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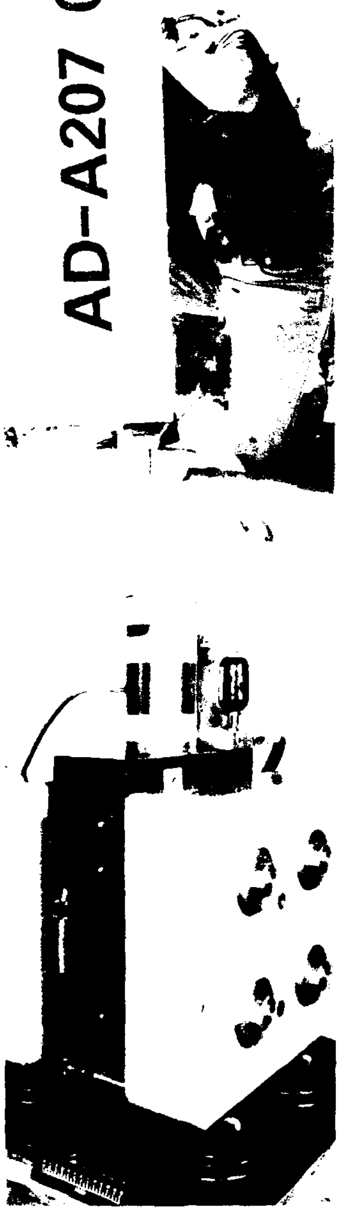

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EARTHQUAKE-INDUCED LIQUEFACTION OF FINE-GRAINED SOILS—CONSIDERATIONS FROM JAPANESE RESEARCH

by

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Liquefaction potential of various types of soils has received a great deal of research attention in the geotechnical community over the previous two decades. Dramatic occurrences of liquefaction in saturated deposits of fine, uniformly graded sands in the Japanese city of Niigata and surrounding areas resulting from ground shaking during the 16 June 1964 earthquake spawned extensive studies to develop methodologies for assessing the potential for liquefaction of predominantly clean sands throughout Japan and worldwide. The Tangshan, People's Republic of China (PRC), earthquake of 28 July 1976 caused widespread property damage, some of which has been attributed to liquefaction of soils containing appreciable quantities of silts and sometimes plastic clays. Japanese researchers have recently performed laboratory and field studies to assess the influence of variations in grain size distribution and soil index properties on liquefaction potential of fine-grained soils.

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19. ABSTRACT (Continued).

Several Corps of Engineers dams in seismically active areas are founded on fine-grained, low plasticity alluvial deposits. This report reviews current practices applied to study the phenomenon of fine-grained soil liquefaction, with emphasis on recent Japanese laboratory and in situ testing research. The findings will promote efficiency of effort in the conduct of subsequent laboratory testing efforts toward the development of specific procedures for use by the Corps and others in assessing the potential for earthquake-induced liquefaction to occur in fine-grained soils.

PREFACE

The preparation of this report was sponsored by the Headquarters, US Army Corps of Engineers (USACE), US Army Civil Works Research and Development (CWRD) Soils Program, specifically CWRD Work Unit 32255 entitled "Liquefaction Potential of Fine-Grained Soils". The USACE Technical Monitor on this project was Mr. Richard F. Davidson. The report was prepared by Messrs. Joseph P. Koester, Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES), and Takashi Tsuchida, Port and Harbor Research Institute (PHRI), Ministry of Transport, Japan, under the general direction of Dr. William F. Marcuson III, Chief, GL, and Dr. Arley G. Franklin, Chief, EEGD, GL. Mr. Tsuchida visited WES during FY 1986 on a one-year temporary assignment sponsored jointly by the Japanese Ministry of Transport and WES.

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COL Dwayne G. Lee is the present Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

EARTHQUAKE-INDUCED LIQUEFACTION OF FINE-GRAINED SOILS--CONSIDERATIONS FROM JAPANESE RESEARCH

PART I: INTRODUCTION

Background

1. Liquefaction potential of various types of soils has received a great deal of research attention in the geotechnical community over the previous two decades. Japan has experienced more strong earthquakes during the past century than perhaps any other area in the world, and has thus been afforded many opportunities to evaluate the response of soil deposits to strong shaking, with particular attention given to liquefaction. Iwasaki (1985) extended a study by Kuribayashi, Tatsuoka, and Yoshida (1974) of earthquakes causing liquefaction in Japan to 50 such events, beginning with the earthquake at Hamada in 1872, and ending with the Nihonkai Chubu (Japan Sea) earthquake of 1983. Dramatic occurrences of liquefaction in saturated deposits of fine, uniformly graded sands in the Japanese city of Niigata and surrounding areas resulting from ground shaking during the June 16, 1964 earthquake (Japan National Committee on Earthquake Engineering, 1965) spawned extensive studies to develop methodologies for assessing the potential for liquefaction of similar soils throughout Japan and worldwide.

2. Tsuchida (1970) investigated gradation curves associated with soils for which liquefaction behavior was observed during several previous Japanese earthquakes and those exhibiting similar response in laboratory shake table tests. He proposed the results shown in Figure 1 as an index to distinguish liquefiable and non-liquefiable deposits. According to Figure 1, fine-grained soils (silts and finer) whose mean grain size (i.e., 50 percent of the sample weight is finer) is at least 0.02 millimetres are regarded as potentially vulnerable to liquefaction under some unspecified level of shaking; indeed, this graph indicates liquefaction potential in soils which are entirely composed of silt-and-finer particles (that is, particles smaller than 0.074 millimetres, which would pass through the openings in a US Standard No. 200 sieve). Ishihara, et al. (1980) claim that the Tsuchida (1970) chart is based only on performance of soils of

alluvial, diluvial, or volcanic origin, and the boundaries of "most liquefiable soils" are unconservative with regard to soils containing a high fraction of low plasticity clay-size particles such as are present in mine tailings. Tokimatsu and Yoshimi (1983) compiled field performance data from Japanese earthquakes that correlate observed ground behavior with gradation characteristics of local soils; soils containing up to 60 percent by weight silt-size particles and 12 percent clay-size particles (that is, particles smaller than 0.005 millimetres) exhibited moderate-to-extensive liquefaction (in terms of affected land area). Their compilation did not catalog soil index properties, for example, Atterberg limits, which have been shown to influence cyclic strength. The influence of soil index properties on liquefaction resistance will be discussed later in this report.

3. The Tangshan, People's Republic of China (PRC) earthquake of July 28, 1976 caused widespread property damage, some of which has been attributed to liquefaction of soils that plot outside the "boundaries for most liquefiable" soils in Figure 1 (Chang, 1987). There are few data available on case histories or comprehensive evaluations of the liquefaction susceptibility of materials of such gradations outside a limited number of published studies of the Tangshan event. Wang (1979) first reported geotechnical aspects of the Tangshan earthquake, some of which indicated a potential for certain types of clayey soils to undergo drastic strength loss when subjected to earthquake shaking. Statistically, such soils contained less than 15 percent clay-size particles, had a liquid limit less than 35, and had an in situ water content greater than 90 percent of their liquid limit. The Modified Chinese Building Code (Jennings, 1980) employs the following criteria, which are not entirely consistent with Wang's observations, as conditions for liquefaction occurrence:

Parameter	Threshold
Mean Grain Size, D_{50} (mm)	$0.02 < D_{50} < 1.0$
Clay Particle Content (%)	< 10
Uniformity Coefficient, C_u	< 10
Void Ratio	> 0.8
Plasticity Index, I_p	< 10

These values represent a compilation of field observations for which ground motions are not specified (or likely, measured), thus the level of shaking required to initiate liquefaction was not considered.

4. Most comprehensive liquefaction studies published in the literature to date have emphasized clean sands. A Civil Works Research and Development (CWRD) study was initiated in FY 1984 at the US Army Engineer Waterways Experiment Station (WES) to evaluate the hazard posed by fine-grained alluvial soils in the foundations of Corps of Engineers dams and, if necessary, to develop a methodology to assess the potential for earthquake-induced liquefaction of such soils. The first step in the CWRD study approach was to collect information on field occurrences of liquefaction in sandy soils containing more than about 5 percent silt-and-finer particles. Chang (1987) reports the findings of a preliminary examination of the data base maintained among various PRC geotechnical earthquake engineering research institutions of ground response characteristics observed during several strong earthquakes.

5. Japanese researchers have recently performed laboratory and field studies to assess the influence of variations in grain size distribution and soil index properties on liquefaction potential of fine-grained soils. The geomorphology of Japan has produced characteristically uniform sand deposits that generally contain less silt and clay than those found in the peneplain and coastal regions of the PRC (Chang, 1987). A careful study of deposit stratigraphy and soil properties observed at liquefied and non-liquefied sites within the wide geographic region affected by the 1983 Nihonkai-Chubu (Japan Sea) earthquake supports this contention (Tohno and Shamato, 1986). Man-made fine-grained deposits, however, such as mine tailings and some compacted sand fills, often contain high fine grain size fractions and are important to the industry and economy of Japan. Their cyclic strength has been studied to develop seismic design criteria (Ishihara, et al. 1980, and Tatsuoka, Ochi, and Fujii, 1984).

Purpose and Scope

6. This report reviews current practices applied to evaluate the potential for fine-grained soil liquefaction, with emphasis on recent Japanese research. Recent Japanese laboratory testing efforts yielded useful information regarding sample preparation and cyclic loading techniques, and this review is intended to promote efficiency of effort in a subsequent reconnaissance laboratory cyclic testing program to investigate the liquefaction susceptibility of fine-grained soils on the bases of plasticity, grain-size distribution, and density. Japanese in situ testing experience is also briefly reviewed in this report to provide a background for development of in situ test correlations. Soil "fines" are defined as those soil materials passing a US Standard No. 200 sieve (silts, or clays, or mixtures of both) for the purposes of this report. Static testing methods applied to liquefaction potential evaluation were not considered, and the review is confined to completely saturated soils. The term "fine-grained" soil herein describes mixtures of soil materials that contain appreciable fines, yet do not classify as a clay (CL or CH) in the Unified Soil Classification System.

PART II: LABORATORY TESTING EXPERIENCE

Laboratory Tests Using Undisturbed Specimens

7. The following is a list of several of the factors most often studied t assess their respective or collective influence on cyclic strength of level ground subjected to earthquake shaking:

- a. Water content or degree of saturation
- b. Density
- c. Atterberg limits (liquid limit and plasticity index)
- d. Gradation (mean grain size and uniformity coefficient)
- e. Silt- and clay-size content (percent by weight finer than 0.074 mm)
- f. Aging and inner structure
- g. Stress history (static overconsolidation, preshearing during prior earthquakes)
- h. Coefficient of lateral earth pressure at rest

Laboratory tests on "undisturbed" samples can, to some degree, account for all of the above factors as regards their influence on in-situ cyclic undrained strength. Reconstitution or artificial mixture specimen construction removes the effects, beneficial or otherwise, of factors f-h, and it could also be argued that natural soil water imparts yet other effects that cannot be reproduced in the laboratory. Table 1 (Mulilis, Chan, and Seed, 1975) documents a few studies where dynamic strengths of undisturbed specimens (here defined as stress ratios required to cause 5 percent double amplitude (peak-to-peak) cyclic axial strain or 100 percent pore pressure response in 10 loading cycles in undrained cyclic triaxial tests) were compared to strengths of specimens of the same soil remolded to the same pretest conditions. Eight of the nine cases shown in the table indicated that undisturbed specimens were measurably stronger, and soils containing larger percentages of fines were increasingly affected by remolding disturbance.

8. Seed, Idriss, and Arango (1983) suggested that testing is the best way to assess the cyclic loading response of clayey soils with the characteristics observed by Wang (1979) (discussed in the Background section of the report) and that plot above the A-line on the Casagrande plasticity chart, though they did not specify a particular type of test. Few laboratory cyclic testing efforts involving undisturbed samples of fine-grained low-plasticity soils are reported in the literature. This is probably at least partially a result of difficulties typically encountered in obtaining high-quality undisturbed samples of such materials. Ishihara, et al. (1980) noted that finely-layered undisturbed samples of silt obtained from tailings slimes deposits were generally very soft and had water contents well above their liquid limit so that samples would collapse under self weight.

9. Tatsuoka, et al. (1978) compared cyclic triaxial strengths of undisturbed fine-grained mixture soils to those measured for reconstituted specimens of the same soils prepared to the same initial relative densities. Figure 2 shows that 35 to 100 percent greater cyclic strengths (defined by Tatsuoka, et al., 1978 as the cyclic stress ratios causing 5 percent double amplitude cyclic axial strain in 20 cycles) were measured using undisturbed specimens of soils containing between 10 and 40 percent fines than when using their reconstituted counterparts. No index properties were reported in the referenced study, nor was any mention made of potential effects of varying index properties on cyclic strength. The authors conceded that density variations may have resulted from the sampling procedures they employed to obtain their more silty specimens. They did not wish to risk frost heave effects that might have resulted from in situ freezing techniques on these materials, such as were successfully used by Yoshimi, Hatanaka, and Oh-oka (1977) and Hatanaka (1977) to avoid sample handling disturbance in clean sands. A similar comparison was made using clean Niigata sands, for which Ishihara (1985) measured 20 percent greater cyclic strengths in undisturbed specimens. He attributed the larger undisturbed strengths to inner structure and aging effects, both of which are lost on remolding. These two Japanese studies indicated generally greater strength differences between disturbed and undisturbed fine-grained soils than between clean sands similarly tested.

10. Matsumoto, Sasaki, and Kondoh (1988) and Sasaki, Matsumoto, and Kondoh (1988) report a program of 102 cyclic triaxial tests on undisturbed samples of soils ranging from coarse sands to sandy silts, with emphasis on fines content (maximum of 41 percent by weight of particles finer than 0.074 mm; no Atterberg limits data reported). The samples were obtained from shallow borings at seven sites in an effort to refine predictive equations for liquefaction strength (defined as the cyclic stress ratio required to cause 5 percent double amplitude axial strain in 20 loading cycles). Previous predictive equations, based on Standard Penetration Test blowcounts (SPT N-value), had been proposed to account for gradation characteristics, for example D_{50}, but had not considered the effects of amount of particles finer than 0.074 millimetres (Tatsuoka, et al., 1978). Predictive equations will be discussed in a later section on in situ testing research.

Laboratory Tests Using Reconstituted Specimens

11. The degree to which various gradation factors and constitutive properties of fines affect laboratory cyclic behavior is controllable, to a large extent, through use of reconstituted replications of in situ deposits. Soil specimens so prepared serve as "benchmarks" to assist researchers in characterizing the response of natural deposits to particular perturbations.

12. Lee and Fitton (1969) performed cyclic triaxial tests on several uniform clean sand specimens (artificially prepared from fractions of pit run parent soil to gradations with a coefficient of uniformity, C_u, of about 1.5), as well as on a silt and a silty clay. The cyclic stress ratio (in this case, the ratio of pulsating vertical deviator stress to effective confining stress) attributed to cause 5 percent double amplitude axial strain in 30 cycles of vertical deviator stress is plotted in Figure 3 to illustrate relative cyclic strength among soils of varying mean grain size for two pre-loading "relative densities", D_r. Lee and Fitton (1969) cautioned that the relative density values corresponding to the silt (mean grain size about 0.02 mm, or 2×10^{-5} m) and silty clay (mean grain size 0.005 mm, or 5×10^{-6} m) strengths were determined by extrapolation, since no means was available by which to measure the minimum density of such soils. The authors used maximum modified Proctor density to represent

maximum density for the silt (26 percent liquid limit, plasticity index equal to 3) and the silty clay (38 percent liquid limit, plasticity index equal to 17) specimens, and extrapolated the minimum density values for both materials from that measured for a nonplastic, silty, hydraulic fill sand (cyclic strength not given). The silt exhibited about the same resistance to pulsating load as did the fine sands tested (mean grain size as large as 0.3 mm, or 3×10^{-4} m). The silty clay showed a significantly higher strength against the specified cyclic strain development than did the sands or the silts prepared to a similar density condition. The silty clay specimen prepared to a 75 percent "relative density" condition in the manner just described exhibited a much greater increase in cyclic strength over its looser counterpart than did the coarser soils. Examples of the difficulties experienced by other researchers in the measurement and use of relative density in fine-grained soils will be discussed in later paragraphs.

13. Kaufman (1981) and Sherif, Tien, and Pan (1983) examined the effects of addition of a low plasticity silt (different silt used in each study, but with similar gradation and index properties) in varying quantities to a clean, uniform sand on the cyclic resistance of the mixtures, as determined by cyclic triaxial shear and cyclic torsional simple shear tests, respectively. Figure 4 is replotted from the data of each study and illustrates the mutual finding that cyclic resistance was not significantly altered until the admixture of silt comprised upwards of 30 percent of the total sample weight. Cyclic strength values were obtained in both studies by normalizing cyclic deviator stress values to the mean effective confining stress to which each sample was consolidated prior to cyclic loading. Pure silt specimens exhibited about 45 percent higher cyclic strengths than did specimens with 30 percent or less silt in the Kaufman (1981) cyclic triaxial tests and about 65 percent higher strengths in cyclic torsional shear tests by Sherif, Tien, and Pan (1983). A slightly higher cyclic strength was observed in clean sand specimens over that measured for specimens containing 10 and 20 percent silt in both studies; Kaufman (1981) applied membrane compliance corrections proposed by Martin, Finn, and Seed (1978) that later reduced his clean sand strengths. Membrane compliance was not accounted for by Sherif, Tien, and Pan (1983), who chose to suggest that the higher cyclic strength in clean sands was accurate, but that silt might act as a lubricant to reduce frictional resistance to shear at contents below 35 percent. They proposed that the increase in cyclic

strength above 35 percent silt content might result from "wedging effects" whereby the now predominantly silt matrix retards deformation. Cyclic triaxial shear strengths measured by Kaufman (1981) differed markedly from cyclic torsional simple shear strengths measured by Sherif, Tien, and Pan (1983), although test conditions and soil types were very similar. One goal of the CWRD project is to evaluate the relationship between cyclic torsional shear strength and triaxial strength; at least one study has shown that such a relationship may not be either consistent or readily predictable. Later paragraphs will treat this issue with more detail.

14. Ishihara (1976) investigated the effects of silt content and overconsolidation on the cyclic triaxial strength of reconstituted specimens of several sands. Figure 5 shows the cyclic stress ratios causing development of 5 percent double amplitude strain in 30 cycles of load pulsation for various consolidation conditions and silt contents. Void ratio was maintained relatively constant. The addition of a given silt exerted little influence on the cyclic strength of normally consolidated sand-silt mixtures, yet increasing silt content exerted an increased effect on cyclic strength with increased overconsolidation. No details were available from the published results of this study as to whether the void ratios reported were obtained prior to consolidation or following consolidation, or whether high fines contents inhibited the achievement or maintenance of desired void ratio.

15. Ishihara, et al. (1980) investigated the effects of index properties of the fines constituents of tailings slimes on cyclic strength characteristics. Figures 6(a), 6(b), and 6(c) show cyclic strength (defined here as the cyclic stress ratio causing 5 percent double amplitude strain in 20 cyclic triaxial test load cycles) as a function of void ratio for tailings sands, low-plasticity tailings slimes (tailings composed primarily of silt-and-finer particles, usually hydraulically deposited in settlement ponds), and high-plasticity tailings slimes (having a plasticity index (termed either PI or I_p) as high as 28, unusually high for such inorganic, artificially created soils), respectively. This study illustrates the importance of the consistency characteristics of the fines themselves to cyclic loading response in a comprehensive evaluation of fine-grained soil liquefaction potential. Gradation alone is insufficient information on which to base a decision as to the liquefaction susceptibility of a given soil deposit. Ishihara (1985) offers that soils deriving their internal

resistance to shear deformation from friction between particles while under confining pressure are most susceptible to liquefaction. He observed that tailings deposits contain fine-grained particles that are virtually free from surface adhesion, thus separation of these particles under decreased confining pressure is uninhibited, and their liquefaction potential is usually high.

16. Relative density has been used successfully to regulate specimen preparation in most studies of cyclic response in granular soils. The determination of relative density is affected by fines content. Kaufman (1981) found it difficult to obtain post-consolidation void ratios required for a given relative density in his more silty specimens, and attributed this to migration of fines within the sand matrix during the consolidation phase of sample preparation. Increased fines content may be expected to impart increased consolidation effects. Relative density is generally recognized as an appropriate index parameter for delineating cyclic or static mechanical behavior of clean sands, as the cyclic loading response of these soils has been demonstrated to be a convincingly strong function of relative density. Clean sand specimens are more easily reproduced in this regard than are fine-grained soils. Mechanical properties of clays, on the other hand, have been correlated to such parameters as water content, plasticity index (Bjerrum, 1973), and overconsolidation ratio (Ladd and Foott, 1974). Fine-grained soils, as defined in the context of this report, fall somewhere in between, and no definitive index has yet been conclusively established to govern specimen preparation and replication.

17. Several researchers have attempted to predict fine-grained soil dynamic response by extending the proven relation of relative density to cyclic strength in clean sands, in spite of the lack of an accepted procedure to measure relative density in such materials. Lee and Fitton (1969), as discussed earlier, experienced particular difficulty in determining minimum densities in soil mixtures with significant fines content. Tatsuoka, et al. (1978) reported that no standard method for measuring maximum and minimum void ratio in such materials had been established in Japan as of that time. Ishihara, et al. (1980) abandoned efforts to measure minimum and maximum void ratios in their study of tailings materials, and opted to relate cyclic strength to post-consolidation void ratio. Figures 6(a) and 6(b) support their observation that the cyclic strength of low-plasticity slimes they tested

was generally lower than that of tailings sands at void ratios below about 0.85, but the strengths were nearly identical at higher void ratios. As fines content increases, consolidation effects, and perhaps the index properties of the fines themselves, impart increasing uncertainty as to the correct minimum and maximum void ratio values to use. Figure 6(c) is included to demonstrate the cyclic strength increase observed by Ishihara, et al. (1980) in tailings slimes containing "high plasticity" (I_p as high as 28 percent) fines. Tatsuoka, Ochi, and Fujii (1984) tested the effects of various methods of sample preparation on the cyclic strength of Toyoura sand and Sengenyama sand. They concluded that relative density was not a good index for comparing density conditions between different kinds of sands, in that samples prepared to obtain maximum and minimum void ratio values were prepared differently than were samples prepared for cyclic strength testing, and cyclic strength was significantly dependent on the method used to prepare samples. Their sample preparation procedures will be detailed later. The Sengenyama sand tested in their study contained only 2.4 percent fines (no Atterberg limits data available), and the Toyoura sand was clean.

18. Sherif, Tien, and Pan (1983) examined the sensitivity of relative density to silt content from the standpoint of the influence of various amounts of low-plasticity silt (from Silver Lake, Washington, liquid limit: 26 percent; plastic limit: 22 percent) on particle-to-particle contacts in the sand matrix. Various mixtures of Ottawa parent sand and the silt were poured, dry, into a special torsional simple shear specimen mold and vibrated until a maximum density was achieved (or a fraction thereof, depending on the desired relative density). The authors did not define the procedure by which they established minimum density, nor did they report any difficulties in so doing. Figure 7 illustrates the relationship they observed between silt content and extreme void ratios determined in the manner just described. Maximum void ratio (minimum density) was shown to be constant for silt contents up to 35 percent. The authors interpreted this to represent the silt content just sufficient to fill the voids between sand grains in soil at its loosest possible state. The finer particles apparently exert increasing influence on packing and compaction characteristics at silt contents beyond 35 percent, which in turn affects relative density determination. Minimum void ratio (maximum density) varied

with the addition of any amount of silt to the parent sand, reached a low value at a silt content of about 50 percent, and increased with higher silt contents.

19. Kondoh, Sasaki, and Matsumoto (1987) performed a brief series of laboratory cyclic triaxial tests to assess the effects of a given type of fines (kaolin) on liquefaction strength. Sufficient amounts of kaolin clay were added to Sengenyama sand to prepare five specimens each with 10 and 20 percent fines (by weight) and five "clean" sand specimens. Tatsuoka, Ochi, and Fujii (1984) indicated that Sengenyama sand already contains 2.4 percent silt fines, and it is herein assumed that Kondoh, Sasaki, and Matsumoto (1987) either sieved their parent "clean" sand to remove material finer than 0.074 millimetres or disregarded the inherent fines. Figure 8 shows the liquefaction strengths (R_1, defined as the cyclic stress ratio required to cause 5 percent double-amplitude axial strain) measured as a function of number of load cycles for kaolin contents of 0, 10, and 30 percent. The difference in cyclic strength realized from the addition of kaolin clay is small, yet the trend is consistent. Figure 9 shows the increased cyclic stress ratio required to produce 5 percent double-amplitude axial strain in 20 loading cycles, replotted to emphasize the data of Figure 8. Kondoh, Sasaki, and Matsumoto (1987) reported that cyclic strengths varied in the same manner as the post-consolidation specimen densities, that is, higher density samples produced higher strengths, thus at least some of the strength increase they observed with the addition of kaolin must also be attributed to density effects. Kondoh, Sasaki, and Matsumoto (1987) also reported cyclic triaxial strengths measured for 30 percent kaolin samples at different overconsolidation ratios, or OCR's (Figure 10). They determined that specimens preconsolidated to an OCR of 2 would require 50 percent greater cyclic stress ratios than would normally consolidated specimens to cause 5 percent double-amplitude cyclic axial strain in 30 cycles. Curves generated by Ishihara (1976) (Figure 5) indicate that the same level of overconsolidation (OCR equal to 2) could impart a 25 percent increase in resistance to development of 5 percent double-amplitude cyclic axial strain in 30 cycles in specimens containing 30 percent silt. Clearly, the effects of fines type and associated index properties exert a more complex influence on cyclic strength than may be expressed as a simple function of percent finer than 0.074 millimetres.

20. Specimen preparation procedures have been demonstrated to have profound effects on liquefaction resistance of reconstituted specimens of sandy soils. Mulilis, Chan, and Seed (1975) report a study of 11 different compaction processes and their relative effects on cyclic triaxial strength of laboratory test sands, from which they concluded that dynamic strength is affected by method of preparation to different degrees depending on soil type. They observed that moist tamping or moist vibratory compaction techniques generally produce higher remolded cyclic triaxial strengths in specimens of similar pre-consolidation dry density than do other typical methods, thus such specimens would best replicate undisturbed behavior.

21. Ishihara, et al. (1980) used four different specimen preparation procedures to vary specimen density in their study of the cyclic triaxial strength of ground quartz powder and mine tailings materials sampled from tailings dams in Chile and Japan. Coarse grained (sands) and fine grained (slimes) materials were sampled separately from laminated deposits at each of the tailings dam sites. Loose sand or slime specimens were prepared by pluviation through air with no additional external disturbance. "Medium dense" sand specimens were formed by a combination of pluviation through air and tapping with rubber hammer on the specimen mold. Dense tailings sand specimens were prepared by pluviating sand through air and applying 50 Hz vibrations to the side of the specimen mold. These three procedures were performed using dry tailings sands or slimes, whereupon carbon dioxide gas was flushed through the specimen prior to backpressure saturation. Dense slime specimens were mixed with water to have a water content about equal to the liquid limit (20 to 40 percent) and spooned into the specimen mold in two equal lifts that were each vibrated for five minutes as above. The slurry was consolidated under a surcharge pressure for up to three days and frozen to prevent handling disturbance during test setup. The frozen slurry specimens were thawed and flushed with carbon dioxide gas prior to backpressure saturation. The findings of the authors regarding the effects of fines content and type on relative to dynamic strength were discussed earlier. Only the effects of specimen density (corresponding to post-consolidation void ratio) on cyclic triaxial strength were evaluated in this study; the contributions to strength of the various preparation methods themselves revealed by Mulilis, Chan, and Seed (1975) were not considered.

22. Most researchers believe that cyclic simple shear tests, in general, replicate stresses induced in soil deposits by earthquake shaking better than do cyclic triaxial tests. Cyclic simple shear test equipment was not readily available in early liquefaction studies, and an alternate procedure was developed whereby cyclic triaxial tests, with proper interpretation, could be used to assess the various factors affecting cyclic behavior and pore pressure response in laboratory soil specimens (Seed and Lee, 1966, and Seed, 1979). Seed (1979) describes the experimental determination of a correction factor, C_r, defined as the ratio of cyclic triaxial strength to cyclic simple shear strength. Here, strength is defined as the cyclic stress ratio (that is, the ratio of maximum shear stress to effective ambient pressure in cyclic triaxial tests, and the ratio of shear stress on a horizontal plane to effective overburden pressure in cyclic simple shear tests) required to produce 100 percent pore pressure response. He detailed the evolution of laboratory cyclic simple shear devices and reported that a C_r value of about 0.63 was determined from comparison of several large- and small-scale laboratory cyclic simple shear tests on normally consolidated Monterey No. 0 sand specimens (K_o equal to about 0.4, that is, horizontal confining stress equal to 0.4 times the vertical, or overburden stress). Seed (1979) proposed that the C_r values be multiplied by 0.9 to account for the 10 percent reduction in resistance to development of 100 percent pore pressure response attributed to multi-directional earthquake shaking in earlier studies (Pyke, Chan, and Seed, 1974, and Seed, Pyke, and Martin, 1975), thus the complete correction factor was computed to be about 0.57. Test results were compared as a function of relative density, thus one could expect a different relationship for fine-grained soils as treated in this report, since relative density has been shown to be an unreliable index in such materials.

23. The difference in major and minor principal strains is equal to the maximum shear strain in an undrained triaxial test, which is equivalent to 1.5 times the axial strain (assuming isotropic confinement prior to axial loading). Silver, et al. (1980) avoided the use of 100 percent pore pressure response as a criterion, and instead compared the number of loading cycles for a specified double amplitude axial strain in a cyclic triaxial test with the number of loading cycles for 1.5 times the double amplitude shear strain in a cyclic simple shear test, for example, 10 percent double amplitude (axial) strain in cyclic triaxial tests was assumed equivalent to

15 percent double amplitude shear strain in cyclic simple shear tests. Cyclic simple shear tests were performed by applying a cyclic horizontal load to the top of a solid cylindrical specimen confined in an unreinforced membrane and consolidated to have a K_o equal to 0.4 prior to shear loading. Tatsuoka, Ochi, and Fujii (1984) used the same strain equivalence procedure to study the effects of sample preparation method on liquefaction resistance as determined by both cyclic triaxial and cyclic torsional simple shear tests, the latter performed using hollow cylindrical specimens consolidated to have a K_o of about 0.5. They varied specimen density by using air pluviation (AP), wet tamping (WT), wet vibration (WV), and water pluviation with vibration (WAV) to form test specimens of Sengenyama and Toyoura sands. The characteristics of these two sands were described earlier. The authors opted to pluviate Sengenyama sand through air rather than water prior to saturation and vibration using the WAV method in order to prevent segregation of coarse and fine particles. Test results agreed with the finding of Mulilis, Chan, and Seed (1975) that wet (or moist) tamped specimens produced the highest cyclic triaxial test strengths, but observed that this was not necessarily the case with cyclic torsional shear tests. Specimens prepared by the WV and WT methods always exhibited similar cyclic triaxial strengths that were the highest measured, but cyclic torsional shear strengths were sometimes significantly higher for specimens prepared by the WV method.

24. The types of laboratory cyclic strength tests mentioned thus far do not account for soil anisotropy; loading is uni-directional in either the cyclic triaxial or cyclic simple shear tests. Continuous principal stress rotation tests have been conducted to evaluate the effects of multi-directional cyclic loading such as is possible in the case of earthquake ground motions (Donaghe and Gilbert, 1983) or undulating stress fields as occur in the sea floor during the passage of sea waves (Ishihara and Towhata, 1983). Test results appear to create a new lower bound for cyclic strength in laboratory specimens prepared in the same manner as for conventional cyclic tests. Hollow cylindrical specimens are subjected to simultaneous cyclic variations of axial and torsional load, with a controlled phase difference maintained between the two. Donaghe and Gilbert (1983) demonstrated that cyclic strengths so obtained were lower than those obtained in other types of cyclic loading tests, and suggested that this was attributable to all planes within the principal stress

rotation specimen having been subjected to the maximum stress at one time or another. No test data are available to date on the effects of principal stress rotation on cyclic strength of fine-grained soils; whether any layering ~r inner structure features are peculiar to such soils to somehow make them more or less susceptible to liquefaction under this type of loading is not known.

PART III: IN SITU TESTING EXPERIENCE

25. Numerous empirical correlations have been proposed between in situ test results and laboratory cyclic test results or performance during past earthquakes to evaluate in situ liquefaction potential (Seed and Idriss, 1971, Tatsuoka, et al., 1978, Tatsuoka, et al., 1980, Tokimatsu and Yoshimi, 1981, Tokimatsu and Yoshimi, 1983, Seed, Idriss, and Arango, 1983, Seed, et al. 1984, Seed, et al. 1985, and Matsumoto, Sasaki, and Kondoh, 1988). The Standard Penetration Test (SPT) has been used most extensively to assess the in situ integrity of sandy deposits, largely due to the experience gained at liquefied and nonliquefied sites at Niigata, Japan. Ishihara (1985) summarizes the evolution of SPT correlations in Japan through 1985, derived from both laboratory studies such as described earlier and observations of field performance at sites where SPT data is available. Farrar (1988) reports a study of SPT, cone penetrometer, and dilatometer correlations at six Japanese sites where liquefaction has occurred. Fine-grained soils have received special attention, due to the fact that they usually exhibit lower penetration resistance. Laboratory tests and field observations confirm that liquefaction resistance is generally enhanced by the presence of fines, particularly clayey fines, thus correlations that are derived from clean sand tests and observations are not necessarily appropriate. Further, it has been shown that formulae which attribute added cyclic strength to a given deposit based only on fines content may overpredict strengths of tailings deposits where the fines constituents are nonplastic (Ishihara, 1985). All of the studies reviewed for this report have recommended further investigation of the effects of silt and clay-size content and properties on in situ test correlations.

26. Tokimatsu and Yoshimi (1983) presented a convincing argument for the continued use of SPT data to establish reliable dynamic strength correlations, in view of recent improvements toward standardization of field procedures. The authors pointed out the advantages of the SPT, namely:

a. The SPT is a true in situ test, such that it reflects stress history, soil fabric, horizontal effective stress, and is sensitive to the joint effects of relative density and overburden stress (Seed, 1979).

b. SPT data are available as obtained prior to earthquakes at sites where liquefaction has occurred, reflecting real in situ stress conditions, and a large body of data is now available for sites where liquefaction may occur in the future.

c. The SPT is essentially an undrained shear strength test.

d. Soil samples are retained for index properties determination.

e. Many SPT's can be performed at a given site for low cost, augmenting coverage.

SPT correlations are stressed here, in recognition of the above statements and the likelihood that SPT's will remain a popular in situ evaluation method as a result of recent improvements in SPT data interpretation techniques.

27. The development of an empirical correlation between undrained cyclic triaxial strength, here defined as the cyclic stress ratio required to cause 5 percent double amplitude axial strain in 20 cycles, and SPT blowcounts, serves as an illustration of the emphasis placed on fines content. Ishihara (1977) first proposed an expression of the following form, based on a program of cyclic triaxial tests on undisturbed sand specimens:

$$R_{liq} = 0.0676 \sqrt{N_1} \qquad (1)$$

where: R_{liq} is the undrained cyclic triaxial strength as just defined; and N_1 is a corrected SPT blowcount, whereby the actual blowcount value is normalized to an effective overburden pressure of 1 kgf/cm^2, as follows (based on the formula of Meyerhof, 1957):

$$N_1 = C_N \cdot N$$

$$\text{for } C_N = \frac{1.7}{\sigma_v' + 0.7} \qquad (2)$$

where N is the actual measured SPT blowcount; and σ_v' is the effective overburden pressure in kgf/cm^2 at the depth of the SPT measurement.

28. Tatsuoka, et al. (1978) performed a series cyclic triaxial tests on undisturbed samples of young alluvial sand deposits and uncompacted hydraulic fills to study the effects of grain size on SPT penetration resistance. The following expressions were derived from the test data:

$$R_{liq} = 0.0676\sqrt{N_1} + 0.225\log_{10}\left(\frac{0.35}{D_{50}}\right)$$

$$\text{for } 0.04 \text{ mm} \le D_{50} \le 0.6 \text{ mm} \qquad (3)$$

$$R_{liq} = 0.0676\sqrt{N_1} - 0.05$$

$$\text{for } 0.6 \text{ mm} < D_{50} \le 1.5 \text{ mm}$$

where D_{50} is mean grain size, as defined earlier. It can be seen that cyclic triaxial strength increases with decreasing mean grain size for a given blowcount by an amount described by the logarithmic term in the above equation. The determination of D_{50} requires both sieve and hydrometer grain size analyses, the latter of which may not be done routinely, thus a formulation based on fines content is desirable. Tatsuoka, et al. (1980) proposed such an expression, again based on laboratory cyclic triaxial test results:

$$R_{liq} = 0.0676\sqrt{N_1} + 0.0035C \qquad (4)$$

where C is the fines content, or the percent finer than 0.074 mm.

29. Ishihara (1979) also offered a fines content relationship, based on the results of cyclic triaxial tests on similar materials retained by an Osterberg type sampler:

$$R_{liq} = 0.009(N_1 + 13 + 6.5\log_{10}C) \qquad (5)$$

where N_1 is again defined by $N_1 = C_N \cdot N$, but where C_N is defined (after Peck, Hanson, and Thornburn, 1974) as:

$$C_N = 0.77\log_{10}(20/\sigma_v') \qquad (6)$$

where σ_v' is in tons per square foot. Equation (6) is only valid for overburden pressures greater than or equal to 0.25 tons per square foot (24 kPa). Ishihara (1985) noted that the above formulae were developed using tube-sampled sandy materials, and should be applied only to loose to medium dense clayey and silty sands in view of likely sampling disturbance effects. Matsumoto, Sasaki, and Kondoh (1988) conducted a program of 102 cyclic triaxial tests on undisturbed samples of soils ranging from coarse sands to sandy silts (maximum fines content of 96 percent), and suggested that the following term be added to Equation (3) to correct for fines contents in excess of 40 percent:

$$R_{corr} = 0.004\ C - 0.16 \tag{7}$$

30. The data of Tatsuoka, et al. (1980) were obtained using alluvial and hydraulic fill materials. Tailings materials containing low plasticity fines, as discussed earlier, may exhibit low cyclic strengths as observed in loose sands. Ishihara, Yasuda, and Yokota (1981) developed the following expression from cyclic triaxial tests on undisturbed specimens of material from 15 tailings dam sites:

$$R_{liq} = 0.0676\ \sqrt{N_1} + 0.085\ \log_{10}\left(\frac{0.50}{D_{50}}\right) \tag{8}$$

where N_1 is the SPT blowcount normalized as for Equations (1)-(4). Figure 11 compares the relationships proposed by Tatsuoka, et al. (1980) with one determined by Ishihara, Yasuda, and Yokota (1981), and shows that the strengthening effect of decreasing particle size is less pronounced in the mine tailings studied than in alluvial materials. The average relationship determined for materials sampled at the Kuroko mine site is also shown in Figure 11; Kuroko tailings are typically plastic, and apparently retain cyclic strength across a wide range of mean grain size. The same study examined the influence of the plasticity of various tailings materials containing an average of about 30 percent fines on cyclic strength, the implication of which is shown in Figure 12. Assuming that the $I_p/35$ strengthening effect (I_p is plasticity index) shown in Figure 12 holds for fines contents greater than 30 percent, Equations (4) and (5) may be modified to include the plasticity effect as follows:

$$R_{liq} = 0.0676\sqrt{N_1} + 0.0001 \cdot I_p \cdot C \tag{9}$$

and

$$R_{liq} = 0.009(N_1 + 13) + 0.00167 \cdot I_p \cdot \log_{10} C \tag{10}$$

The choice as to which expression to use depends on the selection of the respective C_N correction factor.

31. Several relationships have been developed to relate SPT blowcounts to actual field performance during earthquakes. Seed (1979) modified an earlier expression for average earthquake shear stresses by Seed and Idriss (1971) and proposed that the cyclic shear stress ratio induced by earthquake shaking in a level ground deposit is given by:

$$\frac{[\tau_h]_{av}}{\sigma_o'} = 0.65 \cdot \frac{a_{max}}{g} \cdot r_d \cdot \frac{\sigma_o}{\sigma_o'} \tag{11}$$

where $[\tau_h]_{av}$ is the average cyclic shear stress developed by shaking on a horizontal plane in the deposit; σ_o' and σ_o are the initial effective and total overburden stresses, respectively, at the depth in question; a_{max} is the maximum acceleration at the ground surface; g is the acceleration of gravity in units consistent with a_{max}; and r_d is a stress reduction factor that varies from a value of 1 at the ground surface to 0.9 at a depth of 30 ft (9 m). Charts have since been prepared to relate SPT blowcounts, corrected for the effects of overburden pressure, and field procedures, and fines contents, to earthquake shear stresses calculated from Equation (11) for over a hundred well-documented sites worldwide where liquefaction either has or has not occurred (Seed, Idriss, and Arango, 1983, Seed, et al. 1984, and Seed, et al. 1985). Figure 13 (from Koester and Franklin, 1985) is a chart used to estimate liquefaction resistance of clean sands using corrected SPT blowcounts (shown as $(N_1)_{60}$, normalized to an effective overburden pressure of 1 tsf (96 kPa), and adjusted to correspond to an SPT test having an energy equivalent to 60 percent of theoretical maximum free fall, after Seed, et al. 1984). Field performance data indicate that combinations of average earthquake shear stress ratio and $(N_1)_{60}$ plotting to the left of the appropriate magnitude curve indicate potentially liquefiable deposits, whereas deposits for which points plot to the right are considered nonliquefiable. Figure 14 is currently used to estimate liquefaction potential in silty sands. Figure 15 (Koester and Franklin, 1985) is included to summarize the empirical procedure developed by Seed, Idriss, and Arango (1983).

32. Ishihara (1985) compares empirical correlations developed on the bases of field performance and laboratory strength, under the assumption that average earthquake shear stress ratios used to estimate liquefaction potential based on field performance may be converted to the maximum shear stress ratios measured as laboratory strengths by dividing the former by 0.65. Clean sand correlations so compared are in general agreement among several independent studies. Figure 16 compares converted estimates of earthquake cyclic stress ratio required to cause liquefaction in a field performance study by Tokimatsu and Yoshimi (1983) to cyclic triaxial test stress ratios causing 5 percent double amplitude cyclic strain in laboratory studies by Tatsuoka, et al. (1978) (curve labeled Japanese Code of Bridge Design) and Ishihara (1979) as functions of fines content for overburden pressure corrected SPT blowcounts of 10 and 15. The Japanese Code of Bridge Design gives a higher rate of strength increase with increasing fines than do the other two relationships, possibly due to differences in the consistency of the fines fractions studied.

33. A seismic design code in effect in China (PRC) since 1974 incorporates the following empirical criterion to establish critical SPT blowcounts, N_{crit}, for liquefaction resistance, based on field performance (Xie, 1979):

$$N_{crit} = N[1 + 0.125(d_s - 3) - 0.05(d_w - 2)] \qquad (12)$$

where d_s is the depth to the sand layer in question, in meters; d_w is the depth to the water table, in meters; and N is a reference blowcount, determined from earthquake shaking intensity. The 1976 Tangshan earthquake provided enough data on liquefaction of clayey soils to generate several proposed modifications to the 1976 code to account for the presence of clay fines, always having the beneficial effect of reducing the critical blowcount required for soils containing clay, for example (Shi, 1984):

$$N_{crit} = N[1 + 0.125(d_s - 3) - 0.05(d_w - 2) - 0.1(p_c - 3)] \quad (13)$$

or

$$N_{crit} = N[1 + 0.125(d_s - 3) - 0.05(d_w - 2)]\left(\frac{3}{p_c}\right)^{0.5} \quad (14)$$

where p_c is the percent clay fines by weight (finer than 0.005 mm). Seed, et al. (1985) maintain that the blowcount values used to develop the above expressions were obtained with equipment producing 60 percent of the theoretical maximum energy input to the drill stem, thus no energy adjustment appears necessary to permit comparison with energy-corrected data from other sources. The authors report yet another such modification by Taiping, et al. (1984) where the bracketed factor in Equation (12) is reduced by $0.07p_c$. The PRC code and subsequent proposed changes do not account for the index properties of the clay fines, but the code as modified by 1980 (Jennings, 1980) does specify a maximum plasticity index of 10 for liquefiable soils. It is desirable to determine by what laboratory means PRC researchers determined Atterberg limits used to develop the PRC building code; Ledbetter and Krinitzsky (1982) report a study wherein two agencies used different sample drying and wetting procedures to reconstitute the same low-plasticity soil that yielded significantly different plasticity index values.

PART IV: CONCLUSIONS

34. This report reviews recent research applied to evaluating the potential for liquefaction to occur in fine-grained soils as a result of earthquake shaking. The bulk of relevant literature has been generated by Japanese and PRC researchers, due to the heightened interest generated in those countries by field occurrences of liquefaction phenomena in low plasticity soils containing appreciable amounts of material finer than 0.074 mm. The report emphasizes recent Japanese laboratory and in situ testing experience directed toward both natural and man-made deposits, the results of which justify re-examination of traditional liquefaction criteria based on laboratory and field performance of clean sands. Liquefaction criteria based on grain size alone may, in some cases, be inappropriate in view of the findings summarized in this report.

35. Development of practical procedures to evaluate liquefaction potential in fine-grained soils will probably depend on reliable correlation of their cyclic strength to one or more index parameters such as density or plasticity of fines. Relative density is a well-documented indicator to delineate between liquefiable and nonliquefiable clean sand deposits, but density extremes required to establish relative density are difficult to measure in silty or clayey soils. Maximum void ratio (minimum density) is apparently a discontinuous function of fines content; maximum void ratio remains constant in sand-silt mixtures until the silt fraction reaches the amount necessary to just fill the voids in the parent sand, but higher void ratios are attainable as silt content increases.

36. Undisturbed specimens of loose fine-grained soils are expensive and difficult to obtain; the best of sand sampling methods are generally recognized to involve freezing an oversized quantity of material and trimming to the desired size, but fine-grained soils are generally susceptible to volume expansion disturbance on freezing. No new methods were found to improve undisturbed sampling success. Laboratory cyclic tests using soil either sampled from natural deposits and conceded to be disturbed or reconstituted from controlled mixtures of sand and various fines types will still produce reliable data for the purpose of developing useful evaluation procedures. Contributions to cyclic strength attributed to

aging, prior earthquake shaking, overconsolidation, and inner structure in situ are lost on remolding, thus any liquefaction evaluation methodology that does not account for their benefits is conservative.

37. Specimen density is easily monitored throughout the consolidation phase of specimen preparation prior to cyclic loading, and exerts a consistent influence on cyclic strength for soil mixtures containing fines of a given type. The procedure used to build a reconstituted laboratory specimen strongly influences cyclic strength, all initial conditions being equal; wet tamping methods are still recognized as producing cyclic strengths best approaching in situ values. Researchers have found it difficult to accurately predict and control post-consolidation void ratios in rec nstituted fine-grained laboratory specimens built by wet tamping methods, and the difficulty is more pronounced in specimens with greater fines contents. Cyclic strength may, however, be ultimately correlatable to this parameter.

38. Percent fines content alone is not an adequate criterion to distinguish deposits susceptible to liquefaction from those that are not. The clay fraction itself contributes to cyclic strength in relation to its consistency properties. Low plasticity tailings soils with high clay contents were observed to have no greater resistance to liquefaction on cyclic loading than loose fine sands. Clay fraction determination requires hydrometer analysis and is therefore somewhat a less desirable parameter than plasticity index, which is easy to measure, relatively stable, and has been shown to correlate with cyclic strength in a manner similar to clay content.

39. The report describes corrections (reductions) routinely applied to cyclic triaxial test strengths measured in clean sands to account for their deviation from both simple and multi-directional shear loading conditions, the latter of which is generally believed appropriate to simulate the possible effects of earthquake ground motions propagating upward through natural soil deposits. Cyclic triaxial tests, if properly interpreted, are an attractive means of establishing liquefaction resistance for comparison to in situ conditions and their use will likely continue. Appropriate corrections should be developed to adjust cyclic triaxial strengths measured in fine-grained soils to more closely model in situ behavior, as data used to develop the clean sand corrections were compared using relative density.

40. In situ testing research applied to fine-grained soils is reviewed, whereby liquefaction potential has been correlated to field performance of actual soil deposits during past earthquakes and cyclic strengths as measured by various means in the laboratory. In situ test correlations are ultimately desirable, in that large areas may be evaluated quickly and economically. Fine-grained soils do not present the obstacles to penetration tests of any kind that do soils containing gravel and larger particles, and much research has been directed to refining penetration test correlations. Standard Penetration Test (SPT) blowcount correlations are emphasized in this review, based on their popularity and increasing acceptance in the geotechnical engineering community.

PART V: RECOMMENDATIONS

41. Several Corps of Engineers dams in seismically active areas are founded on fine-grained, low plasticity alluvial deposits. Specific procedures should be formulated for use by Corps elements (and others, through technology transfer) to assess the potential for earthquake-induced liquefaction in fine-grained soil deposits.

42. No standard procedures currently exist to measure relative density in fine-grained soils as defined for this report, and experimental evidence to date supports the contention that relative density should not be considered as a control parameter in these materials. Future laboratory cyclic testing efforts should abandon the use of relative density and focus on one or more absolute parameters, such as specimen density or post-consolidation void ratio.

43. A program of 576 isotropically consolidated cyclic triaxial tests on controlled-mixture specimens is under way to evaluate the functional relationships between liquefaction resistance and fines type and content. Cyclic simple shear tests should also be performed using hollow, cylindrical specimens of mixtures selected from the cyclic triaxial test program to generate a body of data from which to develop appropriate simple shear correction factors. The findings reviewed in this report should be carefully considered in the conduct of these two testing programs, with particular emphasis on specimen preparation procedures.

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Firm	Project	Ratio of Undisturbed to Remolded Cyclic Strength*	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), follwed 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

* Cyclic strength defined as the ratio of pulsating deviator stress required to produce 5% double amplitude (peak-to-peak) axial strain or 100% pore pressure response in 10 cycles of load in an undrained cyclic triaxial test

Table 1. Comparison of Undisturbed and Remolded Strength (Mulilis, Chan, and Seed, 1975)

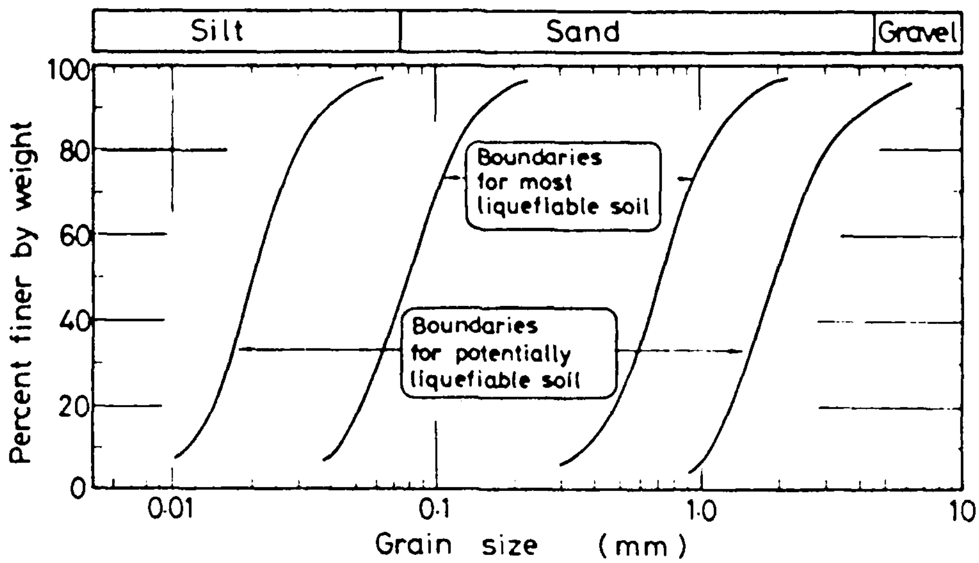


Figure 1. Gradations of liquefiable soils (Tsuchida, 1970)

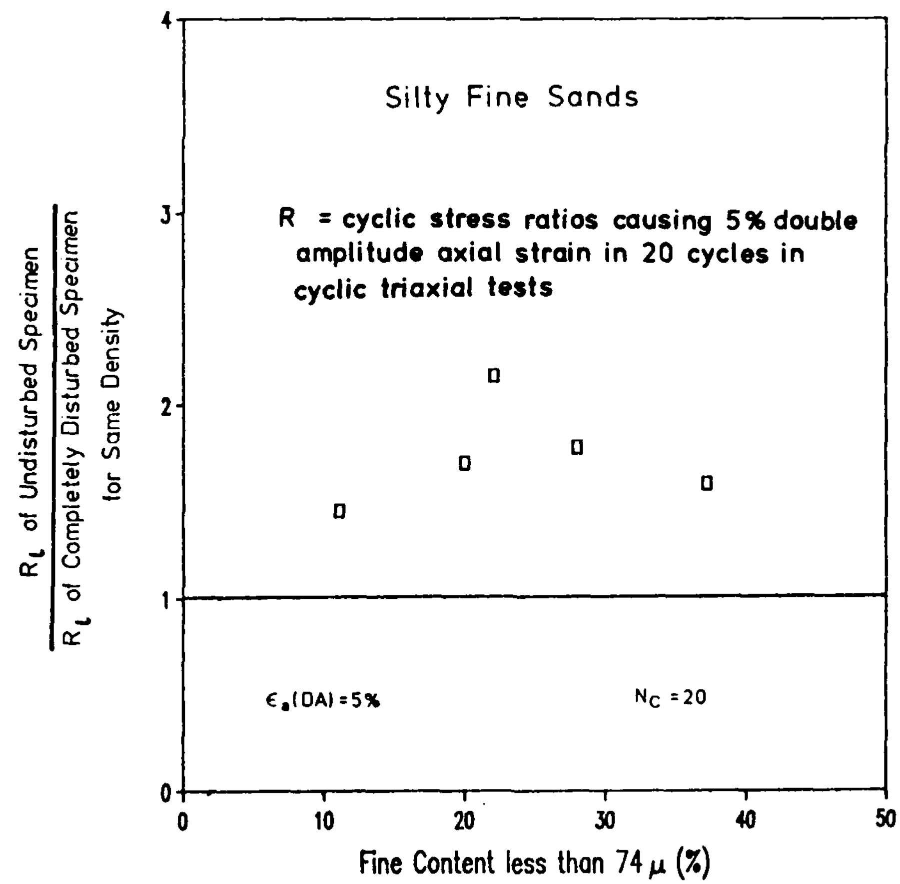


Figure 2. Comparison of liquefaction resistance between undisturbed and disturbed specimens of fine-grained soils (Tatsuoka, et al. 1978)

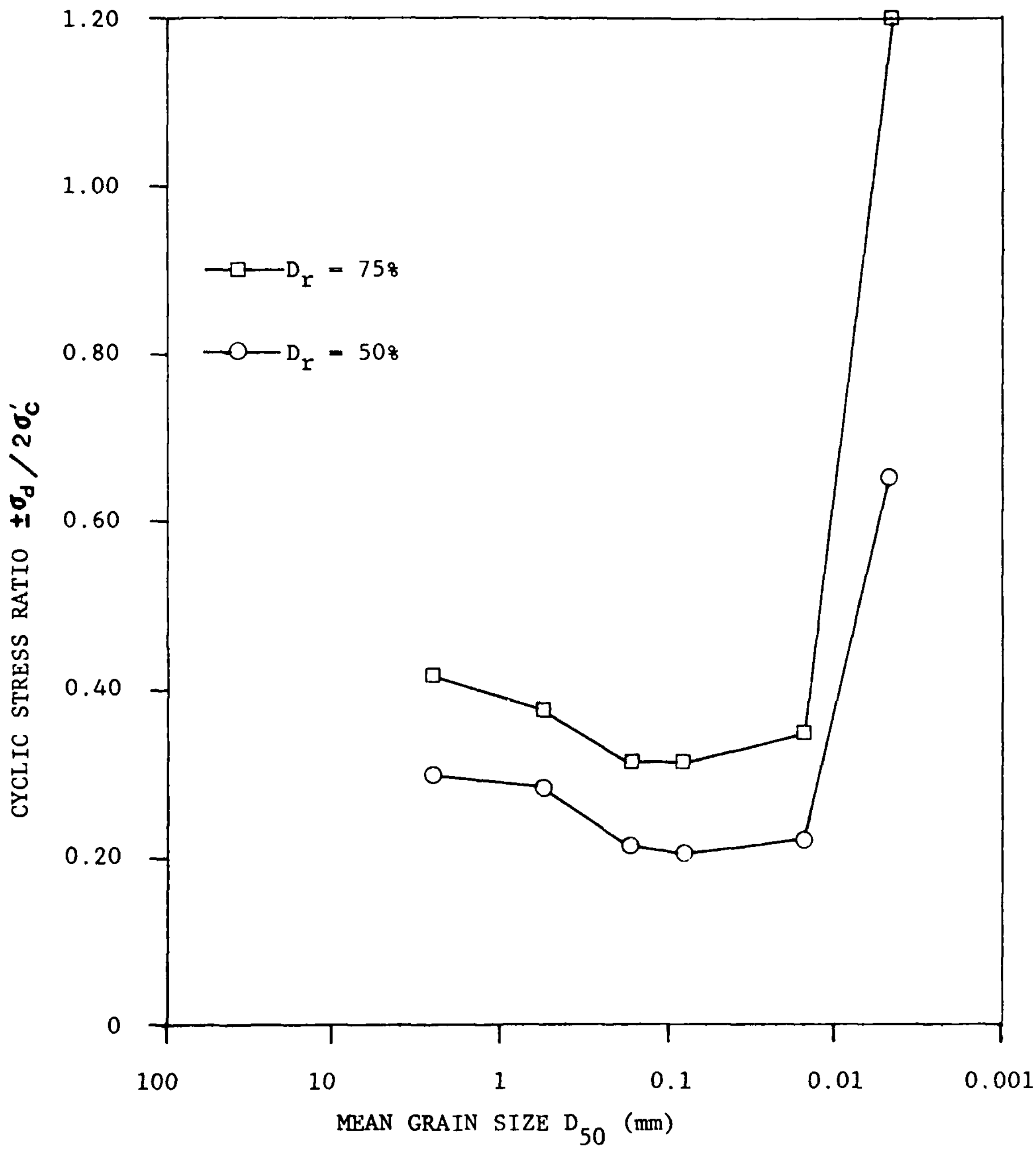


Figure 3. Effects of mean grain size and relative density on cyclic stress ratios causing 5% double amplitude cyclic axial strain in 30 cycles (after Lee and Fitton, 1969)

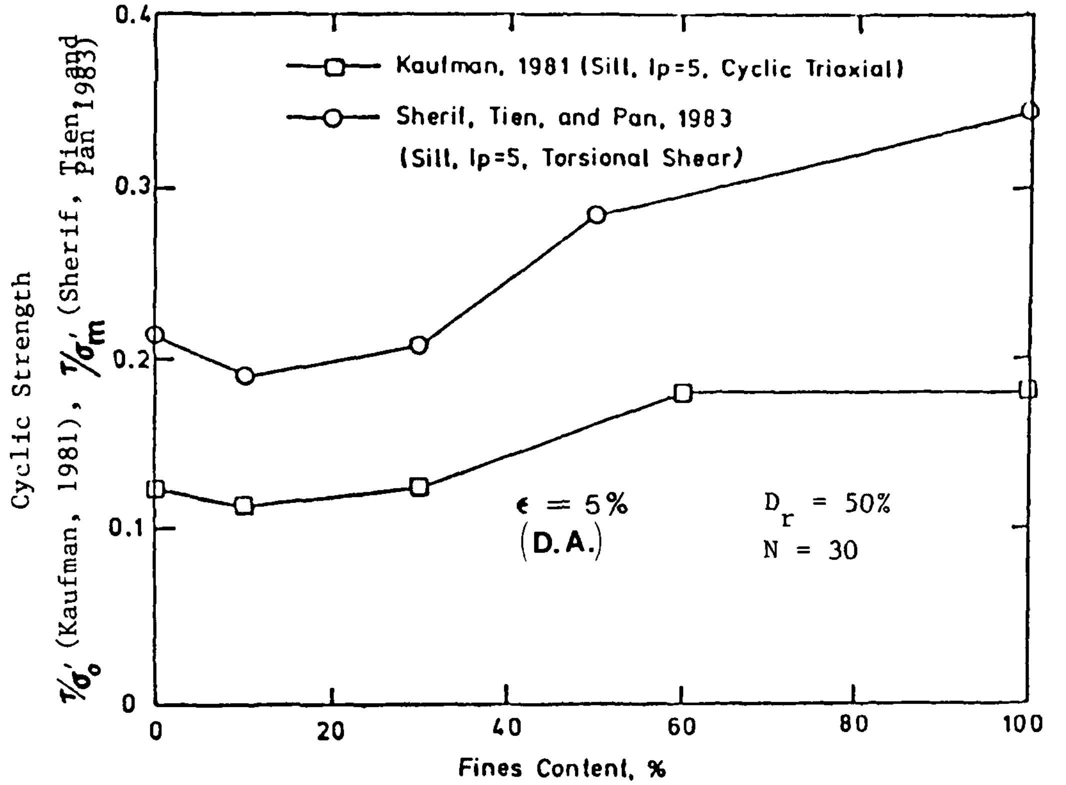


Figure 4. Effects of addition of low plasticity fines (silt) on cyclic triaxial and cyclic torsional shear strength

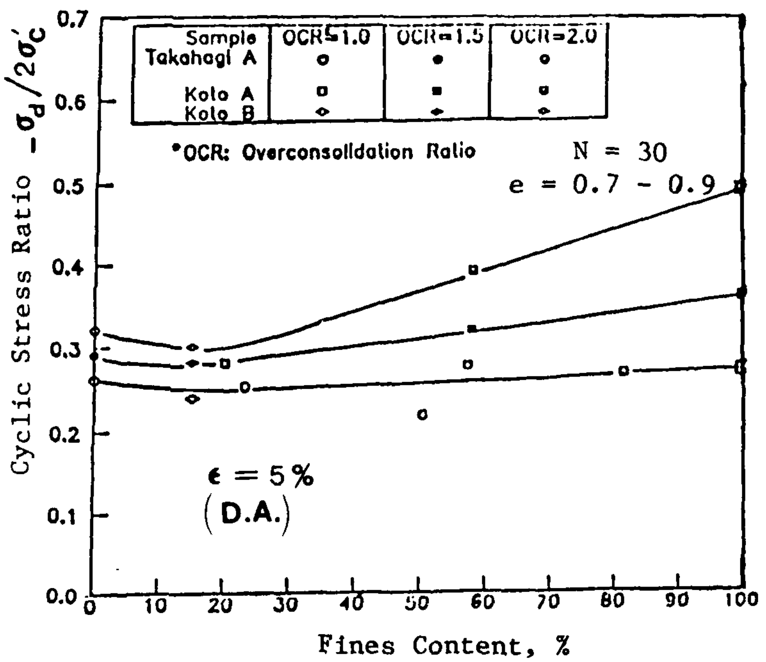
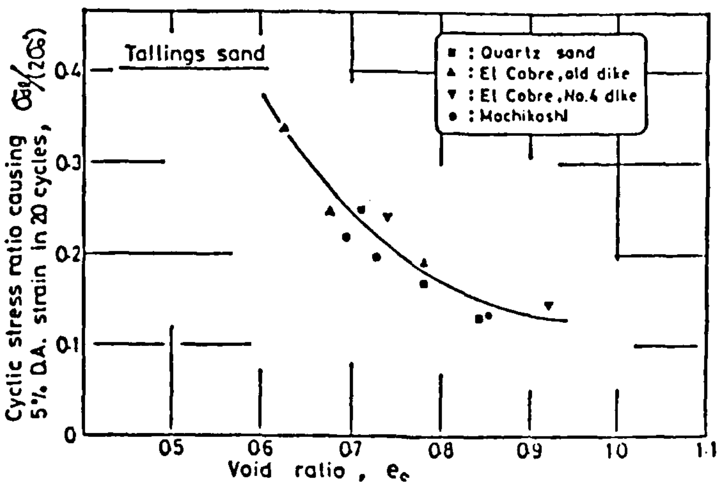
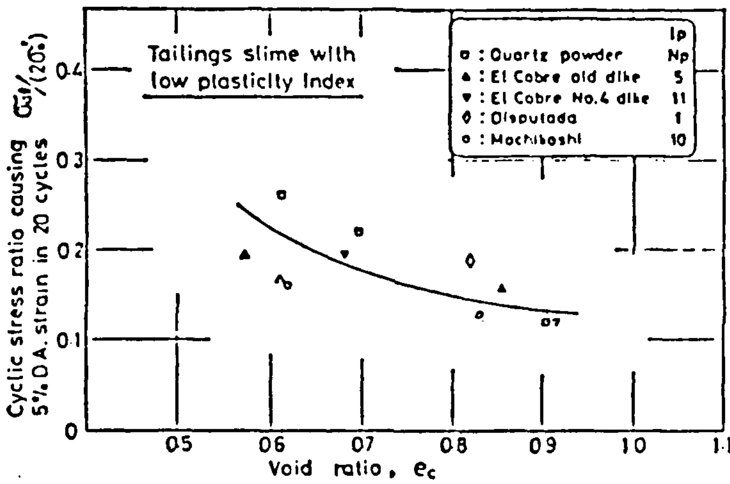


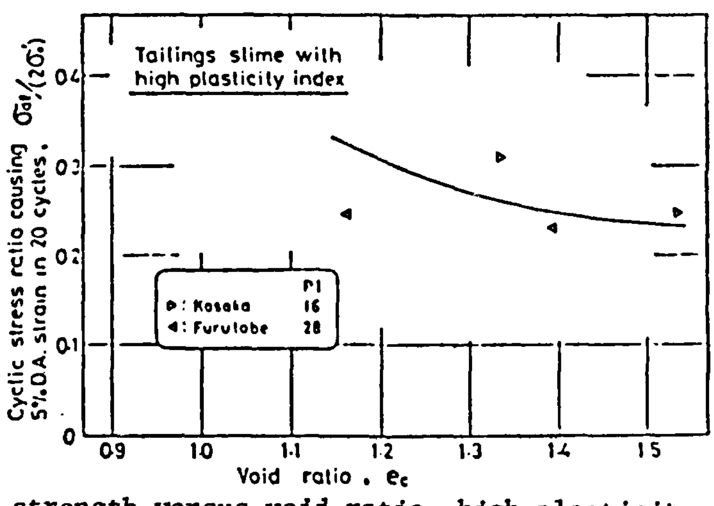
Figure 5. Effects of overconsolidation and fines (silt) content on cyclic triaxial strength of reconstituted specimens (after Ishihara, 1976)



(a) cyclic strength versus void ratio, tailings sands



(b) cyclic strength versus void ratio, low plasticity tailings slimes



(c) cyclic strength versus void ratio, high plasticity tailings slimes

Figure 6. Effects of index properties on cyclic triaxial strengths of tailings materials as a function of void ratio (Ishihara, et al., 1980)

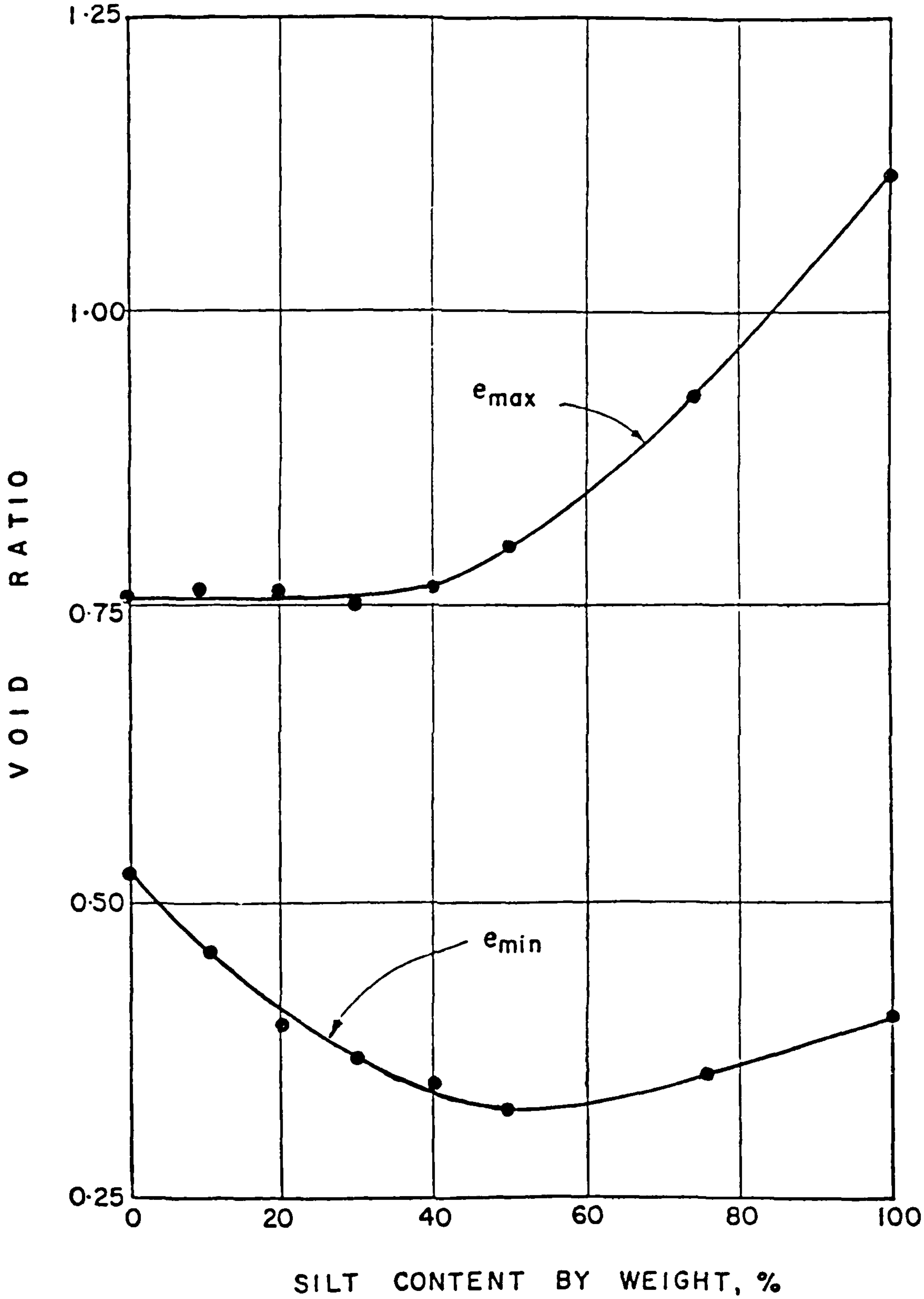


Figure 7. Relationship between maximum void ratio, e_{max}, minimum void ratio, e_{min}, and silt content (Sherif, Tien, and Pan, 1983)

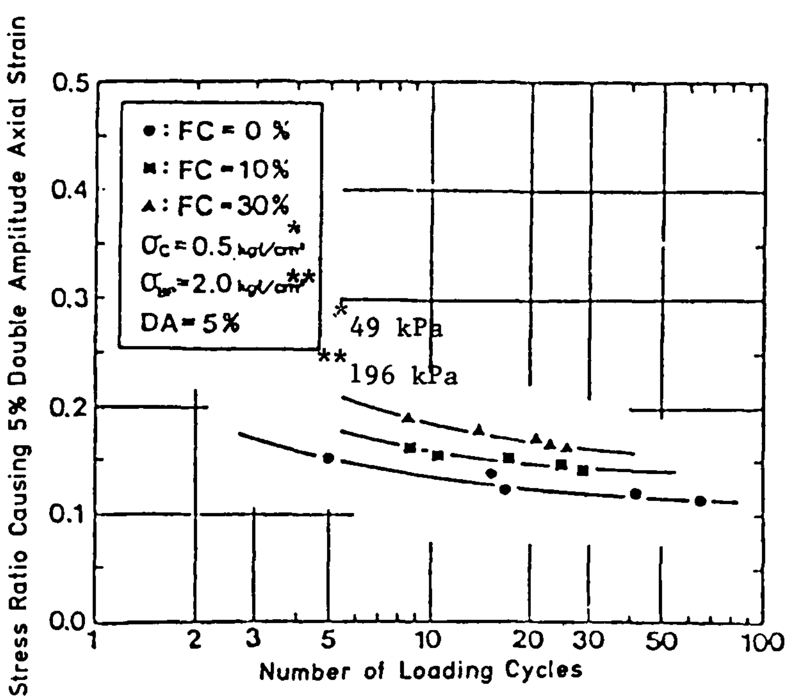


Figure 8. Effect of added fines (kaolin) on cyclic strength of Sengenyama sand (Kondoh, Sasaki, and Matsumoto, 1987)

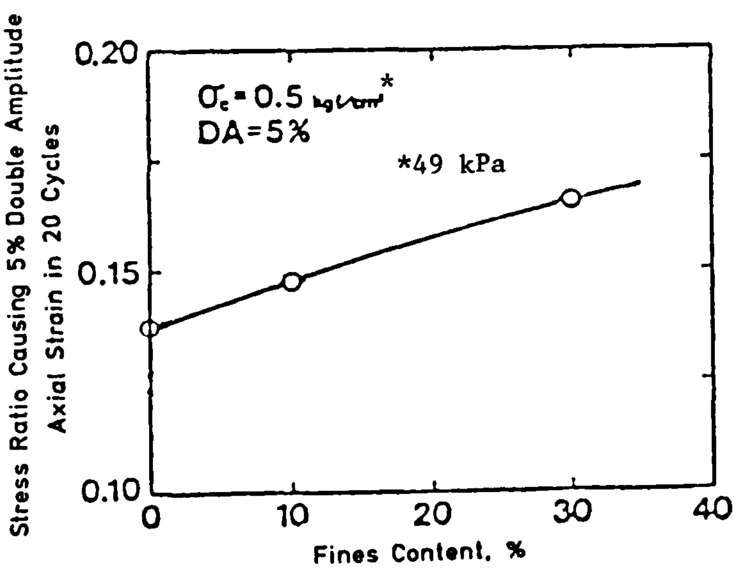


Figure 9. Effect of added fines (kaolin) on cyclic strength of Sengenyama sand at 20 loading cycles (Kondoh, Sasaki, and Matsumoto, 1987)

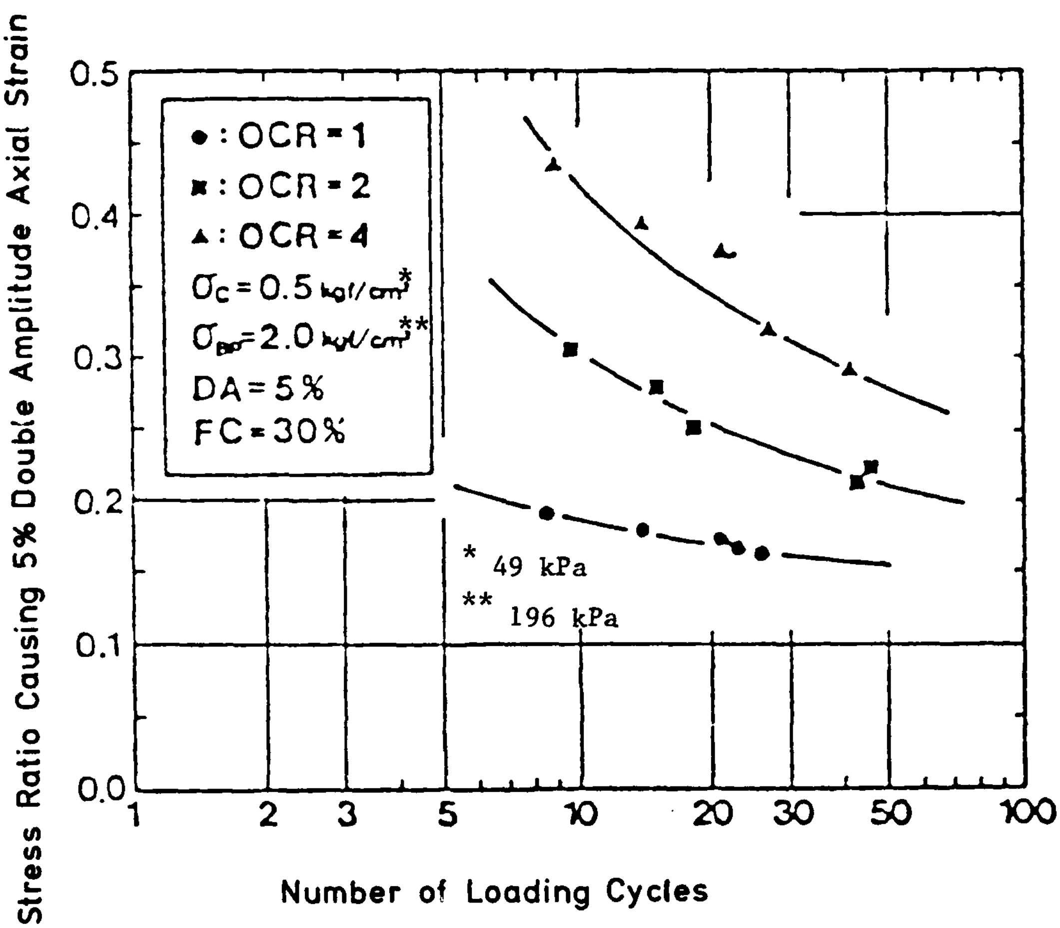


Figure 10. Effect of overconsolidation on cyclic strength of Sengenyama sand mixed with 30% kaolin (Kondoh, Sasaki, and Matsumoto, 1987)

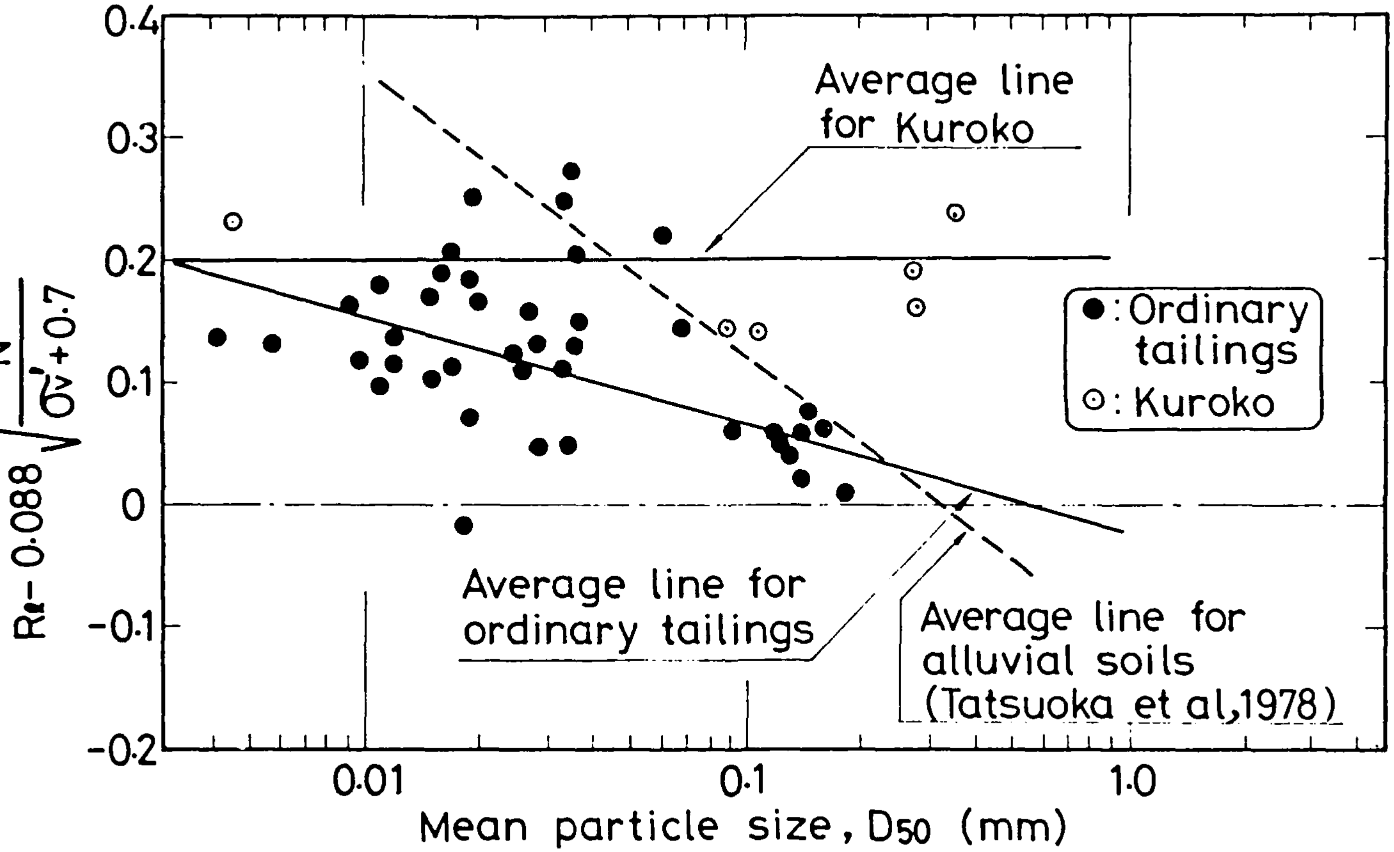


Figure 11. Chart used to develop corrections based on mean particle size for correlations of liquefaction resistance to SPT blowcounts (Ishihara, Yasuda, and Yokota, 1981)

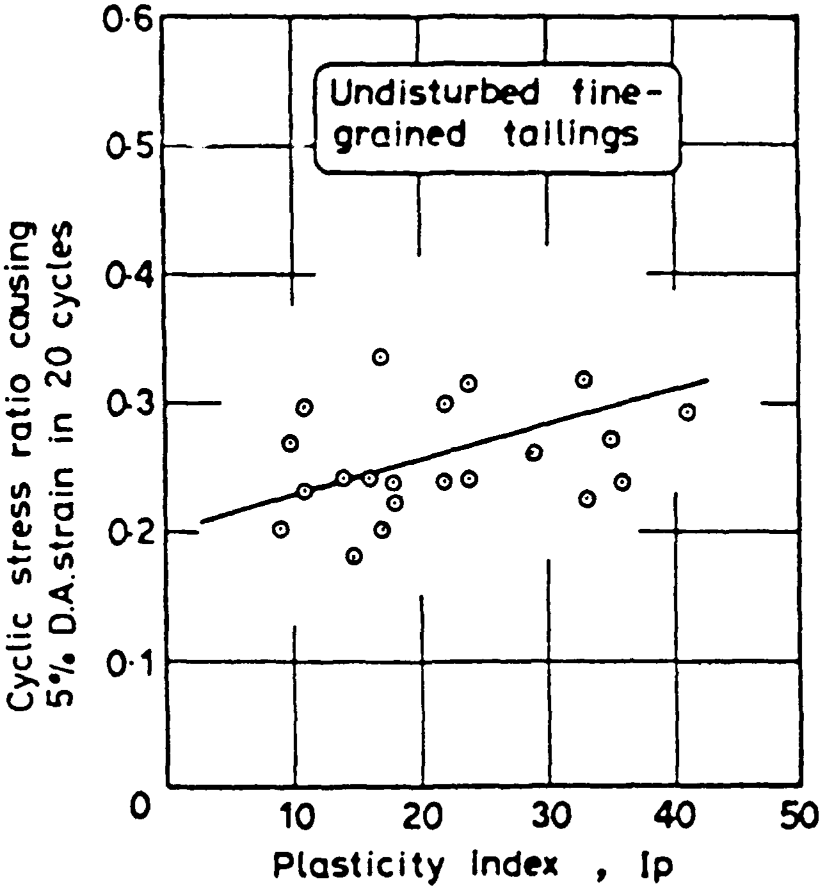


Figure 12. Relationship between cyclic strength and plasticity index of tailings (Ishihara, Yasuda, and Yokota, 1981)

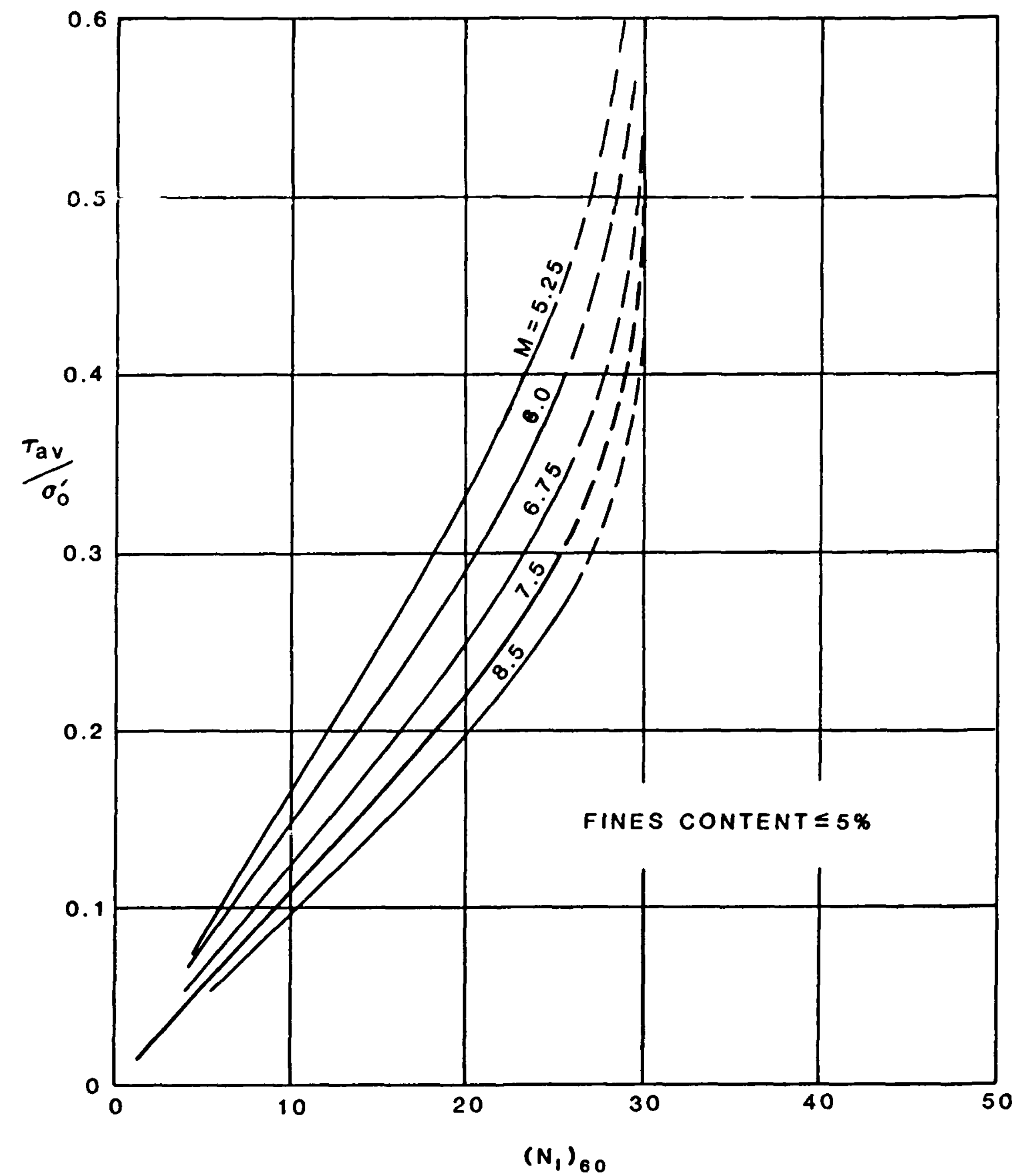


Figure 13. Chart for estimation of liquefaction resistance of clean sands using SPT data, for earthquakes of various Richter Magnitudes (Koester and Franklin, 1985, after Seed, et al., 1984)

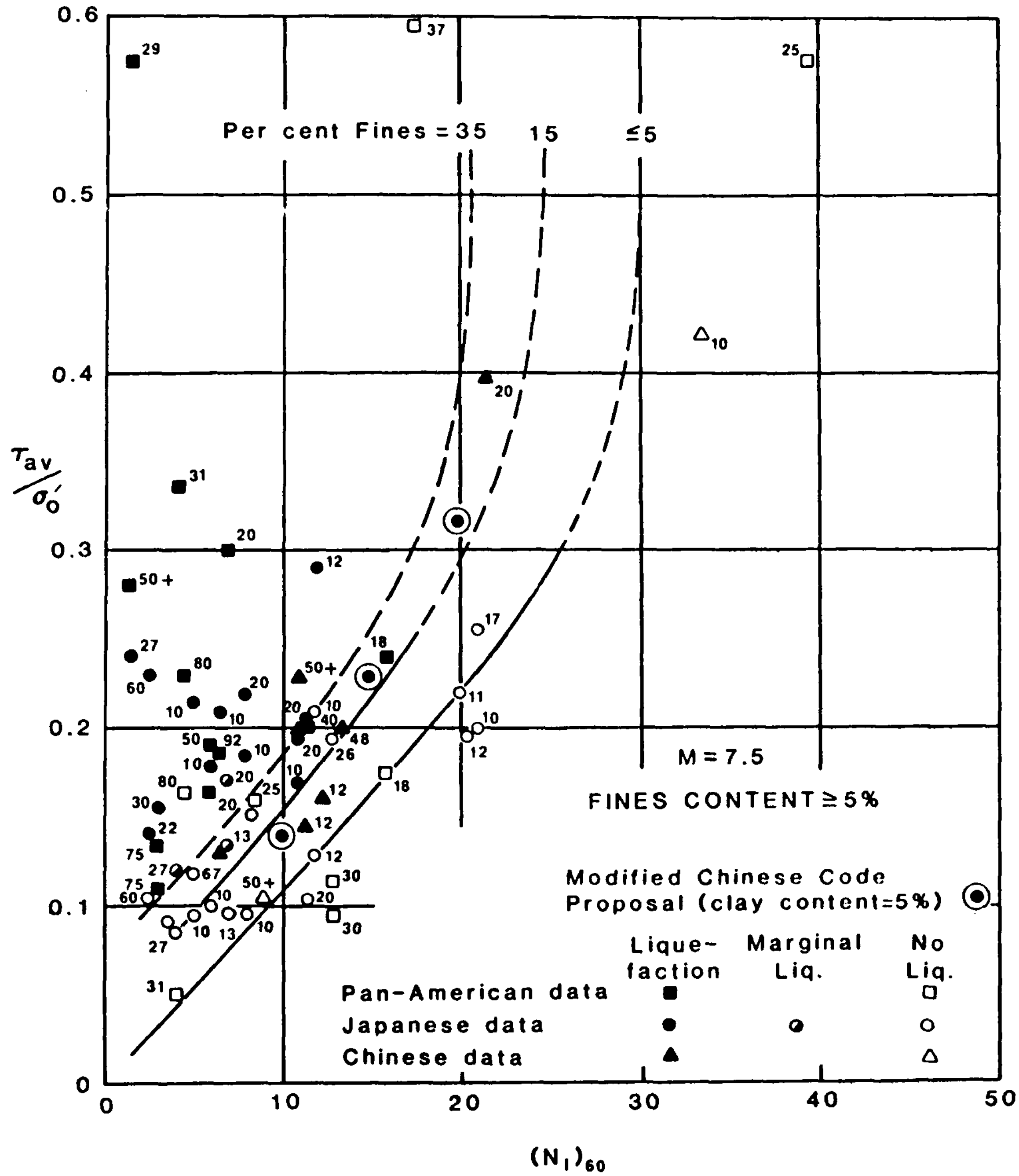


Figure 14. Chart for estimation of liquefaction resistance of silty sands using SPT data, for earthquakes of Richter Magnitude 7.5 (after Seed, et al., 1984)

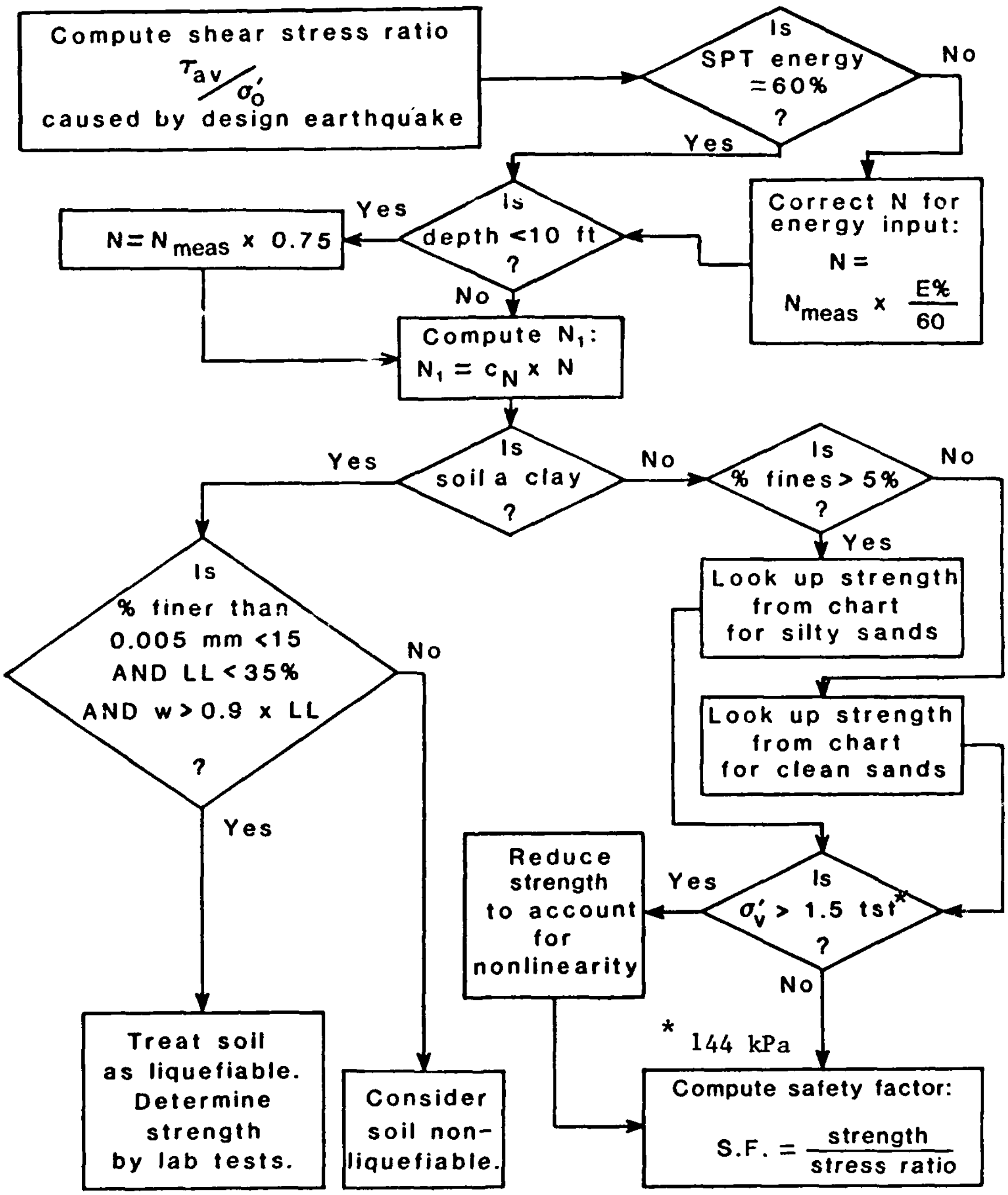


Figure 15. Procedure for empirical liquefaction evaluation using the SPT (Koester and Franklin, 1985)

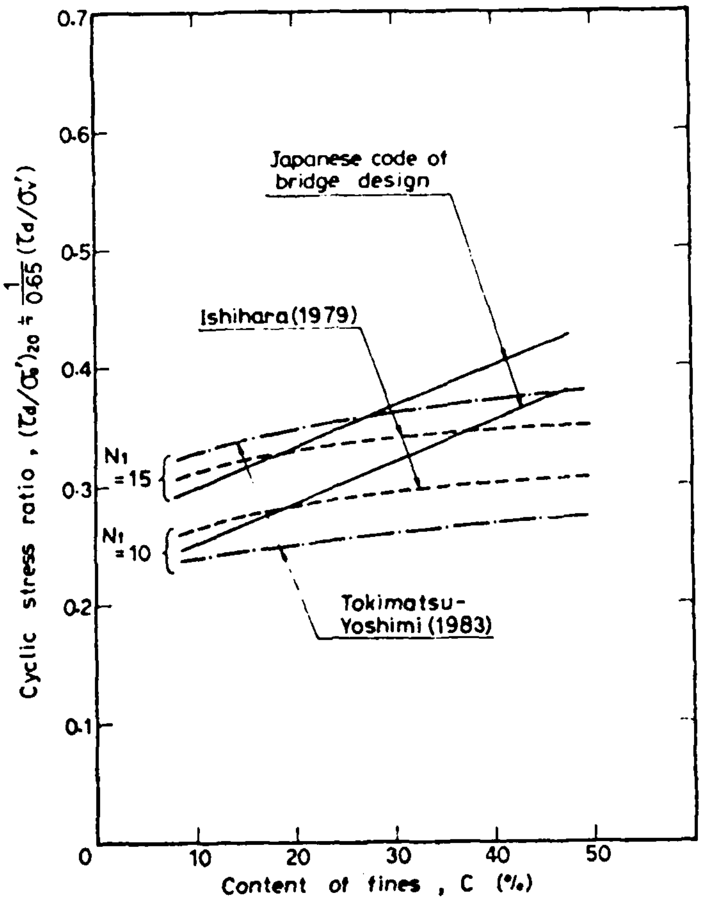


Figure 16. Effect of fines content on correlations of liquefaction resistance of sand-silt mixtures to SPT blowcounts (Ishihara, 1985)