to Remolded Strength[1]	Soil Type	Method of Remolding
Woodward-Clyde (Oakland, Ca.)	South Texas	1.00	silty fine sand, D_{50} = 0.07 to 0.27 mm	moist tamping, 3/4" dia. tamping foot
Woodward-Clyde (Orange, Ca.)	San Onofre	1.15	well-graded coarse to fine sand, 15% - #200 sieve	moist tamping, 3/4" dia. tamping foot
U. C. Berkeley	Blue Hills Texas	1.15	uniform fine silty sand, D_{50} = 0.4 mm, 8% to 15% - #200 sieve	moist tamping, 1.4" dia. tamping foot
Dames & Moore (San Fran., Ca.)	Allens Creek (heat sink area)	1.20	fine silty, clayey sand, D_{50} = 0.03 to 1.6 mm, 0% to 40% - #200 sieve	moist tamping, 1" dia. tamping foot
Dames & Moore (San Fran., Ca.)	Allens Creek (plant area)	1.27	fine silty, clayey sand, D_{50} = 0.03 to 1.6 mm, 0% to 40% - #200 sieve	moist tamping, 1" dia. tamping foot
Converse-Davis	Perris Dam	1.45	clayey sand, LL = 26, PI = 11, 44% - #200 sieve	moist tamping, 1/2" dia. tamping foot
Law Engineering and Testing	Florida sand	1.30	silty sand with shells	dry vertical vibrations, frequency = 120 c.p.s.
W. E. S.	Ft. Peck Dam (foundation)	1.65 to 1.80	uniform fine silty sand	dry rodding (3/8" dia. foot), followed by static compaction
W. E. S.	Ft. Peck Dam (shell)	1.70 to 2.00	uniform fine to medium sand	dry rodding (3/8" dia. foot), followed by static compaction

[1]Ratio of cyclic stress ratios required to cause liquefaction in ten cycles for undisturbed and remolded samples.

teristically between 0 and 45 percent higher than those of samples prepared by tamping moist samples in laboratory compaction tests, and these, in turn, have been found to be stronger than samples prepared by sedimentation through water or pluviation through air (see Fig. 2-6).

There is thus strong evidence that the liquefaction resistance of undisturbed samples of a number of natural deposits is substantially higher than that of freshly deposited laboratory samples at the same density. Possible explanations for this include the following:

a. Natural deposits have a somewhat more stable structure, perhaps due to the greater lateral movements associated with the deposition process, than those of the same sand deposited in the laboratory.

b. Natural deposits invariably acquire some increase in stability due to small local seismic events which occur in most environments, thereby producing a more resistant soil structure and an increase in K_o.

c. Natural deposits acquire some increase in stability as a result of the long periods of sustained stress to which they are subjected, thereby producing some type of "cementation" at particle contacts, in comparison with short-term tests on laboratory samples at the same density.

d. The vibrations inevitably associated with the extraction of samples from the ground are simply another form of seismic history which sometimes tend to make "undisturbed" samples have a higher resistance to liquefaction than they would have in-situ.

In assessing the merits of these possible effects, it should be considered that there is a reasonable expectancy of improvements in soil characteristics due to items (a) and (b) above; there is also a reasonable expectancy that resistance to liquefaction would increase as a result of sustained confining pressures, although there is currently no direct evidence from laboratory studies on sands to support this idea, and any effects of sampling vibrations on the resistance to liquefaction may well be off-set by the effects of sampling disturbance on the density of the samples obtained. In fact, studies conducted by the U.S. Army Corps of Engineers (1952) have shown that the density of medium dense to dense sands is often reduced by sampling operations. There are good reasons to expect this to be so, and Castro (1975) has recently presented convincing evidence to show that the liquefaction resistance of laboratory samples extracted from zones of sand having a high penetration resistance is little better than that of samples extracted from zones of low penetration resistance, Fig. 2-11. This is not in accord with the behavior of laboratory samples having different densities corresponding to the higher and lower penetration resistances, and strongly suggests a loosening of the dense sand during the sampling process. Thus undisturbed samples of medium to dense sands may well be weakened by loosening during sampling more than they are strengthened by the effects of sampling vibrations.

All of these factors must be weighed together in assessing the significance of the comparative strengths of laboratory-prepared samples and undisturbed samples extracted with sufficient care that the structure of the natural deposit remains intact and is not modified by the sampling procedure. Some judgment will inevitably be necessary in assessing the net effect of the various factors involved, since for loose sands, some slight densification and structure change may occur during sampling, while for dense sands, some loosening but negligible structure change is likely to develop during sampling. For these reasons, it would seem desirable to supplement laboratory studies

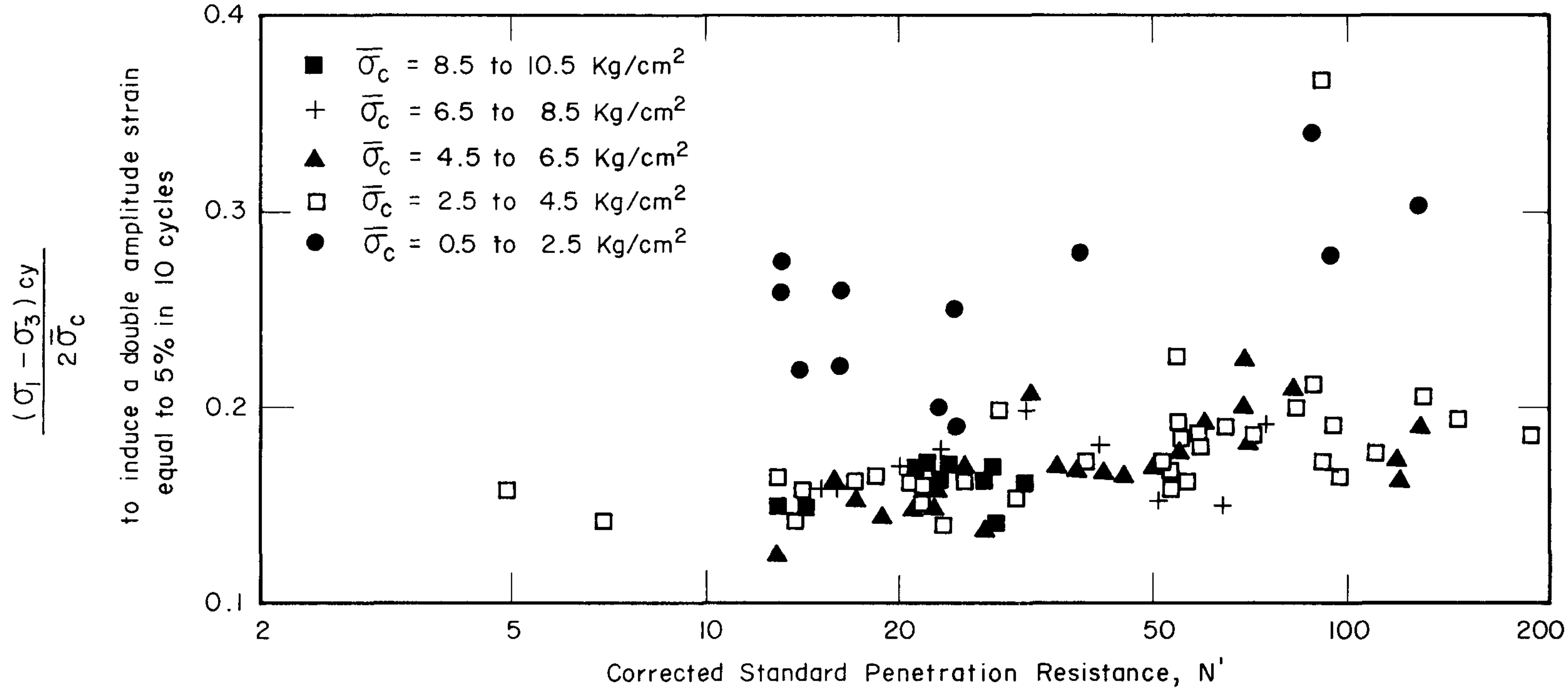


FIG. 2-11 RELATIONSHIP BETWEEN CYCLIC STRESS RATIO CAUSING 5% STRAIN AND STANDARD PENETRATION RESISTANCE (after Castro, 1975)

of the properties of undisturbed samples with correlations of in-situ properties and observed field performance in arriving at a final decision on the liquefaction characteristics of a particular deposit.

CHAPTER 3

EVALUATION OF LIQUEFACTION TEST PROCEDURES

As noted previously, if cyclic loading tests are to provide a reliable index of the stress conditions causing liquefaction in the field, it is necessary that they reproduce field conditions in all respects with a satisfactory degree of accuracy and that their capability to do this be checked against field behavior of soil deposits. While some checks of this type have been made, the number of cases of known field performance and the range of conditions they represent is quite small, and it is highly desirable that they be supplemented in some manner. Since it is impractical to wait for future earthquakes to provide the required data, it seems desirable to generate the required information by test programs designed to represent field conditions as closely as possible.

To this end, some type of simple shear test seems to provide the closest representation of field conditions. However, it is desirable to avoid the stress concentrations believed to develop in small-scale samples and to conduct tests representative of the multi-directional shaking which occurs during an earthquake. Accordingly, a series of tests were conducted by De Alba, Chan, and Seed (1975) using large-scale samples (90 inches long by 42 inches wide by four inches deep) to determine accurately the stress ratios causing initial liquefaction and different levels of shear strain under one-dimensional simple shear conditions. A second series of tests on large samples (42-inch diameter by three inches deep) were conducted by Pyke, Chan, and Seed (1974) to determine the effects of multi-directional as compared with uni-directional shaking. Since these tests should provide accurate data on the behavior of soils under simple shear conditions, they can serve as a basis for evaluating the accuracy of data obtained with other types of apparatus and possibly also supplement field data by extending the range of conditions known to have caused liquefaction problems in the field. The results of these large-scale test programs are, therefore, summarized below.

3.1 LARGE-SCALE SIMPLE SHEAR TESTS USING ONE-DIRECTIONAL SHAKING

The equipment used for the large-scale simple shear tests conducted by De Alba, Chan, and Seed (1975) is shown schematically in Fig. 2-1. Basically, a bed of sand, 90 by 42 by 4 inches deep was constructed on a shaking table; a rubber membrane was placed over the sand to prevent drainage, and a reaction mass, with a flexible base to provide uniform seating on the sand, but a rigid lateral resistance was placed on top of the sand to serve as a reaction mass. Horizontal movements of the base thus produced cyclic stress conditions in the sand, and the dimensions were selected to provide a free-field condition in a substantial part of the central section of the sample. The ends of the sample were tapered as shown in Fig. 2-1 so that it was not in contact with the walls of the box and was free to undergo cyclic strains in response to the applied stresses. Ample instrumentation was provided to measure the build-up of pore pressures at different points in the sample and the deformations which developed with increasing numbers of cyclic stress applications. By capping the sample container with a rigid box, air pressures could be applied to the sand to produce confinement representative of different depths in the ground.

Although the samples were large enough to be essentially free of stress concentrations due to boundary effects, it was found that covering the large surface of the sand by a rubber membrane introduced a compliance in the sand-pore water system due to membrane penetration between grains which permitted a small but significant increase in volume of the system to develop as the pore pressures built up and pushed out the membrane from its original position. Correction factors for this effect were typically about 25 percent and would be of comparable magnitude in any similar tests where membranes are placed over large sample areas to prevent drainage and apply external confining pressures. However, the need for application of such a correction has not generally been recognized.

All of the tests in this program were performed in samples of Monterey No. 0 sand. All of the test samples were prepared by pluvial compaction, and using this method of sample preparation, samples were prepared and tested at relative densities of 54, 68, 82, and 90 percent. In all cases, it was found that pore pressures built up with increasing numbers of cycles until a condition of initial liquefaction developed, and this was accompanied by the development of cyclic strains in the test samples, large strains occurring in the looser samples but much smaller strains in the denser samples.

The results of the test program are summarized in Figs. 3-1 and 3-2. Figure 3-1 shows the corrected stress ratios required to produce a condition of initial liquefaction in different numbers of stress cycles for samples at different relative densities. The average rates of development of pore pressures in the samples at different relative densities are shown in normalized form in Fig. 3-2. These latter results provide a useful basis for determining the rate of increase in pore water pressures under undrained conditions, but they also provide a means for predicting the rate of build-up of pore water pressures in sand deposits where some dissipation of pressure may also occur during the period of cyclic loading (Seed, Martin, and Lysmer, 1975).

The results of the tests shown in Fig. 3-1 are compared with those obtained in other shaking table studies in Fig. 3-3. Since the results of previous studies were not corrected for the effects of membrane penetration and compliance effects, both uncorrected and corrected test data for the study by De Alba, Chan, and Seed (1975) are presented. To provide a common basis for

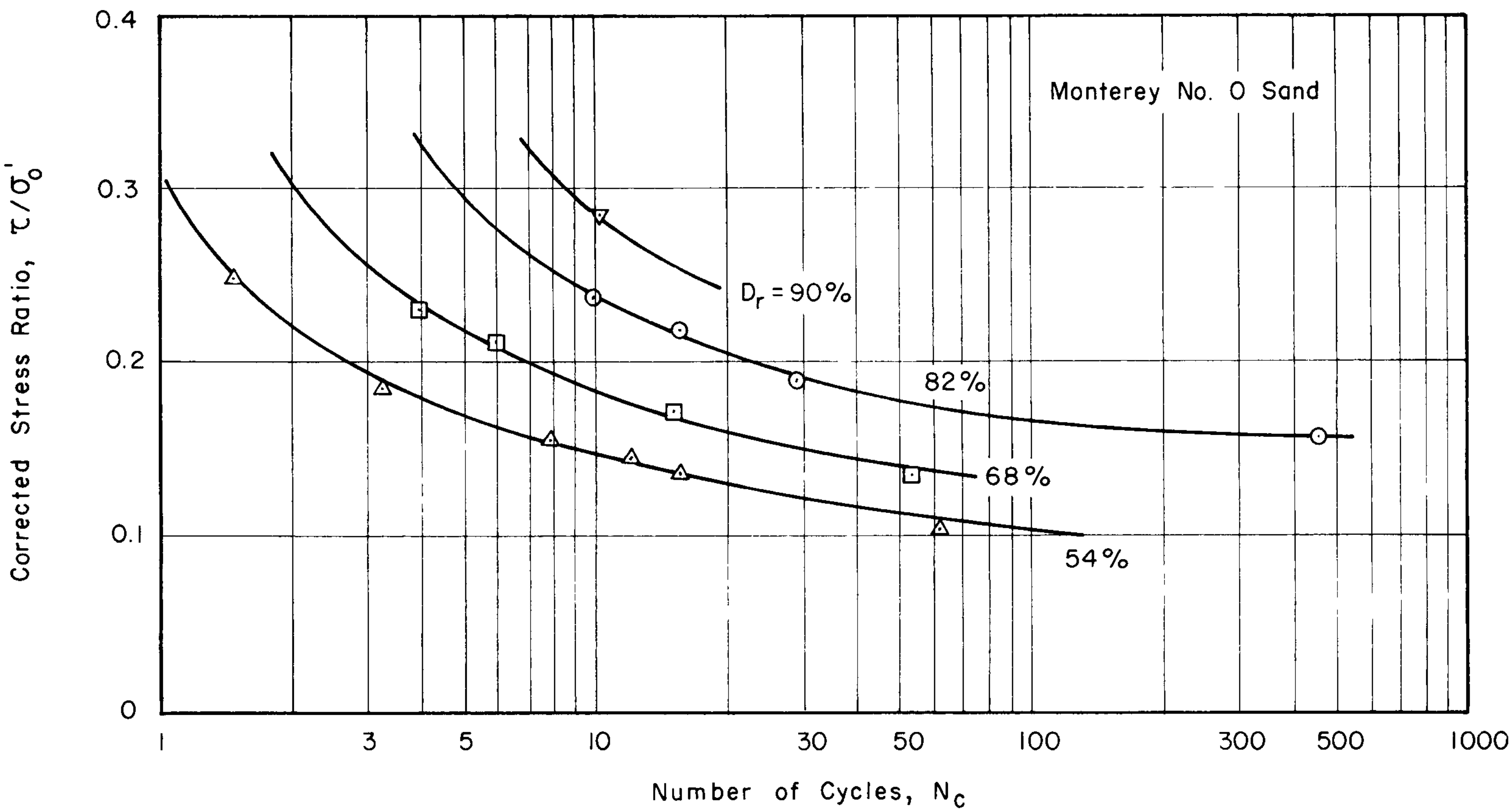


FIG. 3-1 CORRECTED τ/σ_o' VS N_c FOR INITIAL LIQUEFACTION

(After De Alba, Chan and Seed, 1975)

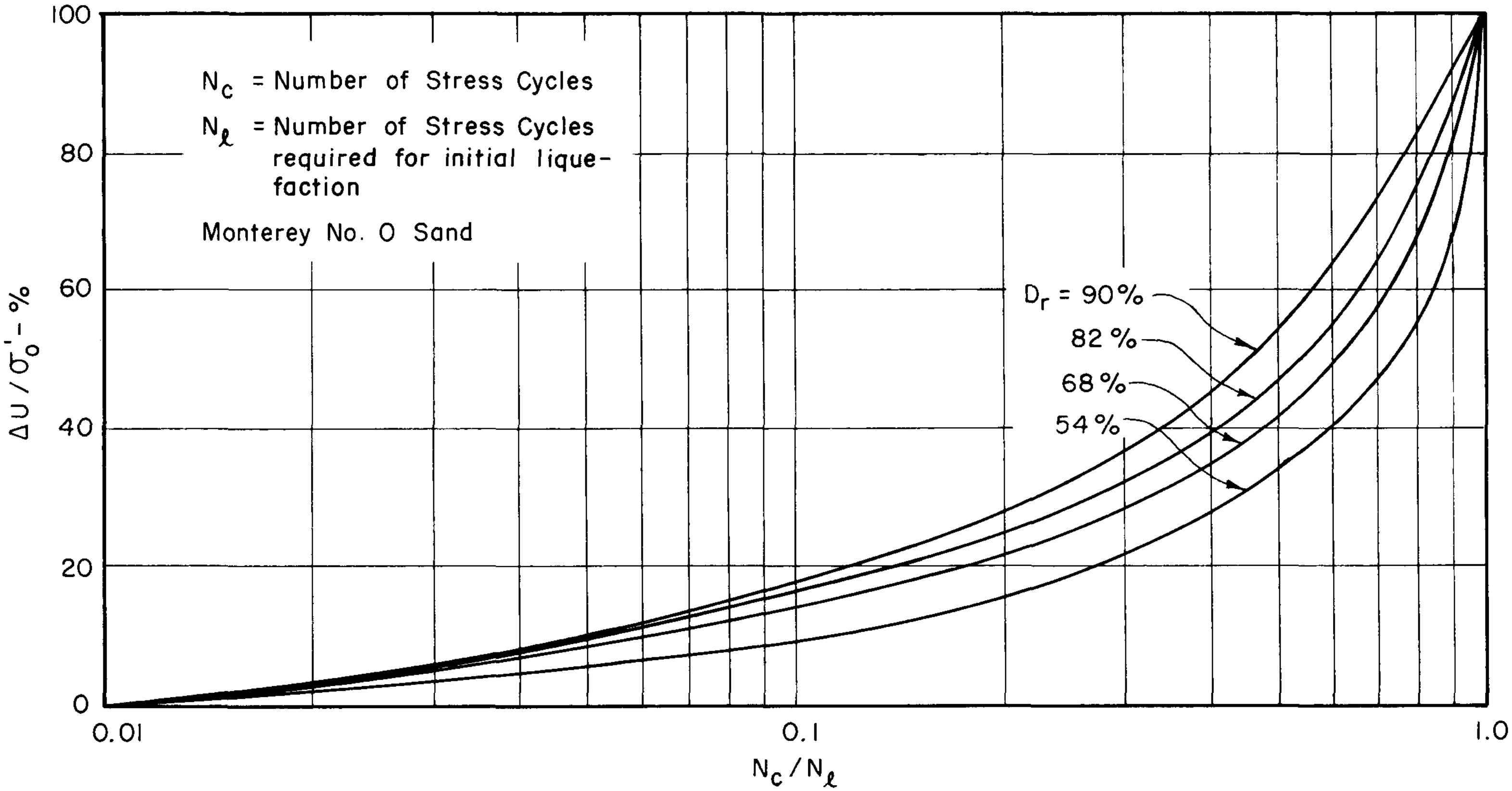


FIG. 3-2 NORMALIZED DYNAMIC PORE WATER PRESSURE CURVES

(After De Alba, Chan and Seed, 1975)

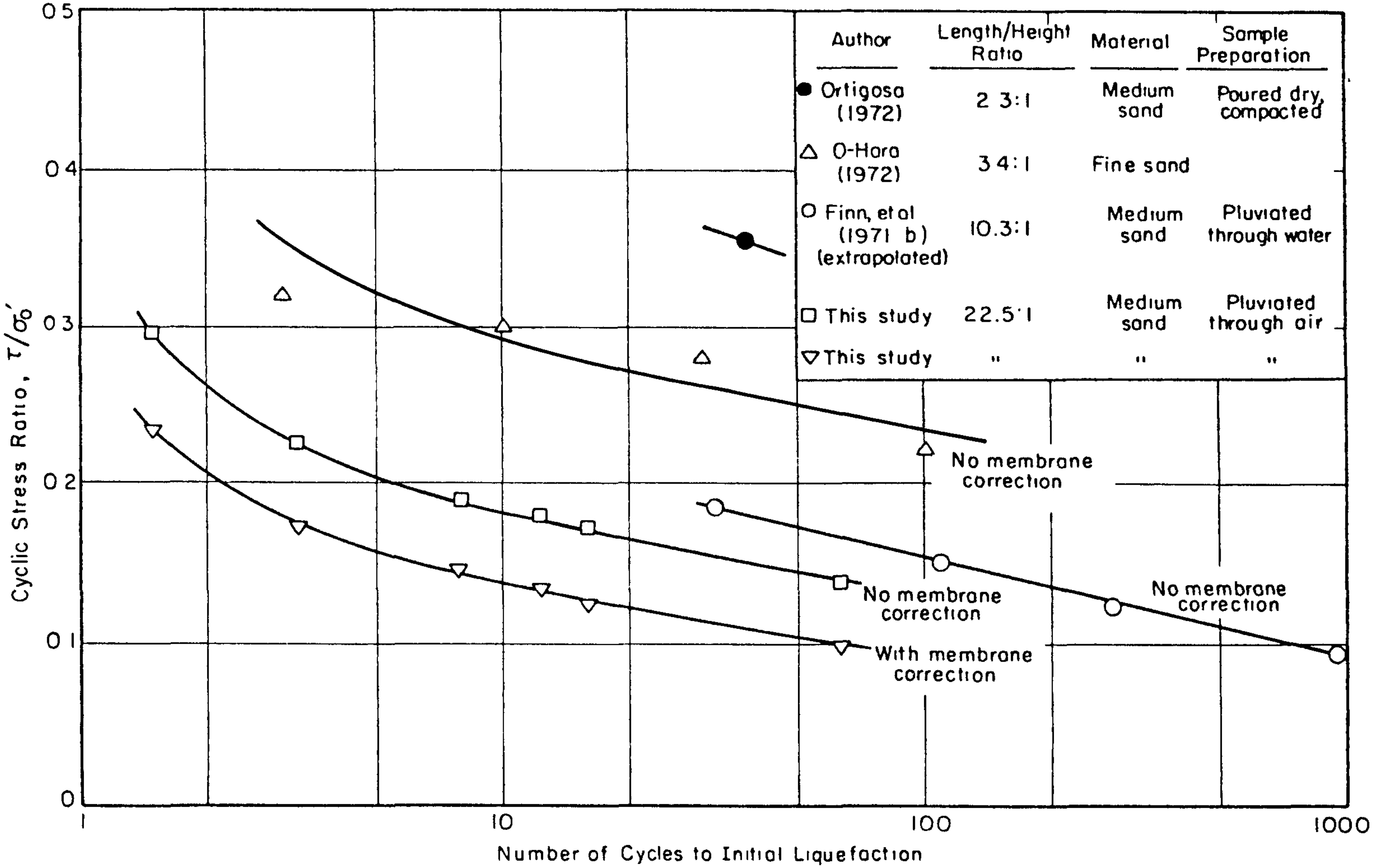


FIG. 3-3 COMPARISON OF SHAKING TABLE TEST RESULTS FOR D_r =50%

(After De Alba, Chan and Seed, 1975)

comparison, data are shown in each case for tests at a relative density of 50 percent; where test data were not determined at this relative density, they were corrected to this condition by using the observation that stress ratios required to cause liquefaction are for practical purposes directly proportional to relative density up to relative densities of about 75 percent.

It may be seen that there is clear evidence that the test results are significantly influenced by the length/height ratio of the test samples and thereby, in most previous investigations where samples have been in contact with the walls of the container, by the stiffness of the walls involved. It appears that the tests by Finn, Emery, and Gupta (1971) with a length to height ratio of about 10.3 may have been sufficiently free from these effects to provide reasonable test data, provided a correction for membrane compliance is also applied. It should be noted that part of the difference in test results in the various investigations is probably due to different methods of sample preparation, but this effect alone could not explain the large differences in reported data. In any case, the different results clearly indicate the care required to provide correct boundary conditions if meaningful data are to be obtained by means of shaking table studies, and the need for careful evaluation of the data in tests conducted in this manner.

The test data obtained by De Alba, Chan, and Seed (1975) are compared with test data from small-scale simple shear devices of different types in Fig. 3-4. Again, all test data have been corrected to a relative density of 50 percent. In this case, samples in all of the studies shown were prepared either by sedimentation through water or by pluvial compaction, so different methods of preparation should not significantly affect the results.

It may be seen that there is very good agreement between the results reported by Seed and Peacock (1971), Finn, Pickering,

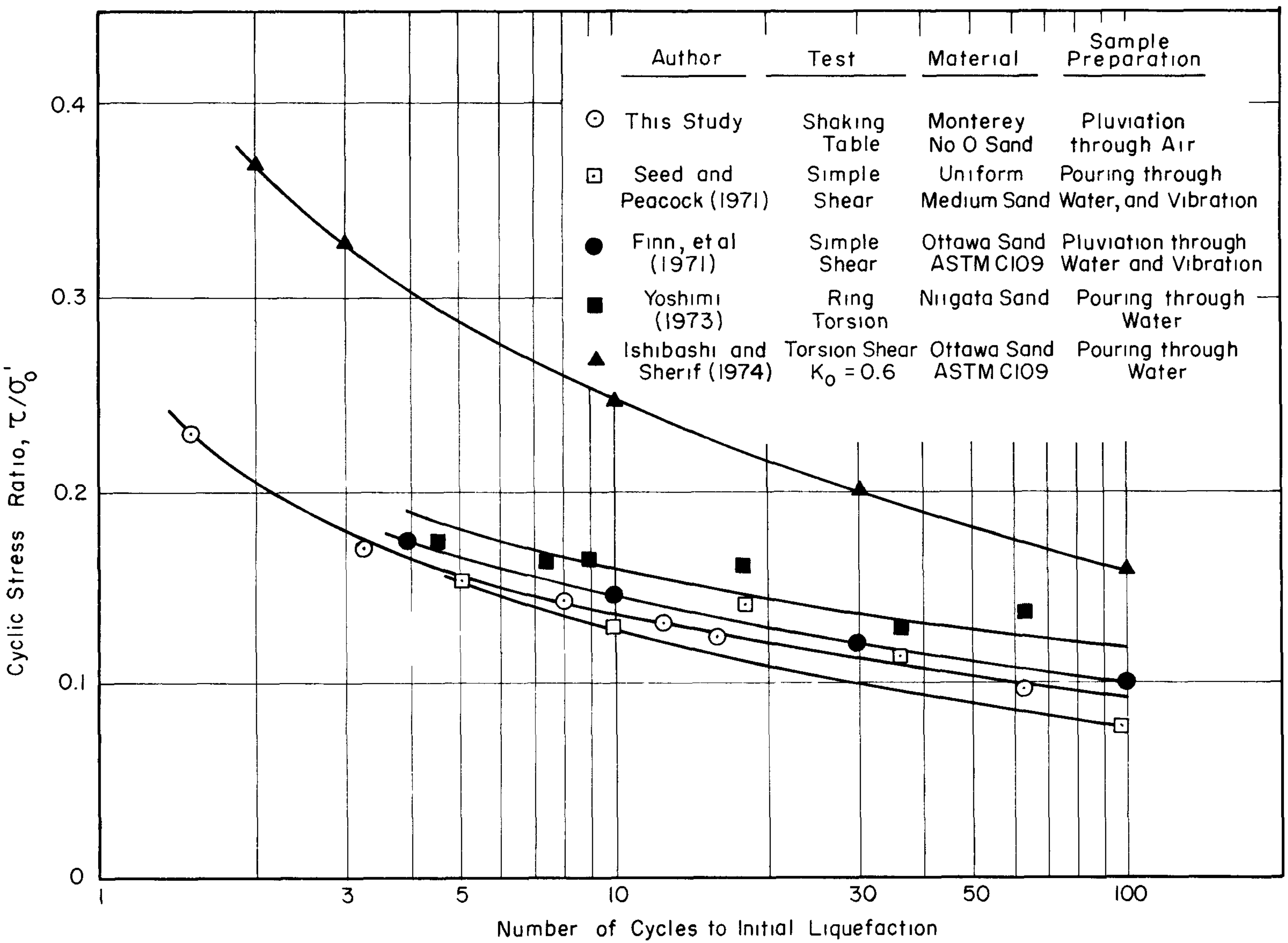


FIG. 3-4 COMPARISON OF SHAKING TABLE AND SIMPLE SHEAR LIQUEFACTION TEST RESULTS FOR $D_r = 50\%$

(After De Alba, Chan and Seed, 1975)

and Bransby (1971), Yoshimi (1973), and those obtained in the large-scale tests, indicating either that the errors due to stress concentrations in small-scale tests may not be so large as has often been claimed (e.g., Castro, 1975), or they are counter-balanced by some other feature of the test.

The test data reported by Ishibashi and Sherif (1974) were obtained using a higher value of K_o than those used in the other investigations and may well be higher than the other data for this reason. In fact, previous studies by Seed and Peacock (1971) have shown that increasing the value of K_o from 0.4 to 0.6 will increase the stress ratio required to cause initial liquefaction by about 50 percent, and this factor alone would almost account for the higher values indicated by the Ishibashi and Sherif data in Fig. 3-4.

It would appear, however, that carefully conducted small-scale simple shear or torsional shear tests using good quality equipment can provide data comparable to that obtained with large-scale test samples and presumably representative of simple shear field conditions if these could develop uni-directionally.

3.2 COMPARISON OF LARGE-SCALE SIMPLE SHEAR TEST DATA WITH CYCLIC TRIAXIAL TEST DATA

It has already been noted that cyclic loading triaxial tests lead to different stress ratios causing liquefaction that cyclic simple shear tests for a variety of reasons, some of them associated with the stress conditions under which the tests are conducted, some associated with the methods of data interpretation used and some associated with limitations of the cyclic triaxial test procedure itself.

The development of high quality data from the large-scale shaking table studies described previously provided an excellent opportunity to compare the results obtained with those determined by cyclic triaxial compression tests on the same sand, prepared in the same manner and tested at the same confining pressure. The results of such a comparison are shown in Fig. 3-5, which compares the relationship between the cyclic stress ratio τ/σ'_o and number of stress cycles required to cause initial liquefaction in simple shear tests with the relationship between the cyclic stress ratio τ_m/σ'_{3c} or $\sigma_{dc}/2\sigma'_{3c}$ and the number of stress cycles required to cause initial liquefaction in cyclic triaxial tests. As before, the comparison is shown for samples having a relative density of 50 percent.

It is readily apparent that for any given number of cycles to initial liquefaction, the cyclic triaxial stress ratio is higher than that for simple shear conditions. Comparison of the ordinates of the two curves shown in Fig. 3-5, therefore, provides values of the stress correction factor

$$c_r = \frac{\tau}{\sigma'_o} / \frac{\tau_{max}}{\sigma'_{3c}}$$

For the data shown, values of c_r vary slightly with the number of cycles from about 0.65 at four cycles to about 0.60 at 50 cycles.

Similar comparisons have been made for samples tested at relative densities ranging from 50 to 90 percent, and the results are summarized in Figs. 3-6 and 3-7. Values of c_r for samples reaching a condition of initial liquefaction in ten cycles are shown in Fig. 3-6. Regardless of the relative density of the samples involved, the value of c_r was the same and equal to 0.63. The same independence of relative density was found for other numbers of cycles. Values of the correction factors c_r found to be applicable for different numbers of cycles at all relative densities are, therefore, summarized in Fig. 3-7.

These values range from about 0.6 to 0.65 and are reasonably close to those indicated by previous experimental studies and

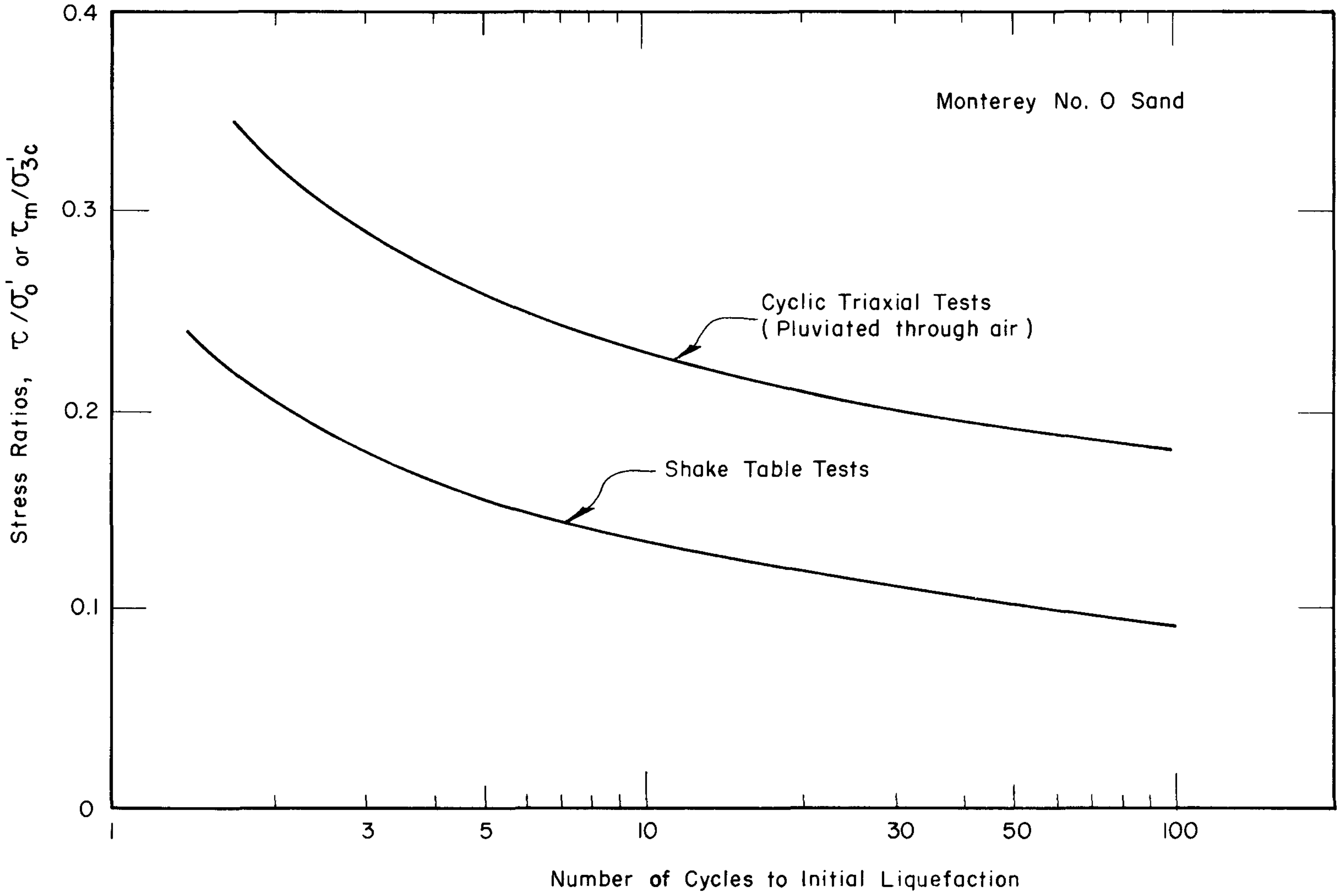


FIG. 3-5 COMPARISON OF SHAKING TABLE AND CYCLIC TRIAXIAL TEST RESULTS FOR $D_r = 50\%$

(After De Alba, Chan and Seed, 1975)

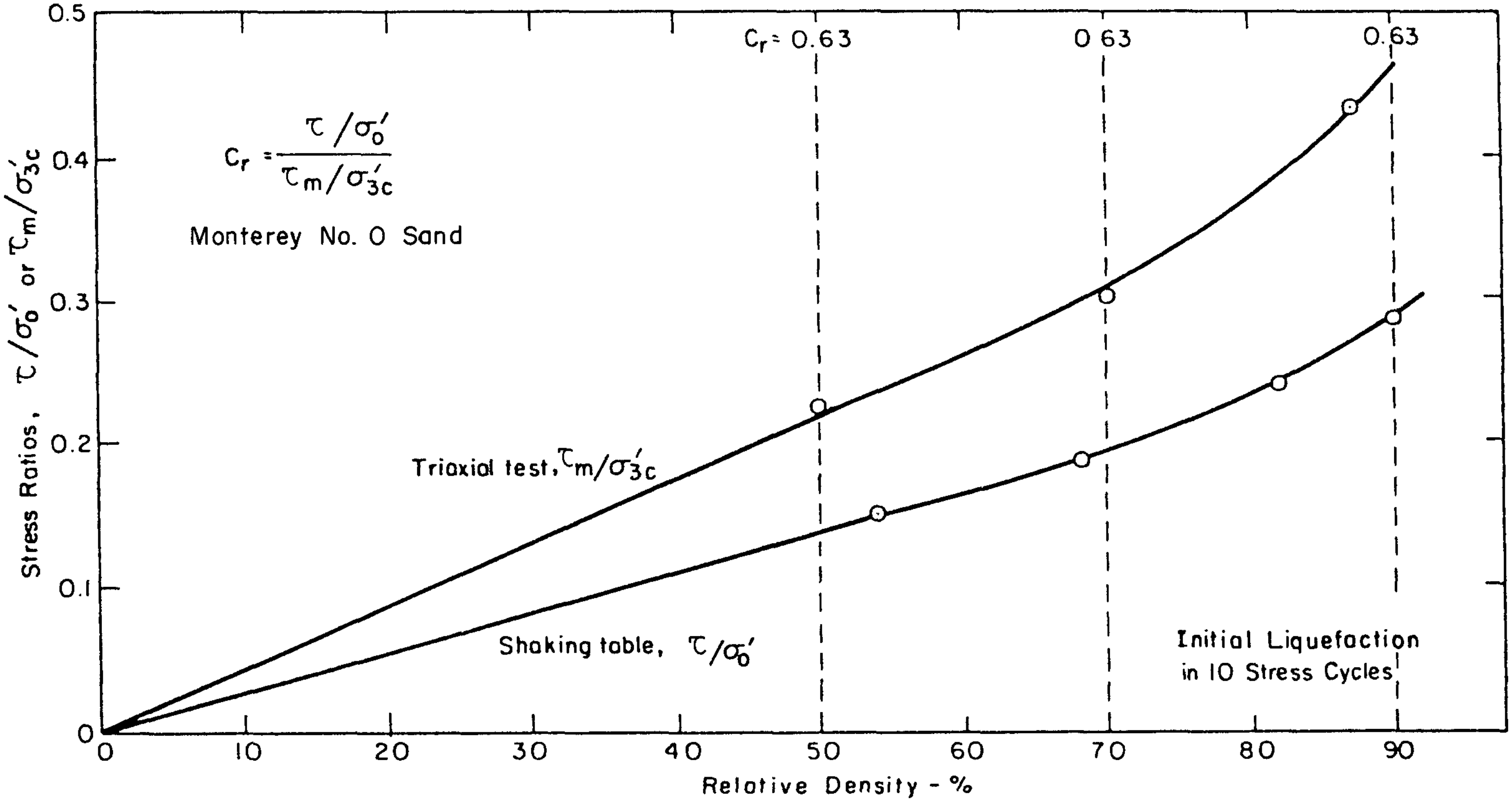


FIG. 3-6 COMPARISON OF SHAKING TABLE AND TRIAXIAL TEST RESULTS

(After De Alba, Chan and Seed, 1975)

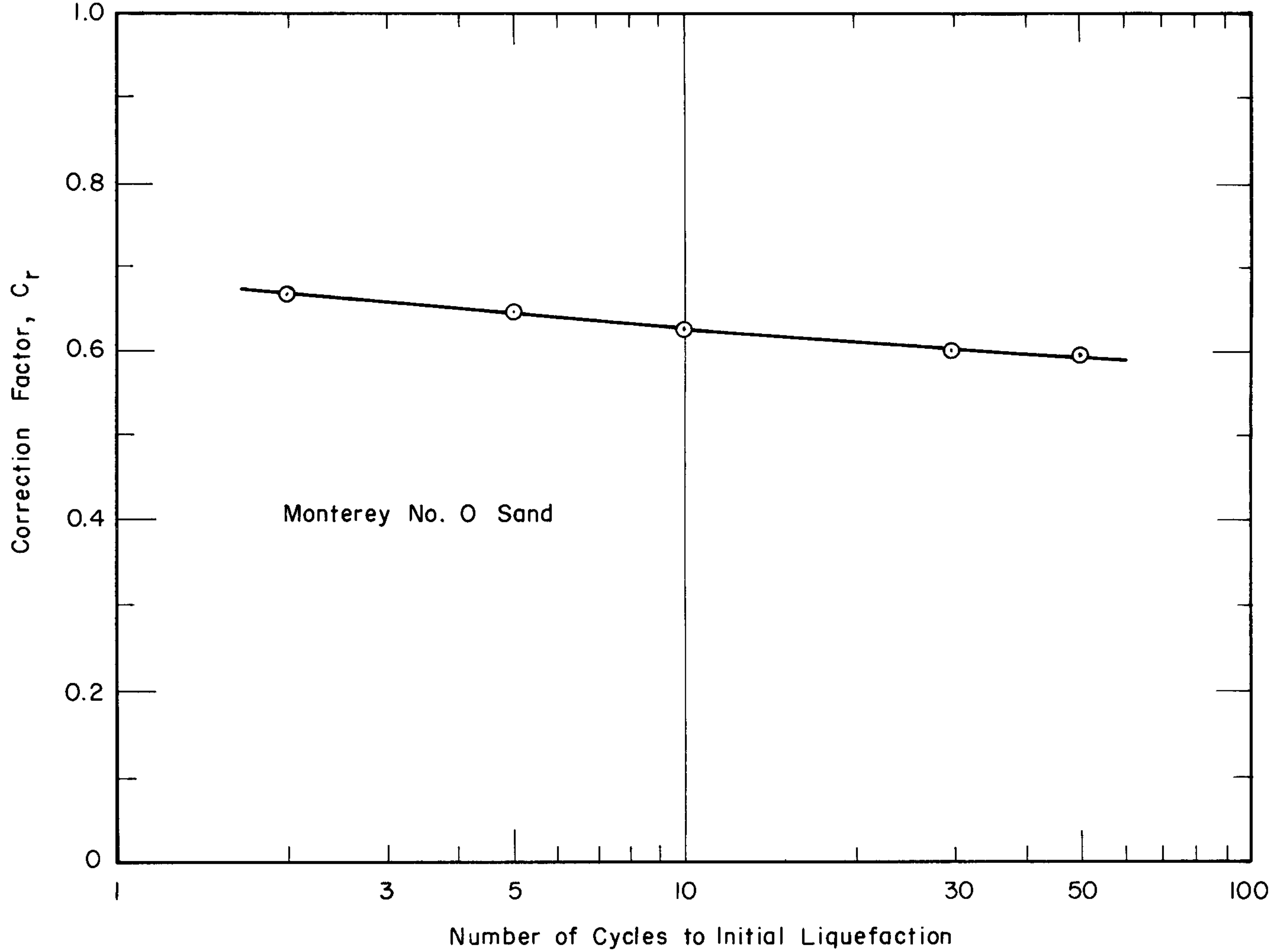


FIG. 3-7 CORRECTION FACTORS FOR TRIAXIAL TEST RESULTS

(After De Alba, Chan and Seed, 1975)

those developed on an analytical basis by Seed and Peacock (1971), Finn, Pickering, and Bransby (1971), and Castro (1975). Accordingly, they would seem to provide a sufficiently reliable basis for correcting the results of cyclic triaxial compression tests to obtain stress ratios corresponding to one-directional simple shear conditions on the same material and at the same maximum confining pressure.

It is interesting to note that in the cyclic triaxial test program conducted by Mulilis to determine the cyclic stress ratio causing liquefaction and different strain amplitudes for samples with relative densities ranging from 50 to 90 percent, involving cyclic stress ratios ranging from 0.2 to 0.5, there was no apparent effect of non-uniform strains or water content redistribution in the samples prior to initial liquefaction on the development of certain limiting strains. However, it was apparent that non-uniform conditions developed once necking occurred in the test specimens. Similar results have been observed in other studies. Thus, it appears that carefully conducted cyclic triaxial tests can provide valid data on cyclic loading characteristics up to initial liquefaction and strains of the order of about five percent for dense samples or 20 percent for loose samples. Reliable data cannot be obtained, however, once necking occurs in any test specimen or if non-uniform conditions exist in the initial sample placement in the triaxial cell.

3.3 DETERMINATION OF THE EFFECTS OF MULTI-DIRECTIONAL SHAKING ON LIQUEFACTION UNDER SIMPLE SHEAR CONDITIONS

A potentially significant difference between the stresses developed on soil elements in the ground during an earthquake and those induced on soil samples in laboratory simple shear tests is the multi-directional nature of the stresses under field conditions compared with the uni-directional nature of cyclic stress applications under laboratory conditions. Accordingly, a series of laboratory investigations have been conducted by Pyke

Chan, and Seed (1974) to determine the significance of this effect.

Since analyses have now been developed (Martin, Finn, and Seed, 1975) to predict the stress conditions causing initial liquefaction of saturated sands from test data on the rate of settlement of the same sand in a dry condition, combined with a knowledge of the rebound and stress-deformation characteristics of the sand, the studies by Pyke, Chan, and Seed were aimed at establishing the difference in settlement characteristics of dry sand under uni-directional and multi-directional shaking conditions. For this purpose, samples of sand three inches deep and with an average diameter of 42 inches were constructed on a shaking table, surrounded by a membrane and fitted with a steel reaction cap as shown schematically in Fig. 3-8 so that cyclic horizontal movements of the table would create cyclic stresses in the test sample. Confining pressures could readily be provided by applying a vacuum to the test specimen.

Samples were tested by subjecting them to pre-programmed random horizontal motions, first in one direction only and then in two directions at right angles (see Fig. 3-9) producing the composite motion characteristics shown in Fig. 3-10. In each test, the settlement was measured as a function of the number of applied stress cycle, and from this data, plots such as Fig. 3-11 could be determined to show the relationship between the applied shear stress ratio τ/σ'_o and the settlement induced in any given number of cycles.

The greater settlements observed in shaking table tests with multi-directional as opposed to uni-directional shaking indicate that pore water pressures will build up more rapidly in the field than is indicated by laboratory tests on saturated samples in which the load is cycled in one direction only. The effect of vertical accelerations in the field may be neglected for satu-

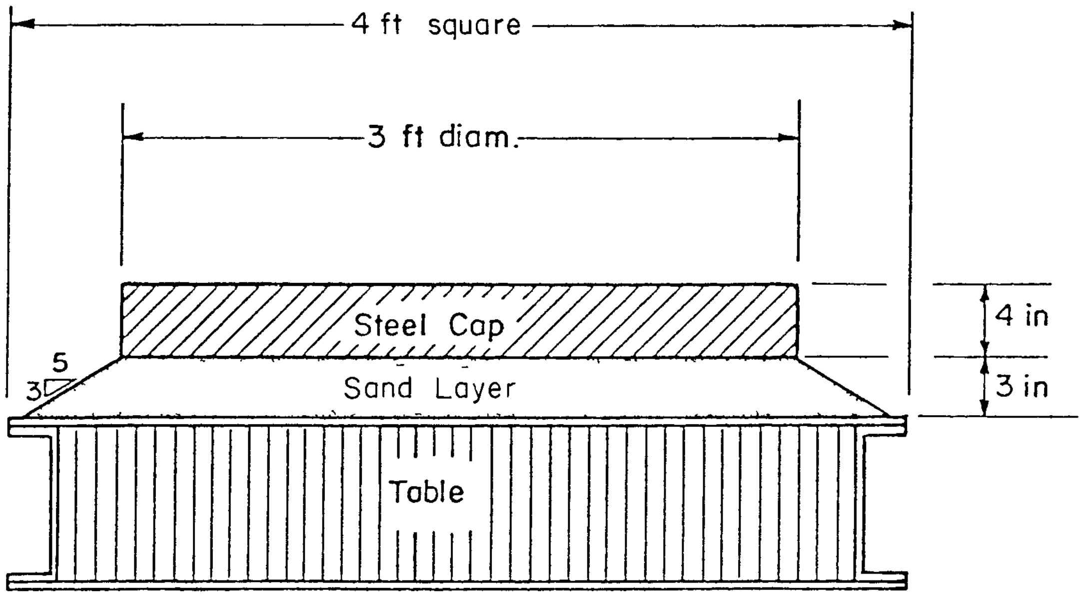


FIG. 3-8 CROSS-SECTION OF TABLE AND SAMPLE

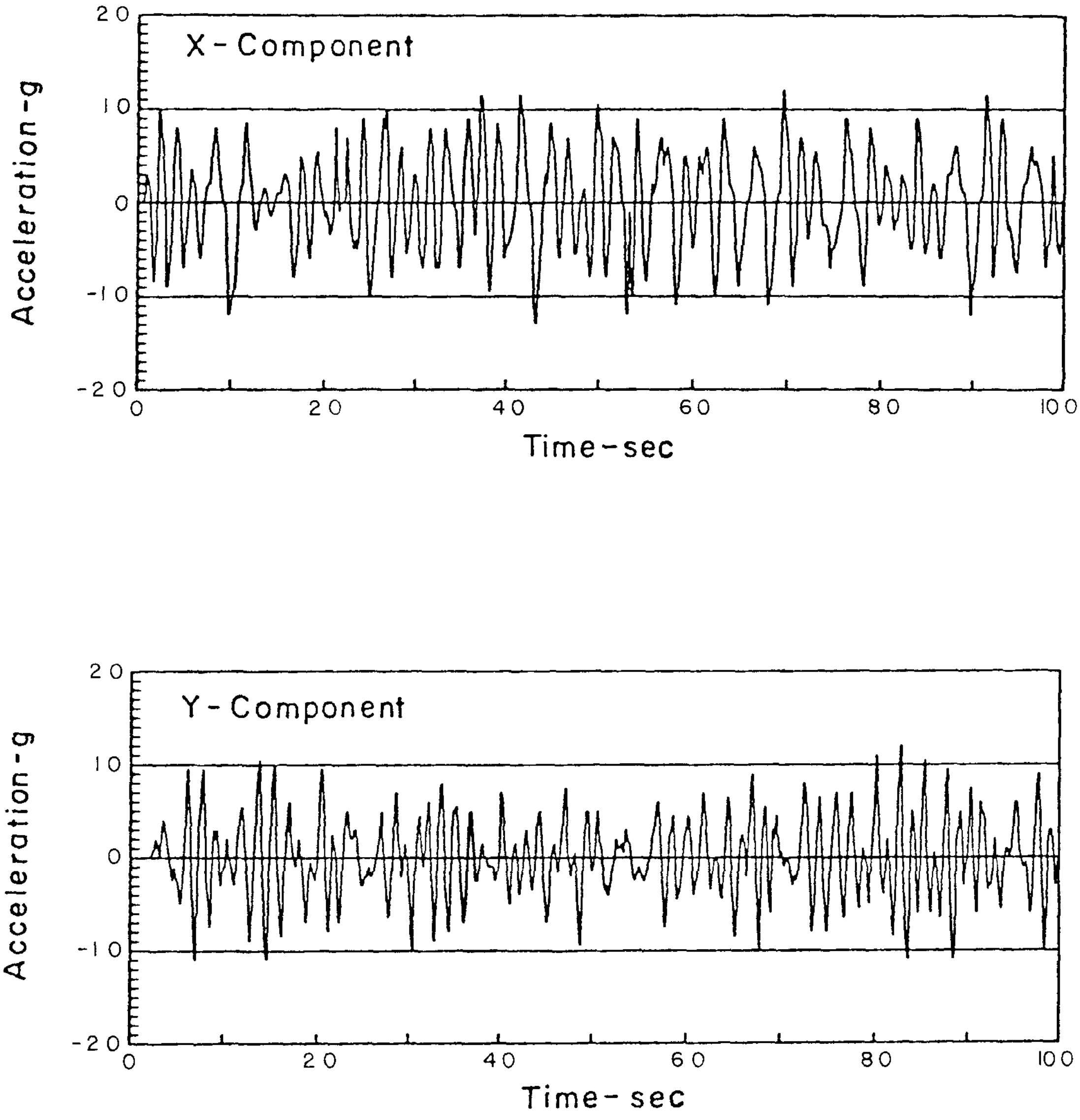
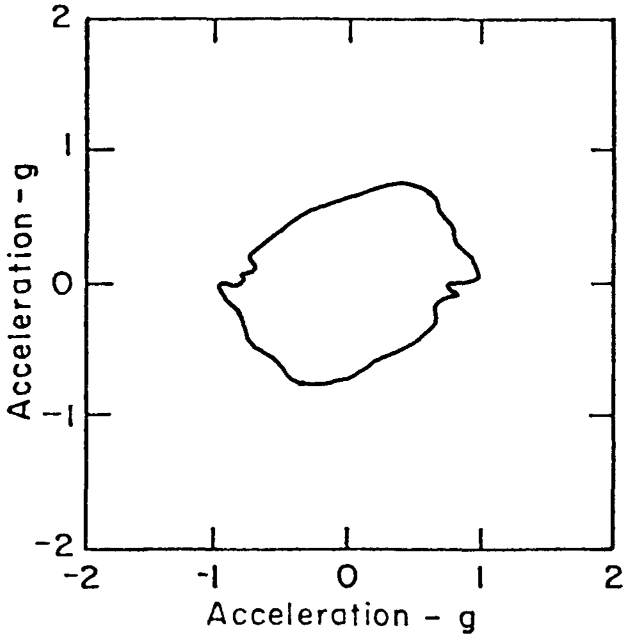
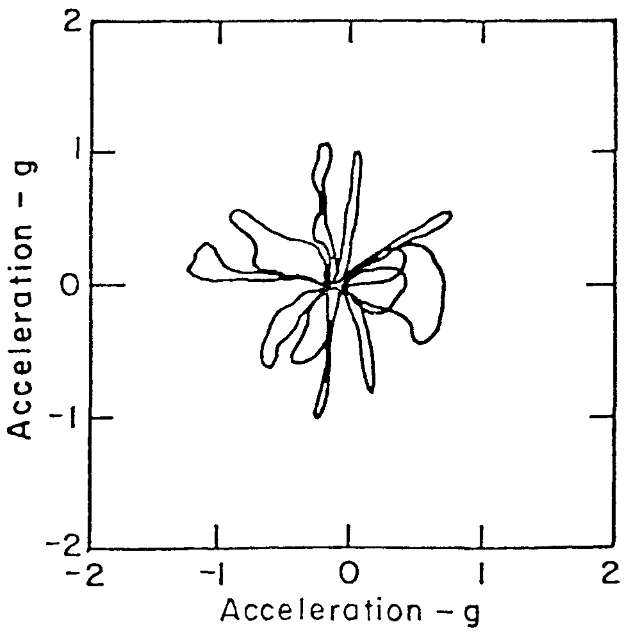


FIG. 3-9 RANDOM ACCELERATION HISTORIES



a) Gyratory



b) Random Motion

FIG. 3-10 RESULTANT ACCELERATIONS IN SHAKING TABLE TESTS IN GYRATORY SHEAR AND RANDOM MOTION

(After Pyke, Chan and Seed, 1974)

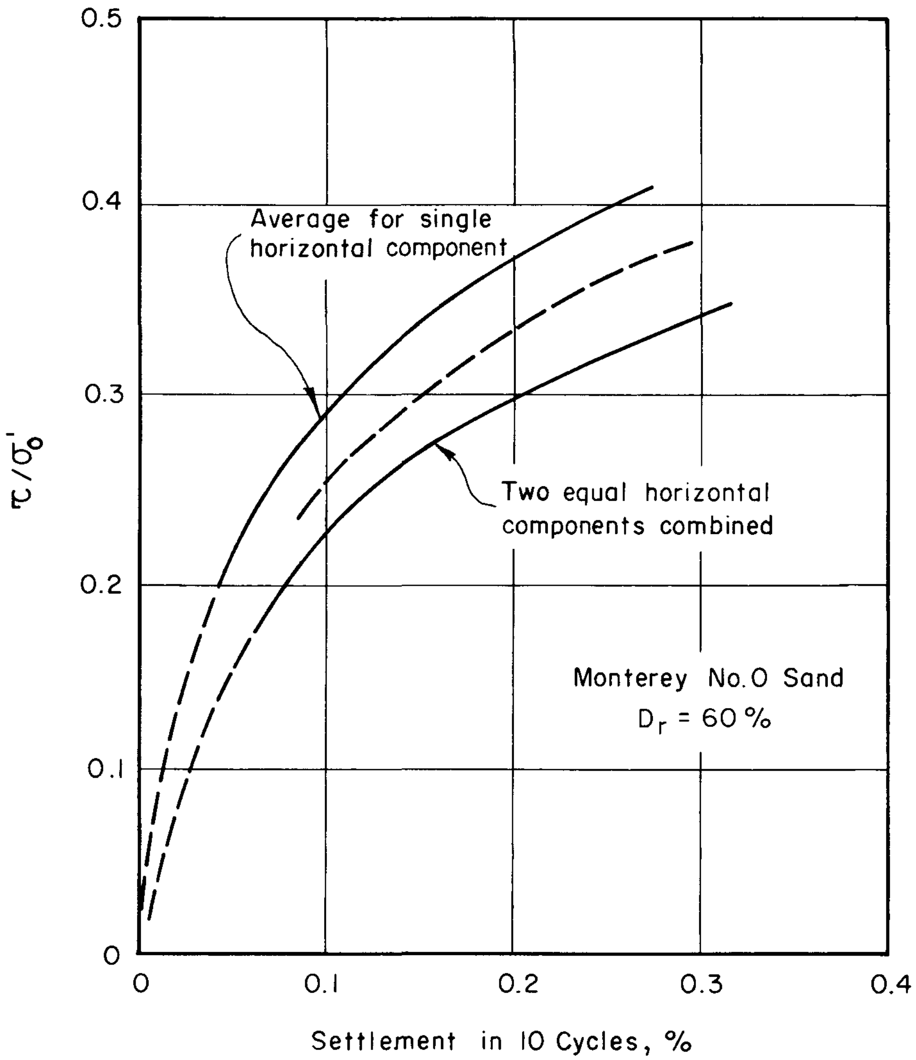


FIG. 3-11 COMPARISON OF STRESS-RATIOS CAUSING GIVEN SETTLEMENTS FOR ONE OR TWO HORIZONTAL MOTIONS

(After Pyke, Chan and Seed, 1974)

rated soils, since they will have virtually no influence on the effective stresses in the soil, but the effects of the two horizontal components of motion should clearly be considered. However, it would be possible to conduct the analysis for shaking in one direction only using the results of presently available laboratory tests if a correction for the effect of the second horizontal component of motion is established. A detailed analysis of such a correction is presented by Pyke, Chan, and Seed (1974).

The effect of the second horizontal component of shaking on the development of pore water pressures leading to liquefaction may be inferred from the results of the shaking table tests on dry sand. It may be shown that there is a certain settlement of dry sand which is equivalent to the development of initial liquefaction (pore water pressure equal to applied confining pressure) in undrained saturated sand. For initial vertical effective stresses of 7 to 22 psi, the settlements of dry sand that are equivalent to the onset of liquefaction in saturated sand are of the order of 0.1 to 0.2 percent (Martin, Finn, and Seed, 1975). Although the stress-ratios used in the presentation of the results of the shaking table tests are not numerically the same as those which would cause liquefaction in the same number of cycles, it may reasonably be assumed that the stress-ratios causing liquefaction are approximately proportional to the values shown in Fig. 3-11. Thus, it may be concluded that the ratio of shear stresses that would cause liquefaction with shaking under two horizontal components as opposed to one component will be about the same as the ratio of the stress ratios causing the same settlement in one- and two-directional shaking table tests.

Based on the data shown in Fig. 3-11, therefore, it would be reasonable to conclude that the stress ratio required to cause initial liquefaction for a ground motion with two equal horizontal components will be about 20 percent less than that required to cause intitial liquefaction with shaking in only one direction. However, statistically, the peak accelerations in two directions at right angles are rarely equal, and if the peak acceleration in one direction is approaching an 85 percentile value, the peak acceleration in the other direction will probably be only about 2/3 of this value, or less, in which case, the settlements due to the combined horizontal components will be those indicated by the dashed line in Fig. 3-11. In this case, the stress ratio required to cause initial liquefaction for the combined horizontal motions would be about ten percent less than that causing initial liquefaction with shaking in only one direction. It would seem appropriate, therefore, to apply a correction factor of this magnitude to test data obtained from one-directional shaking or simple shear tests and to corresponding data obtained from cyclic triaxial tests. This is equivalent to reducing the values of c_r shown in Fig. 3-7 by a further ten percent, leading to the values shown in Fig. 3-12 as being appropriate for two-dimensional shaking conditions. These values range from about 0.59 for motions producing five equivalent cycles to about 0.55 for motions producing about 30 equivalent cycles.

3.4 INFLUENCE OF INITIAL STRESS CONDITIONS ON LIQUEFACTION CHARACTERISTICS

It has been recognized from the earliest stages of cyclic load testing of soils that the initial stress conditions acting on a test specimen have a large influence on the additional stresses to which it can be subjected before developing a condition of initial liquefaction or significant cyclic strains. This is one of the primary reasons for the differences in stress ratios determined in cyclic triaxial and cyclic simple shear tests previously discussed and is attributable to the different initial values of K_o, the coefficient of earth pressure at rest, used in these tests. Correspondingly, it may readily be shown that different initial values of K_o will lead to quite different results in cyclic simple shear tests due to the different stress conditions involved.

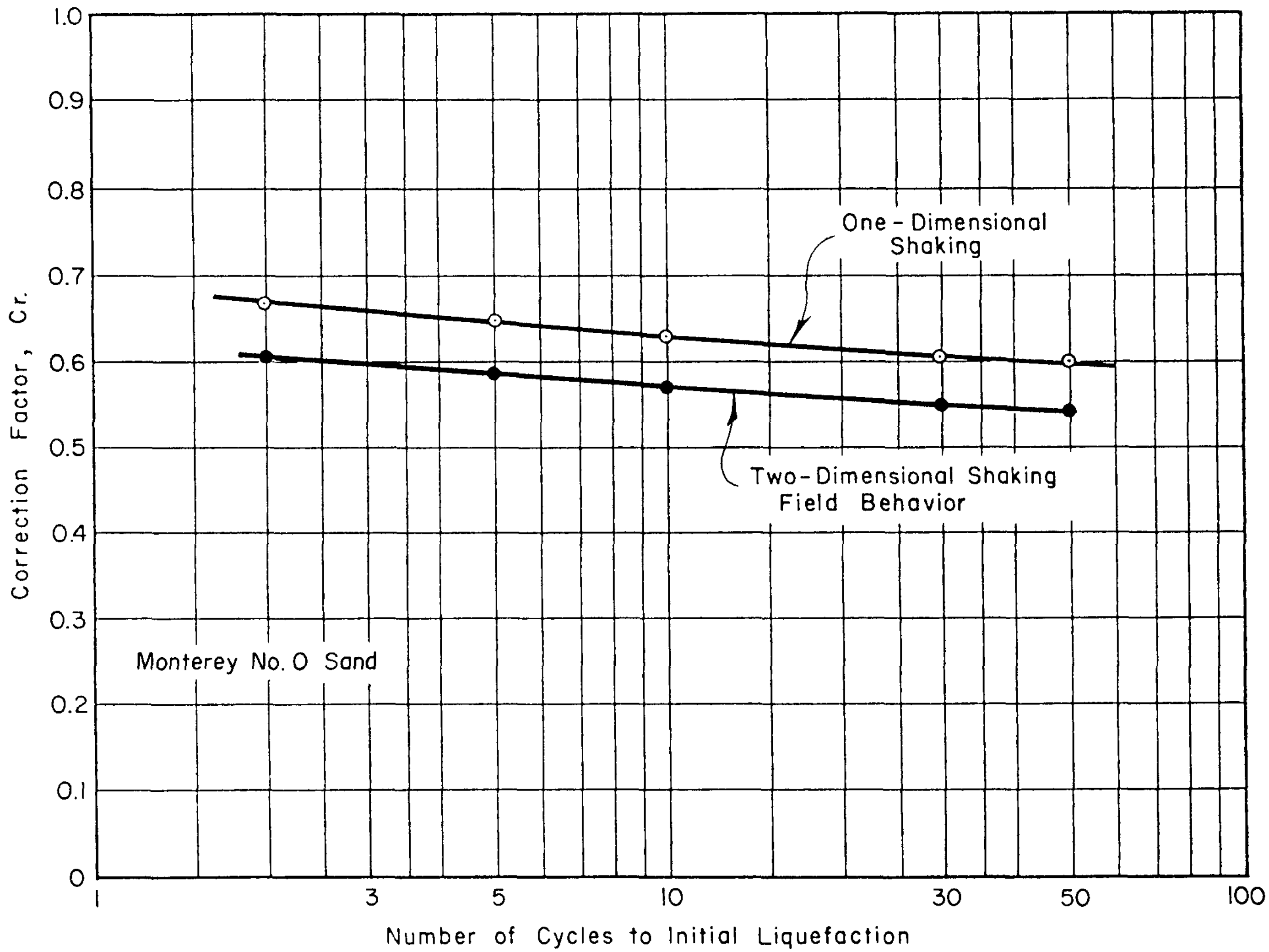


FIG. 3-12 CORRECTION FACTOR FOR CYCLIC TRIAXIAL TESTS ACCOUNTING FOR TWO-DIMENSIONAL SHAKING

In fact, this result has been concluded implicitly or directly by many investigators of the behavior of saturated sands under cyclic loading conditions. Direct experimental evidence of the large effects of K_o on the stress ratios required to cause initial liquefaction and large strains was first provided by Seed and Peacock (1971), who tested samples of saturated sand in a simple shear box after inducing different degrees of overconsolidation, with overconsolidation ratios varying from 1 to 8, to produce different values of K_o, Fig. 3-13. It was found that for values of overconsolidation ratio greater than about 5, the stress ratios required to cause liquefaction were increased by at least 50 percent. Previous work by Hendron (1963) shows that values of overconsolidation ratio of 6 to 8 would be likely to produce values of K_o of 1 or more. Thus, it was concluded that values of OCR sufficiently large to increase the OCR to a value of unity would produce stress ratios in simple shear tests very similar to those obtained in triaxial tests conducted with ambient pressure conditions.

A somewhat similar effect is shown by the cyclic torsional shear and simple shear data presented in Fig. 3-4. Even allowing for some differences in the properties of the sands and methods of sample preparation used in different investigations, the cyclic stress ratios determined by Ishibashi and Sherif (1974) using a value of $K_o = 0.6$ are substantially higher than those determined by other investigators for normally consolidated sands in which K_o was probably closer to 0.4.

In addition to experimental evidence of this type, the results of all analytical studies conducted to determine the relationship between cyclic stress ratios in uni-directional cyclic simple shear tests and cyclic triaxial compression tests have led to the conclusion that the results of these tests would be about the same for conditions where $K_o = 1$. Thus, in the relationship

$$\left(\frac{\tau}{\sigma'_o}\right)_{field} = \alpha \left(\frac{\sigma_{dc}}{2\sigma'_3}\right)_{triaxial}$$

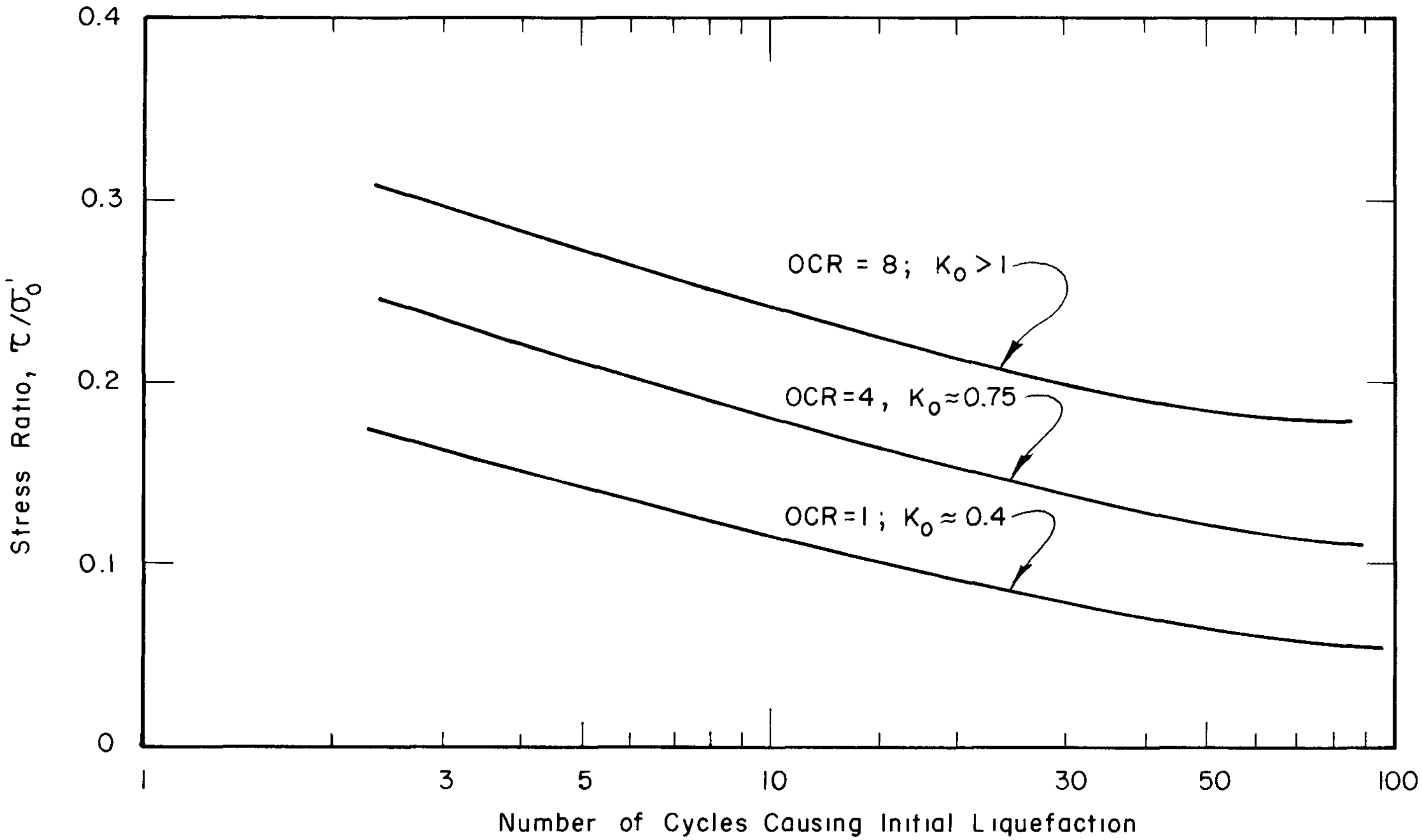


FIG. 3-13 INFLUENCE OF INITIAL PRINCIPAL STRESS RATIO ON STRESSES CAUSING LIQUEFACTION IN SIMPLE SHEAR TESTS

(After Seed and Peacock, 1971)

values of α have been determined as shown in the following table:

Investigator	α	Value of α for $K_o = 1$
Finn, Pickering, & Bransby (1971)	$\frac{1 + 2\,K_o}{3}$	1.0
Seed and Peacock (1971)	Varies	1.0
Castro (1975)	$\frac{2(1 + 2K_o)}{3\sqrt{3}}$	1.15

Accordingly, it is appropriate, on both theoretical and experimental grounds, to use higher values of α for sands known to be overconsolidated, with values of α becoming equal to 1.0 for conditions where the overconsolidation ratio is about 6, a value shown by Hendron to produce a K_o condition of about 1.0. This would indicate a correction factor c_r of 1.0 in applying triaxial test data to field conditions of this type. However, it would still be necessary to reduce this factor by about 10 percent to allow for the effects of multi-directional shaking leading to a correction factor for field conditions of about 0.9.

On this basis, it would seem reasonable to adopt correction factors for triaxial compression test data as shown in Fig. 3-14, with values ranging from $c_r \approx 0.57$ for OCR = 1 to $c_r = 0.9$ for OCR $\approx$ 6, in applying the data to field conditions. This assumes that the correction factor c_r will vary linearly with the overconsolidation ratio, but this assumption would appear to be adequate for all practical purposes at the present time.

Similarly, correction factors c_{ssr} to account for overconsolidation effects in cyclic simple shear tests are presented in Fig. 3-15. The correction factors were developed on the basis of the information presented in Fig. 3-13. As with the triaxial tests, the correction factors are assumed to vary linearly with the overconsolidation ratio.

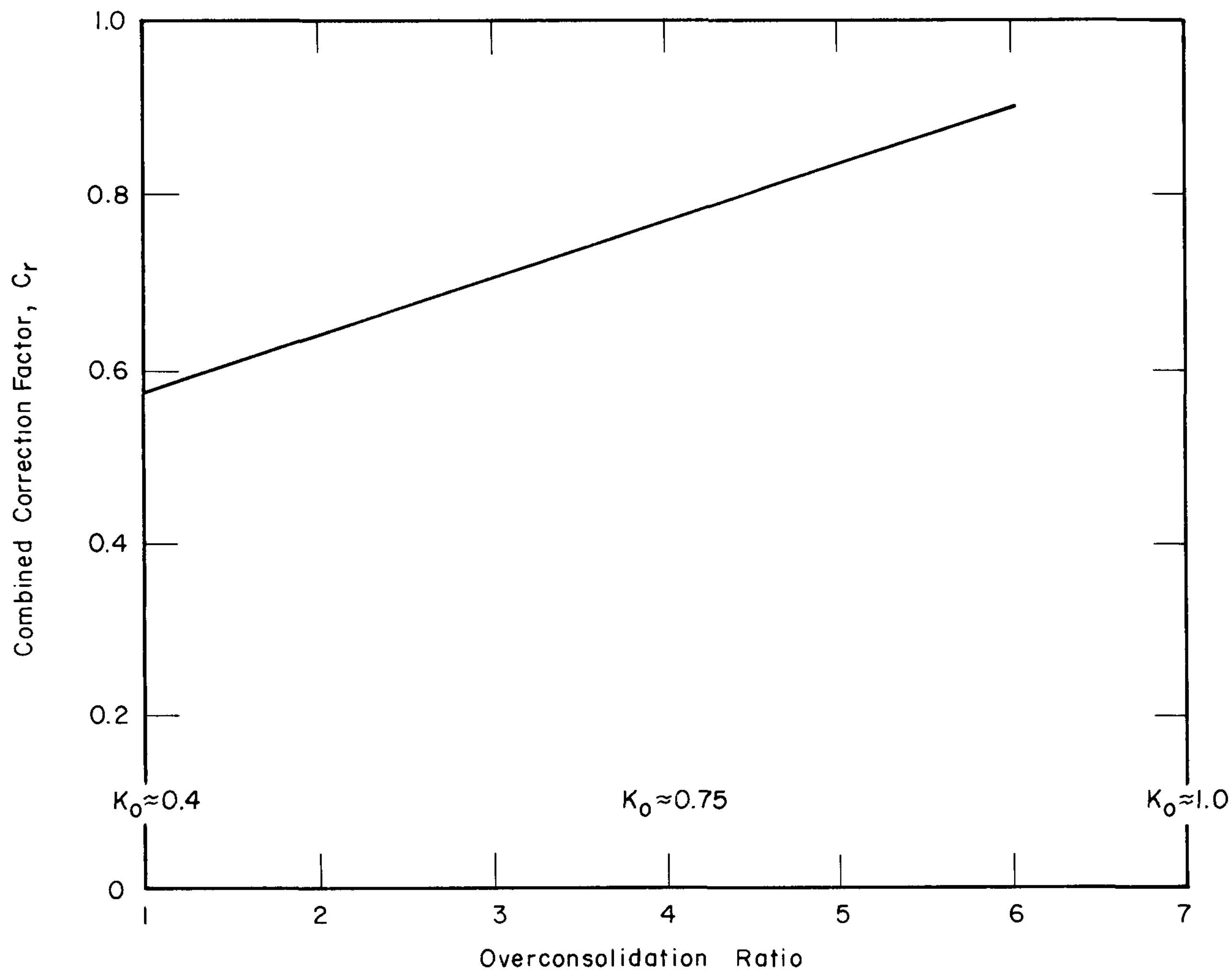


FIG. 3-14 COMBINED CORRECTION FACTOR FOR CYCLIC TRIAXIAL COMPRESSION TESTS ACCOUNTING FOR MULTIDIRECTIONAL SHAKING AND OVERCONSOLIDATION

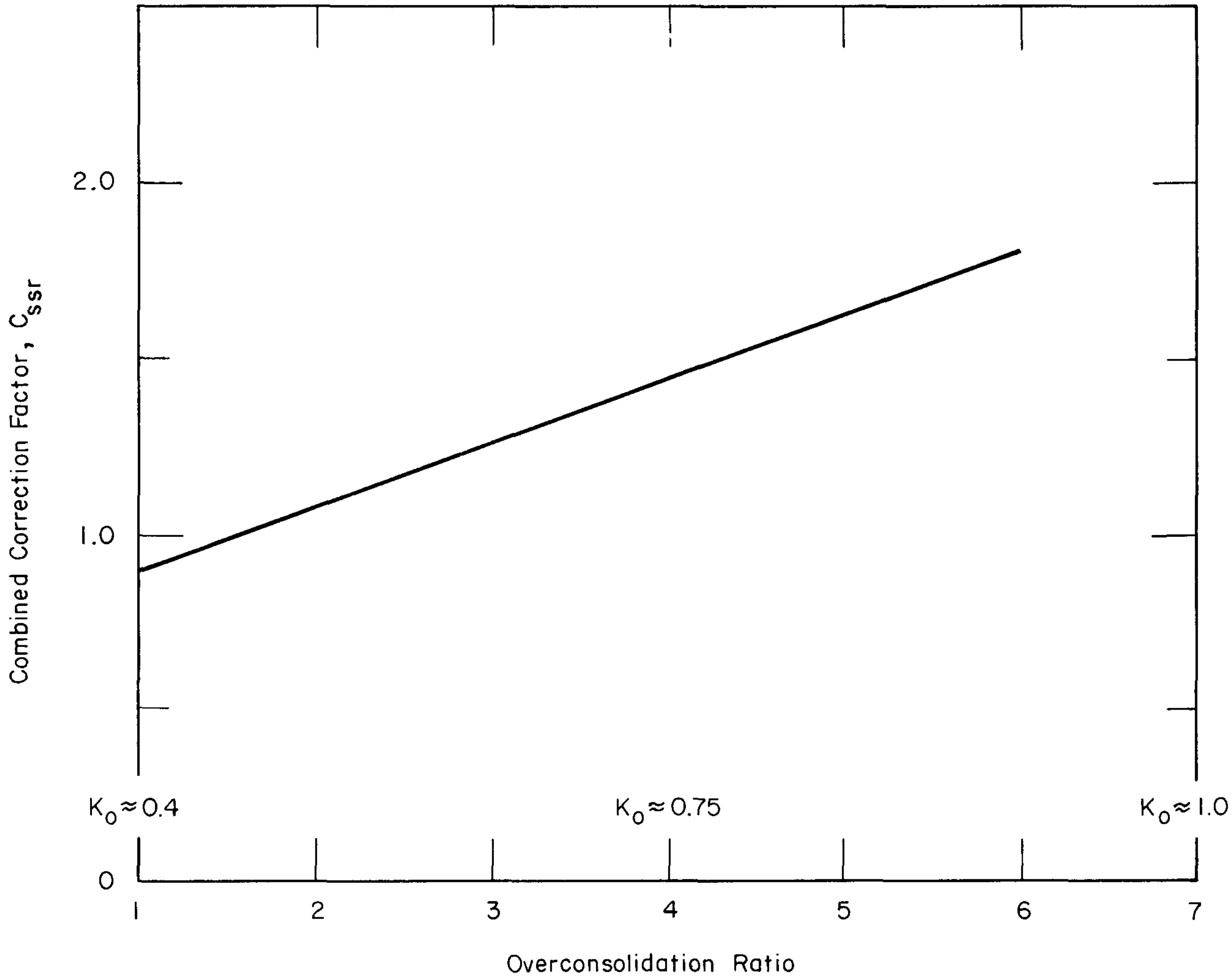


FIG. 3-15 COMBINED CORRECTION FACTOR FOR CYCLIC SIMPLE SHEAR TESTS ACCOUNTING FOR MULTIDIRECTIONAL SHAKING AND OVERCONSOLIDATION

CHAPTER 4

INITIAL LIQUEFACTION WITH LIMITED STRAIN POTENTIAL OR CYCLIC MOBILITY FOR LEVEL GROUND

Two of the most significant deficiencies of the cyclic load triaxial test are:

a. The test cannot be used reliably to investigate the effects of cyclic stress ratios greater than 0.5, since beyond this point, the upward application of the deviator stress tends to lift the cap off the specimen, and stress concentrations then lead to premature failure. To some extent, the cap may succeed in applying a tension force to the top of the sample by suction, but this is of a limited and dubious nature, and the results cannot be considered reliable in this range.

b. The test cannot be used to determine reliably the axial strains resulting from cyclic stress applications once the sample starts to neck during the upward application of a stress cycle. In a well conducted test, the deformations are reasonably symmetrical about the initial height, although there will always tend to be a slightly greater deformation in extension rather than compression. Usually there is no significant tendency for stress concentrations to cause non-uniform deformations until at least a condition of initial liquefaction and cyclic strains exceeding about ± 2.5 percent have been reached. Thus, the data are reasonably reliable and consistent up to this point. For loose samples, deformations increase so rapidly beyond this stage that the accuracy of their rate of increase is not usually significant. However, for dense samples, the specimen will usually tend to neck soon after initial liquefaction of strains of about ±2.5 percent are reached, and once necking occurs, the stress conditions in the sample can no longer be determined and the extension strains become totally unreliable. Thus, there is no way to measure accurate values of axial strains in excess of about ±2.5 percent for dense samples. In some cases, this necking may occur near the top of the sample (Castro, 1975), but this is not always the case. Regardless of where it occurs, however, strain amplitudes beyond this point become unreliable. It should be noted, however, that measurements of non-uniformities developed in test specimens after this stage is reached are not indicative of any potential errors in data up to this point, and furthermore, since necking is the primary cause of the errors, this deficiency does not occur in tests using anisotropic consolidation of triaxial test specimens, such as those used for conditions below sloping ground surfaces and strains are primarily compressive.

Nevertheless, it was the recognition of the above deficiencies and the need to test denser materials under conditions producing larger strains that influenced the development of cyclic simple shear and cyclic torsional shear tests of various types. While few of these have explored the strains developed in denser samples, the large-scale study by De Alba, Chan, and Seed (1975) has provided data on the behavior of dense sands at relatively high stress ratios. In particular, the tests provided clear data to show that for sand at any given relative density, there was apparently a limited amount of shear strain that could be developed, regardless of the magnitude of the applied stress ratio or the number of stress cycles. Typical results for tests on Monterey No. 0 sand at different relative densities are shown in Figs. 4-1 and 4-2. As may be seen from the figures, at relative densities less than about 45 percent, the application of cyclic stress ratios sufficiently high to cause initial liquefaction

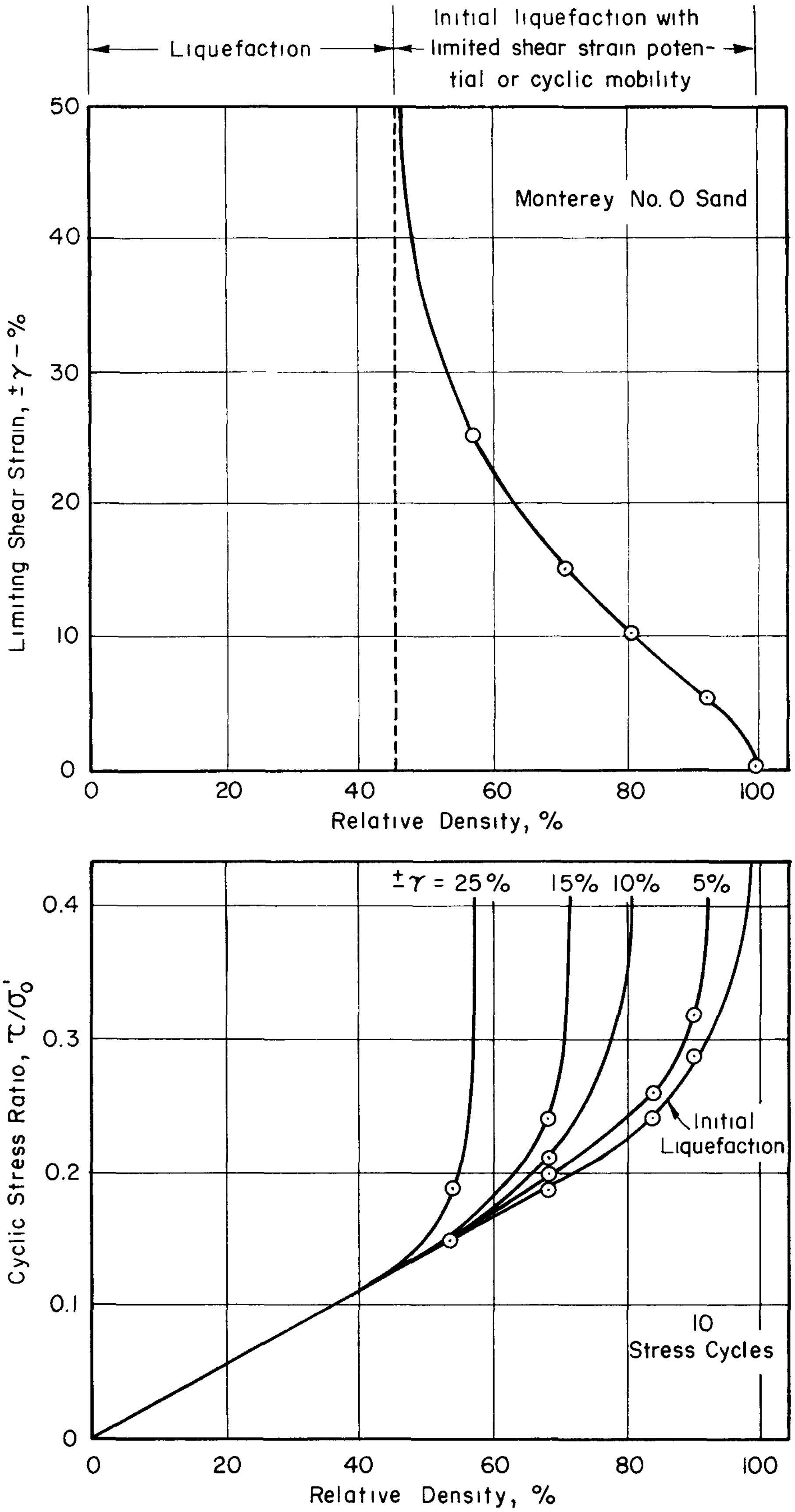


FIG. 4-1 LIMITING SHEAR STRAINS - 10 STRESS CYCLES

(After De Alba, Chan and Seed, 1975)

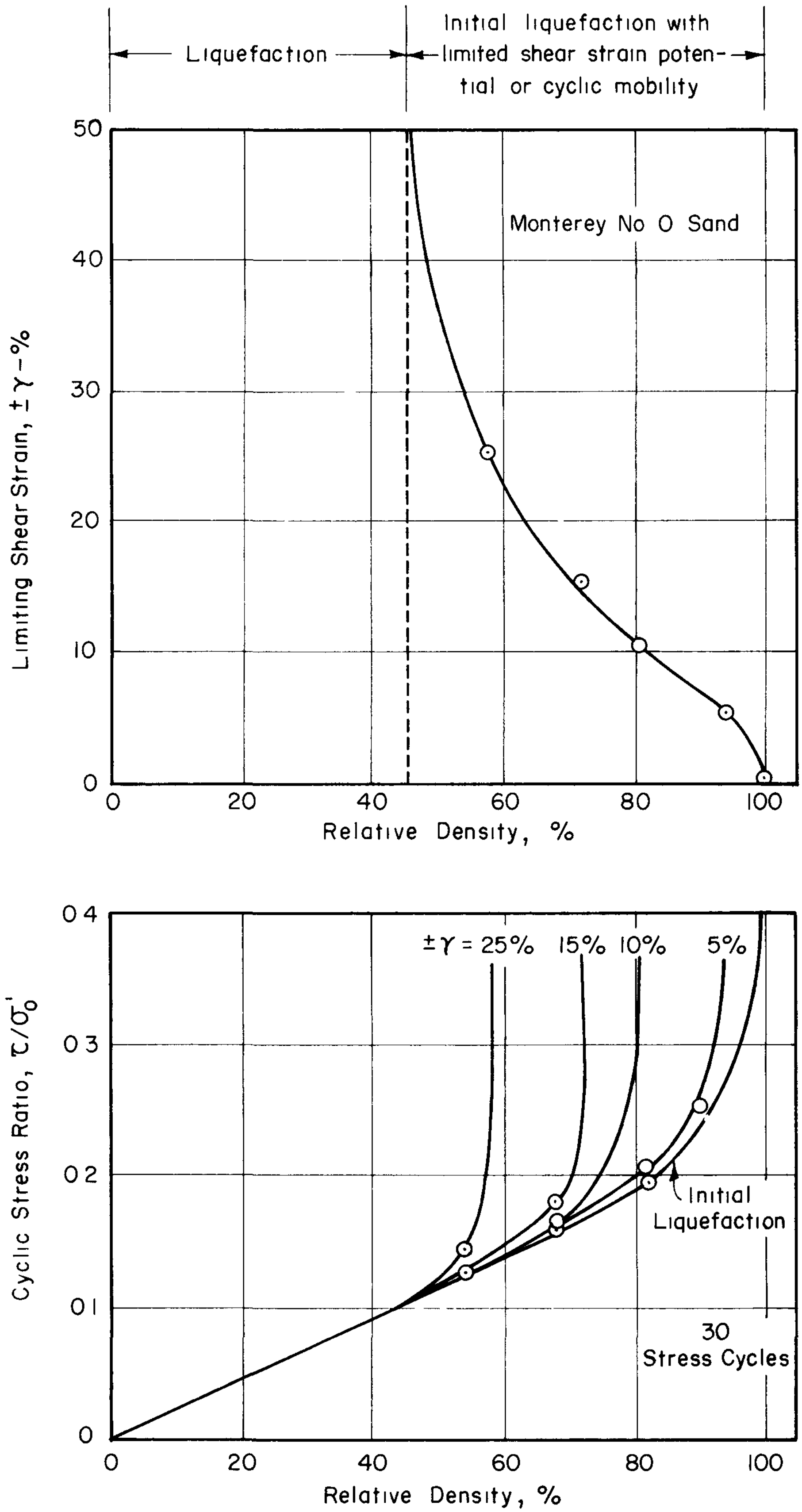


FIG. 4-2 LIMITING SHEAR STRAINS - 30 STRESS CYCLES

(After De Alba, Chan and Seed, 1975)

will also cause extremely high and probably unlimited strains in the soil. This corresponds to a condition of liquefaction. However, for relative densities greater than about 45 percent, the application of stress ratios and numbers of cycles sufficiently high to cause initial liquefaction would result in only a limited amount of shear strain, the limiting strain potential decreasing with increasing relative density. Thus for example, the limiting strain potential for a sample with a relative density of 50 percent might be about ±35 percent, but a sample with a relative density of 90 would have a limiting strain potential of only about ±6 percent.

The condition of "initial liquefaction with limited strain potential" as used above is directly analagous to the condition of "cyclic mobility" used by Casagrande and Castro (1975). However, the authors prefer the use of the former term for several reasons:

a. "Cyclic mobility" does not serve to indicate that high residual pore water pressures exist in the soil, whereas "initial liquefaction" clearly indicates such a condition.

b. "Cyclic mobility" covers a wide range of conditions with potential strains ranging from almost zero to many tens of percent. Thus under some situations, a condition of cyclic mobility may be perfectly acceptable, whereas in others, it would be totally unacceptable. A statement that a soil is in a condition of "initial liquefaction with a limiting strain potential of X percent" seems to provide a more specific and graphic description of the situation than a statement that the soil is cyclically mobile.

However, in the long run, it matters little which terminology is used so long as the phenomena are understood and used in the same manner throughout the profession. It is hoped that the above explanation will serve to clarify any misunderstandings which may have arisen through the use of different terminology by different investigators and emphasize that there is in fact apparently a high degree of agreement on many aspects of the soil liquefaction phenomenon.

A special word of caution would seem to be in order, however, in discussing the possibility of "liquefaction" or "initial liquefaction with a limiting shear strain potential" for sands having a relative density of say 60 percent. Test data for such sands obtained by tests on relatively uniform samples under laboratory conditions clearly indicate that they may have a limiting strain potential of about 20 to 25 percent (see Figs. 4-1 and 4-2). In the field, however, such a sand may be overlain by a considerably more impervious and fine-grained deposit so that high pore water pressures equal to the overburden pressure can build up at the contact boundary. In such cases, it is doubtful that dilation of the sand during shear would be able to reduce the pore water pressure since deformations could take place entirely along the contact surface with no accompanying dilation, leading to the appearance of liquefaction and flow even though a homogeneous deposit of the sand involved would be incapable of such field performance. It is interesting to note that many cases of slides due to liquefaction during earthquakes, involving large lateral translations of soil masses, have occurred in stratified deposits of sand and finer-grained soils (Seed, 1968; Seed, Martin, and Lysmer, 1975).

CHAPTER 5

FACTOR OF SAFETY IN EVALUATING LIQUEFACTION POTENTIAL

In evaluating the liquefaction potential of a saturated sand deposit under some postulated earthquake condition, it is customary to express the result in terms of a factor of safety expressed as:

$$\text{Factor of Safety} = \frac{\begin{array}{c}\text{Uniform shear stress required to cause}\\ \text{initial liquefaction or an acceptable}\\ \text{limit of strain in N cycles}\end{array}}{\begin{array}{c}\text{average shear stress induced by}\\ \text{earthquake for N cycles}\end{array}}$$

This requires determinations of the stresses induced by the earthquake and the stresses which must be applied to the sand to cause initial liquefaction (or some selected degree of strain if this is considered more appropriate).

If the earthquake motions are specified at the ground surface, then the stresses developed in the upper 40 feet of a soil deposit can be assessed (Seed and Idriss, 1971). The preceding pages have discussed at length the procedures required to make a good assessment of the stresses required to cause initial liquefaction or a given degree of strain. The final acceptable factor of safety will clearly depend on the accuracy with which each of these individual assessments can be made in any given case.

The discussion of "limiting strain potential" in Chapter 4 of this report emphasizes a further consideration which must be taken into account in determining what value constitutes an acceptable factor of safety; that is, the consequences arising, if for some reason the actual factor of safety should be reduced to unity. Clearly, this is very different in the case of a loose sand with a relative density of about 55 percent and the same sand in a dense condition, say with a relative density of 82 percent. It may be seen from Figs. 4-1 and 4-2 that the limiting strain for Monterey No. 0 sand at 54 percent relative density is ±30 percent, while the limiting strain for the same sand at 82 percent relative density is only ±10 percent. The stress conditions producing these conditions are shown graphically in Fig. 5-1. It is apparent that if the stress ratio causing five percent strain at a relative density of 54 percent is ever slightly exceeded, then the sand will undergo strains up to ±30 percent with almost certain catastrophic consequences. However, if the stress ratio causing five percent strain at a relative density of 82 percent is slightly exceeded, the only result would be to cause a strain of perhaps six percent and no more than ten percent even if the factor of safety should drop to 0.5 or even 0.2.

This difference in consequences, if for any reason the actual factor of safety reaches a value of unity, is clearly an important factor in determining an allowable factor of safety and in many cases warrants the use of lower factors against initial liquefaction or low strains in dealing with dense sands than in dealing with loose sand deposits.

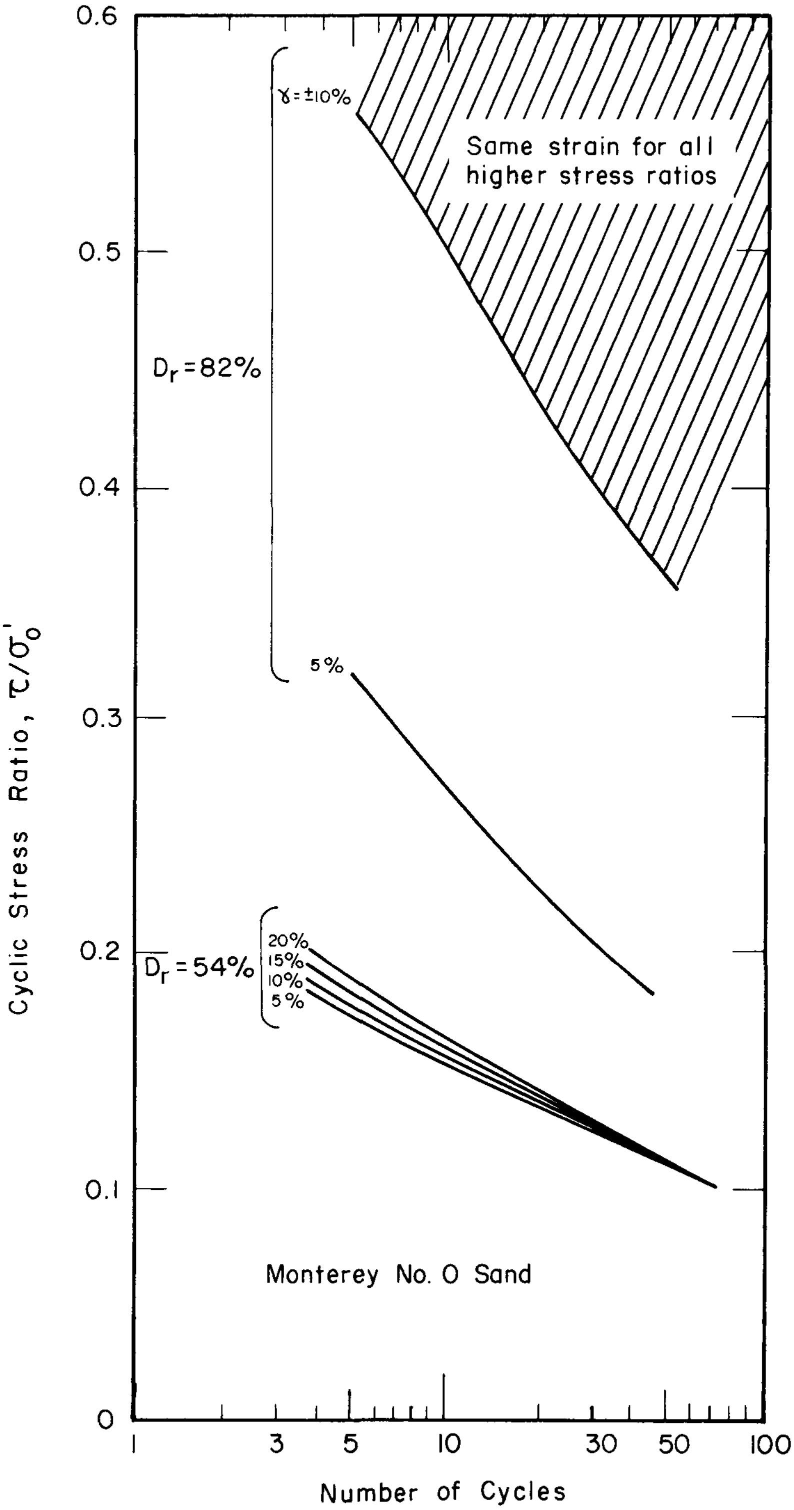


FIG. 5-1 RELATIONSHIP BETWEEN τ/σ_0' AND NUMBER OF CYCLES CAUSING DIFFERENT STRAIN LEVELS

CHAPTER 6

USE OF LARGE-SCALE TEST DATA TO SUPPLEMENT KNOWN CASES OF LIQUEFACTION IN THE FIELD

It was shown previously in this report that there is a scarcity of reliable field data concerning the liquefaction potential of sands with high densities or penetration resistance values subjected to high cyclic stress ratios by earthquake ground motions. Since there has been no opportunity to collect data of this type from the field, it would seem desirable that some attempt be made to explore the possibility of obtaining the data by means of laboratory tests which closely simulate field conditions.

The data obtained by De Alba, Chan, and Seed (1975) in large-scale simple shear tests provide a basis for studies of this type, provided they are suitably modified for other significant factors known to affect the results under field conditions.

Thus for example, the data presented by De Alba, Chan, and Seed (1975) were obtained by uni-directional simple shear tests on samples of Monterey No. 0 sand deposited by pluvial compaction and tested under a confining pressure of 8 psi. The data in Table 2-3 show that stress ratios causing initial liquefaction or prescribed strains for undisturbed samples are conservatively about 15 percent higher than those of samples prepared by moist tamping, and the data by Mulilis show that for Monterey sand, samples prepared by moist tamping are about 60 percent stronger than those prepared by pluvial compaction. Thus, if the simple shear tests had been performed on undisturbed field samples of Monterey sand, the cyclic stress ratios causing liquefaction would probably have been higher than those measured by a factor of 1.6 x 1.15 or about 1.85.

At the same time, cyclic load tests are normally conducted at confining pressures higher than 8 psi, and data by Mulilis, Chan, and Seed (1975) show that the stress ratios causing liquefaction at pressures of about one ton per square foot would be about 10 percent less than those at a pressure of 8 psi, while for two components of motion, as developed in the field, the stress ratios causing liquefaction would be reduced by an additional 10 percent. Thus, cyclic stress ratios causing liquefaction at confining pressures of about one tsf in the field would require that the data by De Alba, Chan, and Seed (1975) be multiplied by a correction factor of $1.85 \times 0.9 \times 0.9 \simeq 1.5$.

Finally, it should be noted that Monterey sand is by no means the poorest type of sand from a liquefaction point of view, and available data indicate that stress ratios causing liquefaction of some sands under comparable conditions may be about 15 percent less than those for Monterey sand. Thus, a lower bound value of cyclic stress ratios causing liquefaction of natural deposits of sands under field conditions might be obtained by applying a final correction factor of $1.5 \times 0.85 \simeq 1.28$ to the test data by De Alba, Chan, and Seed (1975) shown in Fig. 3-1. This would lead to the following results for the lower bound stress ratios causing initial liquefaction.

	5 Cycles		15 Cycles	
Relative Density	$(\frac{\tau}{\sigma'_o})_{test}$	$(\frac{\tau}{\sigma'_o})_{field}$	$(\frac{\tau}{\sigma'_o})_{test}$	$(\frac{\tau}{\sigma'_o})_{field}$
54	0.17	0.22	0.135	0.17
68	0.215	0.275	0.17	0.22
82	0.29	0.37	0.22	0.28
90	0.35	0.45	0.26	0.33

Five uniform stress cycles might be considered representative of earthquakes with magnitude ranging from 5 to 6, and 15 uniform cycles might be considered representative of earthquakes with magnitudes of 7.0 to 7.5 (Seed, Idriss, Madkdisi, and Banerjee, 1975).

In order to relate these results to those for field cases of liquefaction shown in Fig. 1-5, it is necessary to establish a relationship between the relative densities of samples deposited by pluvial compaction and the corrected penetration resistance N_1. The only available basis for determining such a relationship at the present time appears to be the correlation between Standard Penetration Resistance values and relative density proposed by Gibbs and Holtz (1957), Gibbs (1971), Bazaara (1967), and Schultze and Melzer (1965). Based upon these results, it might be estimated that the relationship between relative density and Standard Penetration Resistance under an overburden pressure of one ton per square foot would be approximately as follows:

Relative Density of Sand Deposited by Pluvial Compaction	Standard Penetration Resistance under Overburden Pressure of 1 ton/sq. ft., N_1
54	12
68	19
82	26
90	30

Combining these results with those determined above leads to the following approximate lower bound correlation between the stress ratio likely to cause liquefaction in the field and the corrected penetration resistance N_1:

	Stress Ratios, τ/σ'_o, Causing Initial Liquefaction under Field Conditions	
N_1 - blows/ft.	M = 5 to 6	M = 7 to 7.5
12	0.22	0.17
19	0.275	0.22
26	0.37	0.28
30	0.45	0.33

These values, superimposed on the field data from Fig. 1-5, are shown in Fig. 6-1. It may be seen that they are in reasonable agreement with conditions known to cause liquefaction in the field and provide a basis for establishing a lower bound curve for cyclic stress ratios causing initial liquefaction for sands with high penetration resistance values. The initial liquefaction conditions of such sands will clearly be such that they will only be accompanied by limiting shear strains, the latter depending upon the relative density of the sand, and for ease of reference, the limiting shear strains determined for Monterey sand are plotted in the upper part of Fig. 6-1, again using the Gibbs and Holtz correlation between relative density and penetration resistance to establish the N_1 values.

Thus the data in this figure might well be used as a summary of past field performance concerning liquefaction and as a guide to probable future performance. Supplemented by detailed evaluations of stress conditions and liquefaction characteristics at any given site, it provides a basis for an overall evaluation of probable performance.

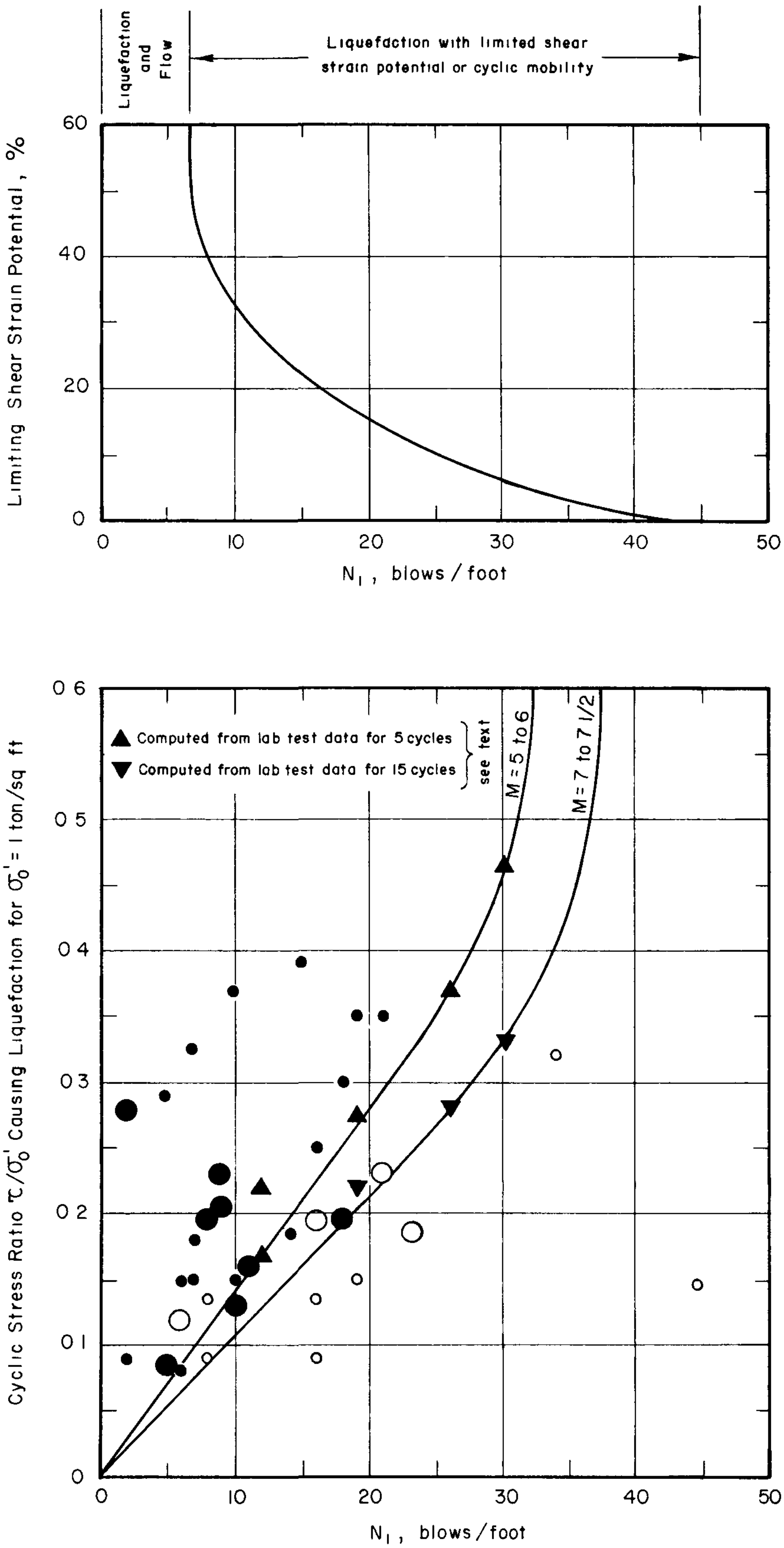


FIG. 6-1 CORRELATION BETWEEN STRESS RATIO CAUSING VARIOUS DEGREES OF LIQUEFACTION IN THE FIELD AND IN THE LABORATORY AND PENETRATION RESISTANCE OF SAND

CHAPTER 7
SUMMARY AND CONCLUSIONS

7.1 SUMMARY

7.1.1 Definitions

The term "liquefaction" as used in this report describes a phenomenon in which a cohesionless soil loses strength during an earthquake and acquires a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet.

It is now generally recognized that the basic cause of liquefaction of saturated cohesionless soils during earthquakes is the build-up of excess hydrostatic pressures due to the application of cyclic stresses induced by the ground motions. These stresses are generally considered to be due primarily to upward propagation of shear waves in a soil deposit, although other forms of wave motions are also expected to occur. As a consequence of the applied cyclic stresses, the structure of the cohesionless soil tends to become more compact with a resulting transfer of stress to the pore water and a reduction in stress on the soil grains. As a result, the soil grain structure rebounds to the extent required to keep the volume constant, and this interplay of volume reduction and soil-structure rebound determines the magnitude of the increase in pore water pressure in the soil.

In an effort to clarify differences in terminology, the following qualifications of the term liquefaction were introduced:

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

b. "Initial Liquefaction with Limited Strain Potential" or "Cyclic Mobility." Denotes a condition in which cyclic stress applications develop a condition of initial liquefaction, and subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil dilates, the pore pressure drops and the soil stabilizes under the applied loads. It should be noted, however, that once the cyclic stress applications stop, if they return to a zero stress condition, there will be a residual pore water pressure in the soil equal to the overburden pressure, and this will inevitably lead to an upward flow of water in the soil which could have deleterious consequences for overlying layers.

c. "Liquefaction." Denotes a condition where a soil will undergo continued deformation at a constant low residual stress or with no residual resistance, due to the build-up of high pore water pressures which reduce the effective confining pressure to a very low value; pore pressure build-up may be due either to static or cyclic stress applications.

7.1.2 Methods for Evaluating the Liquefaction Potential of Sand Deposits (Level Ground)

There are basically two methods available for evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

a. <u>Method Based on Observations of Performance of Sand Deposits in Previous Earthquakes</u>. Post earthquake surveys of the areas where liquefaction has or has not occurred have been used to prepare charts based primarily on the Standard Penetration Resistance of the deposit for differentiating between liquefiable and non-liquefiable conditions. Since empirical comparisons and evaluations of this type take no account of significant factors, such as the duration of shaking or the possibility of drainage, and depend upon the reliability of field measurements of penetration resistance, which, in the opinion of many engineers, is open to serious question, it appears to be the general belief among most engineers that while such correlations can provide useful preliminary evaluations of liquefaction potential, they will often need to be supplemented by detailed studies based on ground response analyses and detailed soil testing programs in order to arrive at a meaningful evaluation of the liquefaction potential of any particular site.

b. <u>Method Based on Evaluation of Stress Conditions in the Field and Laboratory Determinations of the Stress Conditions Causing Liquefaction of Soils</u>. Analytical procedures for evaluating the liquefaction potential of soil deposits involve two independent determinations: 1) an evaluation of the cyclic stresses induced at different levels in the deposit by the earthquake shaking and 2) a laboratory investigation to determine the cyclic stresses which, for given confining pressures representative of specific depths in the deposit, will cause the soil to liquefy or undergo various degrees of cyclic strain. The evaluation of liquefaction potential is then based on a comparison of the cyclic stresses induced in the field with the stresses required to cause liquefaction or an acceptable limit of cyclic strain in representative samples in the laboratory.

The cyclic stresses induced in the ground by an earthquake may be computed by a ground response analysis, by a simplified procedure based on a knowledge of the maximum ground surface acceleration, or by deconvolution of a known ground surface motion.

Various types of laboratory test equipment and procedures have been used to investigate the cyclic stress conditions required to cause liquefaction or initial liquefaction with limited strain potential of saturated sands. These include cyclic simple shear, cyclic torsional shear, shaking table, and cyclic triaxial tests.

It has been generally recognized since the advent of cyclic load testing that virtually all types of tests are subject to sóme degree of error due to equipment limitations.

Furthermore, another important aspect of a cyclic load test program for use in design concerns the selection of representative samples for testing purposes. In the early stages of cyclic load testing, it was generally recognized that the liquefaction characteristics of any given sand varied greatly depending on its density or relative density, but the possible effects of other factors, such as seismic or geologic history, soil structure, or method of sample preparation, were not considered likely to affect the results significantly.

However, detailed studies of soil liquefaction conducted during the past three years at the University of California, Berkeley, under the sponsorship of the U.S. Nuclear Regulatory Commission, together with the results of other investigations, have led to the following conclusions.

7.2 CONCLUSIONS

The behavior of a saturated sand under cyclic loading conditions is a function of its geologic and seismic history and grain structure as well as its placement density.

There is strong evidence that the liquefaction resistance of undisturbed samples is substantially higher than that of freshly deposited laboratory samples at the same density.

The resistance to liquefaction of a sand deposit can best be estimated by laboratory testing of undisturbed samples recovered from the field. This inevitably raises the question of the ability of existing sampling procedures to obtain good quality undisturbed samples of sand and the possible errors introduced if samples are disturbed to some extent in the sampling and handling process. In fact, it seems likely that procedures vary widely in their adequacy in this respect, and the nature of the sampling process requires careful evaluation in assessing the quality of test data obtained from the resulting samples.

If samples prepared in the laboratory are to have the same characteristics as a soil deposit in the field, they must be prepared in a manner producing the same density and grain structure and tested in such a way that the in-situ value of K_o can be taken into account in assessing their field performance.

The creation of a soil structure similar to that of the field deposit is only possible if the structure of the field deposit can be determined, and measurements of grain structure are by no means a standard procedure in soil mechanics laboratories. However, measurement of the "formation factor" for a sand is a relatively rapid and inexpensive procedure, and this method of measuring particle arrangement or fabric may provide a practical means for determining the structure of sands for a variety of purposes. Further studies of the potential usefulness of this index of structure are required, however, before it could be recommended for adoption as a practical tool for design studies.

Laboratory shaking table test results from samples large enough to be essentially free of undesirable boundary effects and to which an appropriate membrane compliance correction has been applied show that most shake table test results previously reported in the literature are significantly influenced by these factors. The different results from this and other investigations clearly indicate that care is required to provide correct boundary conditions if meaningful data are to be obtained by means of shaking table studies. Furthermore, this method of testing cannot be used for undisturbed samples.

Carefully conducted small-scale simple shear or torsional shear tests using good quality equipment can provide data comparable to those obtained with large-scale test samples. However, it is difficult to test undisturbed samples in these types of tests.

Small-scale, properly conducted, simple shear laboratory tests seem to provide a sufficiently reliable basis for correcting the results of cyclic triaxial compression tests to obtain stress ratios corresponding to one-directional simple shear conditions on the same material and at the same maximum confining pressure.

It appears that carefully conducted cyclic triaxial tests, used in conjunction with appropriate correction factors, can provide valid data on cyclic loading characteristics up to initial liquefaction and strains of the order of about 5 percent for dense samples or 20 percent for loose samples. Reliable data cannot be obtained, however, once necking occurs in any test specimen or if non-uniform conditions exist in the initial sample placement in the triaxial cell.

Based on the information developed, it is reasonable to conclude that the stress ratio required to cause initial liquefaction for a ground motion with two equal horizontal components will be about 20 percent less than that required to cause initial liquefaction with shaking in only one direction. However, statistically, the peak accelerations in two directions at right angles are rarely equal, and if the peak acceleration in one direction is approaching an 85 percentile value, the peak acceleration in the other direction will probably be only about 2/3 of this value, or less. In this case, the stress ratio required to cause initial liquefaction for the combined horizontal motions would be about ten percent less than that causing initial liquefaction with shaking in only one direction. It would seem appropriate, therefore, to apply a correction factor of this magnitude to test data obtained from one-directional shaking or simple shear tests and to corresponding data obtained from cyclic triaxial tests.

The results of all analytical and experimental studies conducted to determine the relationship between cyclic stress ratios in uni-directional cyclic simple shear tests and cyclic triaxial compression tests have led to the conclusion that the results of these tests would be about the same for conditions where $K_o=1$. Correspondingly, it may be shown that different initial values of K_o will lead to quite different results in cyclic simple shear tests due to the differences in the stress conditions involved. It is, therefore, appropriate, on theoretical and experimental grounds, to apply correction factors to the triaxial shear test results which vary with the degree of overconsolidation of the material tested.

Typical large shake table tests on Monterey No. 0 sand indicate that at relative densities less than about 45 percent, the application of cyclic stress ratios sufficiently high to cause initial liquefaction will also cause extremely high and probably unlimited strains in the soil. This corresponds to a condition of liquefaction. However, for relative densities greater than about 45 percent, the application of stress ratios and numbers of cycles sufficiently high to cause initial liquefaction would result in only a limited amount of shear strain, the limiting strain potential decreasing with increasing relative density. Thus for example, the limiting strain potential for a sample with a relative density of 50 percent might be about ± 35 percent, but a sample with a relative density of 90 percent would have a limiting strain potential of only about ± 6 percent.

The condition of "initial liquefaction with limited strain potential" as used above is directly analagous to the condition of "cyclic mobility" used by Casagrande and Castro (1975). However, the authors prefer the use of the former term for several reasons: 1) "cyclic mobility" does not serve to indicate that high residual pore water pressures exist in the soil, whereas "initial liquefaction" clearly indicates such a condition and 2) "cyclic mobility" covers a wide range of conditions with potential strains ranging from almost zero to many tens of percent. Thus under some situations, a condition of cyclic mobility may be perfectly acceptable, whereas in others, it would be totally unacceptable. A statement that a soil is in a condition of "initial liquefaction with a limiting strain potential of X percent" seems to provide a more specific and graphic description of the situation than a statement that the soil is cyclically mobile.

The "limiting strain potential" described above indicates that the acceptable value of the factor of safety against initial liquefaction should vary depending upon the relative density of the sand, since the development of initial liquefaction within a dense sand deposit may have as a consequence the development of only limited strains, which may be tolerable for the structures, while in the loose sand deposits, it may involve large, intolerable deformations.

The use of the available field data for known cases of liquefaction in the field supplemented by the results of the large-scale test data might well be used as a summary of past field performance concerning liquefaction and as a guide to probable future performance. Supplemented by detailed evaluations of stress conditions and liquefaction characteristics at any given site, it provides a basis for an overall evaluation of probable perfomance.

CHAPTER 8

EVALUATION OF LIQUEFACTION POTENTIAL FOR LEVEL GROUND RECOMMENDED APPROACH

8.1 INTRODUCTION

As discussed in Chapter 1, there are two basic methods for evaluating the liquefaction potential of a sand deposit. The first method involves the comparison of the field conditions at a proposed site with data concerning soil conditions at sites of known field performance in past earthquakes supplemented with the data from the large-scale tests described in this report. The second method is based on a comparison of the stress conditions likely to develop in the field and laboratory determinations of the stress conditions causing liquefaction of the sand.

For critical structures, it is believed that a final evaluation of liquefaction potential should be based upon both approaches. For less critical structures, however, a comparison of existing site conditions with data for sites known to have developed liquefaction may be adequate for practical purposes, depending on the probable margin of safety indicated by such comparisons.

8.2 BASIC INFORMATION

As a first step, the site soil conditions and the design earthquake for which the site must be analyzed should be established. The site soil conditions are more conveniently presented as a typical profile where the various soil strata are depicted against depth. The elevation of the ground water should also be indicated in the profile. For granular materials, such as sands, the Standard Penetration Resistance, as it varies with depth, should also be indicated. Additional information, such as the grain size distribution and the proportion of fines in the sand, is also helpful in the evaluation.

The design earthquake can be established either by a maximum acceleration at the ground surface or by a time history of acceleration either at the ground surface or at the base of the soil profile. In the first case, an indication of the duration of the strong ground shaking or of the magnitude of the earthquake which is causing the ground motion is also necessary.

8.3 EVALUATION OF THE LIQUEFACTION POTENTIAL

8.3.1 Initial Assessment Based on Empirical Data

The purpose of the initial assessment is to obtain an overall indication of whether the site under the prescribed ground motions is clearly liquefiable, marginal, or clearly safe. In cases where the initial assessment shows that the site is clearly liquefiable or non-liquefiable, no further analysis may be required. Marginal cases, however, should be analyzed in more detail.

The initial liquefaction assessment is carried out with the aid of the information presented in Fig. 6-1. It was discussed in the text that the data in this figure might well be used as a summary of past field performance concerning liquefaction and as a guide to probable future performance. Supplemented by detailed evaluations of stress conditions and liquefaction characteristics of the given site, it provides a basis for an overall evaluation of probable performance.

To use the information presented in the figure, the values of the Standard Penetration Resistance should be corrected to an effective overburden pressure of one ton per square foot by means of the expression presented in Chapter 1; that is:

$$N_1 = C_N \cdot N$$

where $C_N = 1 - 1.25 \log \frac{\sigma'_o}{\sigma_1'}$

N_1 = corrected penetration resistance

N = Standard Penetration Resistance at depth under consideration

σ'_o = effective overburden pressure in tons per square foot where the penetration resistance has the value N

and σ_1' = one ton per square foot

For a representative value of N_1, the cyclic stress ratio causing liquefaction under field conditions can be determined from Fig. 6-1.

The cyclic stress ratio developed by the design earthquake can be determined by the relationship:

$$\frac{\tau}{\sigma'_o} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma'_o} \cdot r_d$$

where a_{max} = maximum acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ'_o = effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of one at the ground surface to a value of 0.9 at depth of 30 feet and a value of 0.75 at 50 feet.

Thus for any given value of maximum ground surface acceleration, the possibility of liquefaction can readily be obtained on an empirical basis by comparing the developed value of τ/σ'_o with the value shown by Fig. 6-1 likely to lead to a condition of liquefaction. Furthermore, the limiting shear strains indicated by the test results for Monterey sand may be used as a guide to judge the amplitude of the shear strains that may be expected at the given site.

8.3.2 Analytical Evaluation of Liquefaction Potential

The general analytical method for evaluating liquefaction potential involves the following steps:

a. An evaluation of the cyclic stresses induced at different levels in the deposit by the earthquake shaking.

b. A laboratory investigation to determine the cyclic stresses which, for given confining pressures representative of specific depths in the deposit, will cause the soil to liquefy or undergo various degrees of cyclic strain. As shown in Fig. 1-6, the evaluation of liquefaction potential is then based on a comparison of the cyclic stresses induced in the field with the stresses required to cause liquefaction or an acceptable limit of cyclic strain in representative samples in the laboratory.

The cyclic stresses induced in the ground by an earthquake may be computed by a ground response analysis (Seed and Idriss, 1967) by a simplified procedure based upon a knowledge of the maximum ground surface acceleration (Seed and Idriss, 1971) or by deconvolution of a known ground surface motion (Schnabel, Lysmer, and Seed, 1972; Roesset and Whitman, 1969). The computed irregular time history of stresses at any given depth is then converted to an equivalent uniform cyclic stress series by an appropriate weighing procedure (Seed, Idriss, Makdisi, and Banerjee, 1975) for use in the analysis.

Various types of laboratory test procedures may be used to investigate the cyclic stress conditions required to cause initial liquefaction and the associated limiting strain potential of saturated sands. Since the object of the test is to reproduce the stresses acting on an element of sand subjected to horizontal shear stresses which reverse direction many times during an earthquake, some form of simple shear test provides the best representation of field conditions. However, since the equipment for conducting any type of simple shear tests is somewhat complicated and not readily available in most laboratories, the cyclic loading triaxial test as developed by Seed and Lee (1966) may be used as a practical and convenient alternative.

If cyclic simple shear tests are used, the laboratory test results should be corrected to account for a) the effects of multi-directional shaking and b) the overconsolidation ratio of the natural sand deposit. Appropriate correction factors are shown in Fig. 3-15.

If cyclic triaxial tests are used, appropriate correction factors have been presented in Fig. 3-14. Those factors account for both the effects of multi-directional shaking and the overconsolidation ratio effect.

Whatever type of test is used, it is considered that the tests should be performed on good quality undisturbed samples which retain the density and structure of the in-situ deposit. However, care is required to ensure that variations in these characteristics are not induced by the sampling and handling process. Where it can be shown that changes in these characteristics have occurred, appropriate corrections to the laboratory test data should be applied on the basis of the known effects of the different factors involved.

Finally, by comparing the shear stresses induced by the earthquake with those required to cause liquefaction, it may be determined whether any zone exists within the deposit where liquefaction is likely to occur (induced stresses exceed those causing initial liquefaction) and from the results shown in Fig. 4-1 and 4-2, the probable extent of cyclic shear strains that may develop can be estimated.

8.4 EVALUATION OF EFFECTS OF LIQUEFACTION

In some cases it will also be desirable to evaluate the effects of liquefaction by computing the rate of increase of pore water pressure in different layers of a deposit during earthquake shaking and the subsequent rate of dissipation of pore pressures following the earthquake. This will be particularly true in soil profiles involving highly pervious materials or very deep ground water tables. For such cases an analysis such as that presented by Seed, Martin and Lysmer (1975) may well provide a deeper insight into the effects of possible pore pressure redistribution and the effects of pore pressure dissipation on the stability of the soil deposits involved.

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