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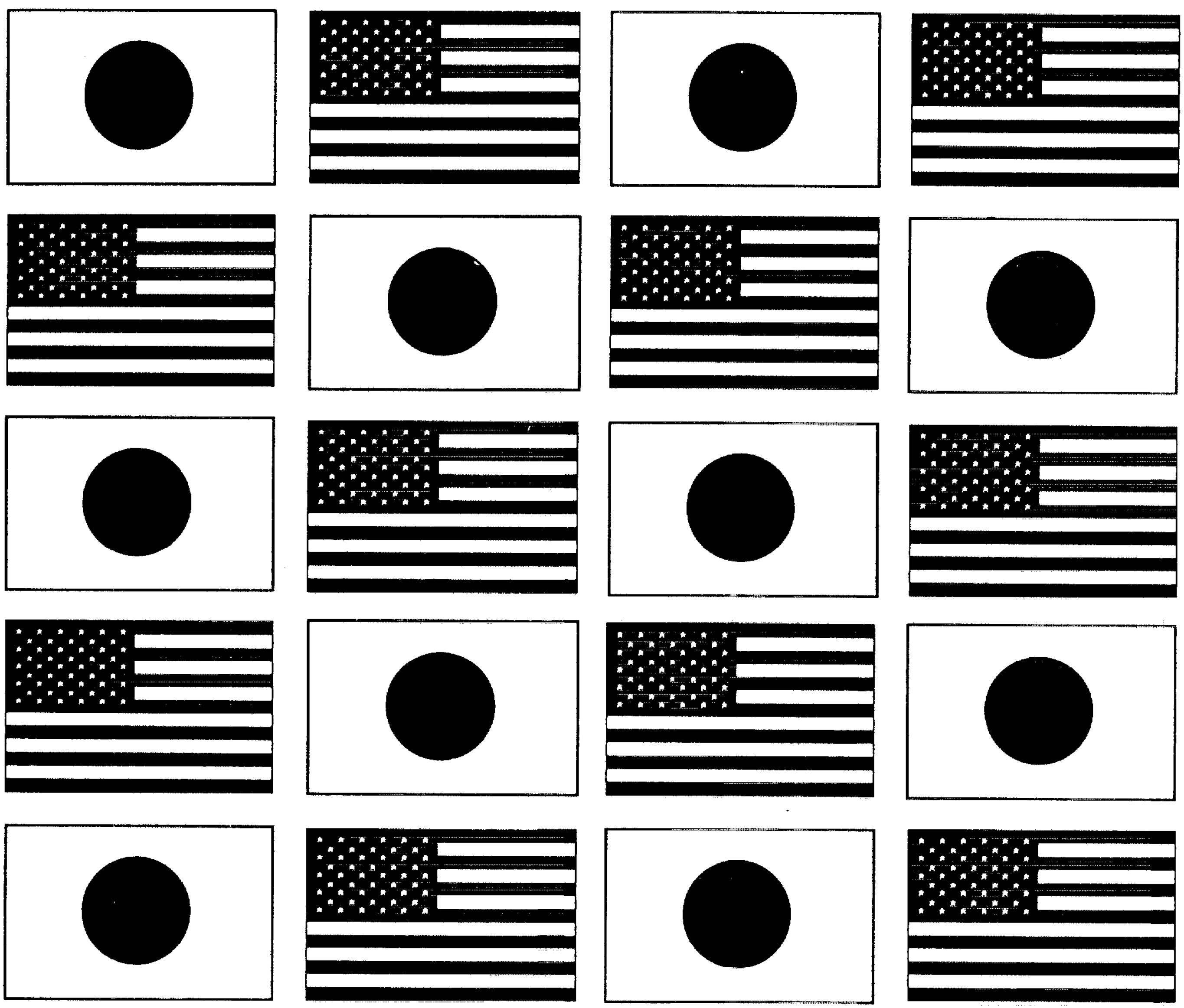

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Noel J. Raufaste
EDITOR

Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899

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The Influence of Confining Stress on Liquefaction Resistance

by

M. E. Hynes, R. S. Olsen and D. E. Yule*

Abstract

Laboratory measurements typically indicate that for a given soil, consistency (relative density for sands and gravels) and stress history there is a non-linear relationship between liquefaction resistance and confining stress (Seed and Idriss 1981; Seed 1983; Seed 1984; Vaid, Chern & Tumi 1985; Seed 1987; Hynes 1988; Harder 1988; Seed & Harder 1990; Pillai & Byrne 1994; Youd & Idriss 1998). Consequently, if cyclic strengths, either from laboratory measurements performed at a confining stress of 1 atm or estimated from correlations to in situ measurements such as Standard Penetration Tests (SPT), are linearly extrapolated to higher effective confining stress levels, the calculated liquefaction resistances may be too high. The effect of confining stress on liquefaction resistance is further complicated by soil compressibility and stress history.

The state-of-the-practice approach to account for the non-linear relationship between liquefaction resistance and vertical effective stress is to use published charts derived from existing laboratory data on similar materials or to determine a site specific relationship with a comprehensive laboratory testing program. Whichever approach is used, liquefaction resistance is conventionally represented as the Cyclic Resistance Ratio (CRR, the ratio of cyclic shear strength divided by the vertical effective stress, σ_v'). For a given soil at a given consistency and stress history, the CRR generally decreases with increasing vertical effective stress. This decrease is described by the factor K_σ which is defined as the ratio of CRR for a given σ_v' to the CRR at a vertical effective stress of 1 atm, CRR_1 (compared at the same relative density).

1. Introduction

The use of laboratory tests to establish CRR_1 for a material has decreased over the last decade in favor of in situ test correlations because of the cost-effectiveness of in situ measurements, the robustness of the Seed SPT-liquefaction chart (Seed et al. 1985, Youd and Idriss 1998), and concerns over sample disturbance and other issues associated with laboratory test results. The CRR_1 can be determined from in situ measurements such as the SPT, Cone Penetration Test (CPT), or shear wave velocity (V_s), or from laboratory measurements. The state-of-the-art for estimating CRR_1 using the SPT is given by Seed et al. (1985); using the CPT is given by Stark et al. (1995) or Olsen, Koester and Hynes (1996); and using V_s is given by Andrus and Stokoe (in Youd and Idriss 1998). The data base for these CRR_1 correlations consists of information from water-laid deposits of sands and silty sands, level to slightly sloping ground, under vertical effective stresses of less than 3 atm. Consequently, laboratory tests have been used to provide a relative scale to adjust the

*Earthquake Engineering and Geosciences Division, Geotechnical Laboratory
USAE Waterways Experiment Station
Vicksburg MS 39180-6199

CRR_1 values from in situ tests to higher confining stress levels and non-level ground stress conditions. The purpose of this study was to review and report the current state of knowledge with respect to the influence of overburden stress on liquefaction resistance.

2. Historical K_σ Data and Charts

Seed and Idriss (1981) to Seed and Harder (1990)

Early tests on sands and silty sands indicated considerable scatter in the values of K_σ. These data, summarized in Figure 1 and taken in part from Harder (1988), include Sacramento River sand (Lee 1965), Monterey No. 0 and Reid-Bedford sands (Townsend and Mullulis 1976), Upper and Lower San Fernando Dams (Seed et al. 1973) and Fort Peck Dam (Marcuson and Krinitzsky 1976). Superimposed on Figure 1 is an early K_σ relationship suggested by Seed and Idriss (1981). As more data became available, the K_σ chart was updated by Seed (1984 and 1987), Harder (1988), and Seed and Harder (1990), shown in Figure 3.

Upper and Lower San Fernando Dams, Seed et al. (1973) and Seed et al. (1989)

Seed et al. (1973) and Seed et al. (1989) report the results of cyclic triaxial tests on undisturbed samples of silty sand and sandy silt from the Upper and Lower San Fernando Dams (USFD and LSFD). LSFD came within inches of being overtopped after an upstream slide developed in the dam as a result of the 1971 San Fernando earthquake. USFD settled 3 ft and moved about 5 ft downstream, but did not fail; if it had failed, the downstream dam would also have been breached. This event resulted in the evacuation of 80,000 people immediately downstream of the two reservoirs.

Construction of LSFD began in 1912, and construction of USFD began in 1925. The method of construction was hydraulic fill, resulting in an in situ relative density of about 55 percent. Seed et al. (1973) and Seed et al. (1989) observed that the cyclic strength was about the same for the silty sands and sandy silts; the method of deposition resulted in a consistency that now had a uniform cyclic strength, although the material type changed. Comparison of the cyclic strength interpreted from SPT correlations was very similar but slightly greater than the strength determined from the undisturbed specimens of the hydraulic fill (Figure 4).

Fort Peck Dam, Marcuson and Krinitzsky (1976)

Marcuson and Krinitzsky (1976) report the results of cyclic triaxial tests on undisturbed and reconstituted (wet pluviation) samples of hydraulic fill and foundation soils from Fort Peck Dam. Construction of Fort Peck Dam began in 1934. Relative densities determined in the laboratory resulted in values of about 40 to 50 percent, rather than the 70 percent average value indicated from the Gibbs and Holtz relationships for corresponding blowcounts. Marcuson and Krinitzsky (1976) made a comparison of the cyclic strength of undisturbed and reconstituted specimens at a confining stress of 8.3 psi (0.57 atm). These data indicate that the cyclic strength of the undisturbed specimens was about 80 (foundation material) to 150 (shell material) percent greater than for reconstituted specimens.

Sardis Dam, U.S. Army Engineer District, Vicksburg (1985, 1988)

Sardis Dam is one of the three hydraulic fill dams owned by the Corps of Engineers. Reevaluation of the seismic stability of Sardis Dam began shortly after the near catastrophic failure of LSFD, in part because of its proximity to the New Madrid fault zone in Central U.S. Sardis Dam was constructed in the late 1930's. Material mixtures in the fill and foundation range from sands to silts to high-water-content, low-plasticity clays. The results of cyclic laboratory tests, conducted over the course of the reevaluation, were provided by U.S. Army Engineer District, Vicksburg, for inclusion in this study.

Olsen (1984)

Olsen (1984) generalized the data trends from the preliminary K_σ relationship by Seed (1984), together with project data at WES into the following expression: $K_\sigma = (\sigma'_v)^{f-1}$. Olsen (1984) reported that the stress exponent, f, ranged from 0.6 to 0.95 with 0.7 recommended for sands. This recommendation is similar to the updated Seed curves shown in Figure 2, and also bounds the data from Vaid and his colleagues described next.

Vaid et al. (1985), Vaid and Thomas (1994)

Cyclic strength, K_σ for pluviated clean sands

Vaid et al. (1985) conducted cyclic triaxial tests on two fine, clean sands, well rounded Ottawa sand and an angular tailings material, to investigate the effects of relative density, compressibility, confining stress, and particle angularity on cyclic strength. CRR for confining stresses of 200, 800, 1600 and 2500 kPa were determined as a function of consolidated relative density. The samples were constructed by dry pluviation, and vibrated to increase density.

Vaid et al. (1985) observed that there are competing effects of increased confining stress; one is densification due to the compressibility of the material with increasing confining stress which has the effect of increasing CRR and the other is the curvature in the cyclic strength envelope which tends to decrease CRR as confining stress increases. If CRR values are compared at the same consolidated relative density, the data indicate a trend toward convergence of the cyclic strength curves, developed for different confining stresses, at lower relative densities. These data imply that K_σ is approximately one for very loose materials, and increases as relative density increases. These data indicate that K_σ for clean sands and silty sands have a limiting value of about 0.6 at a confining stress of about 8 atm. The cyclic triaxial test data from Vaid and Thomas (1994) on Fraser River sand (dry pluviated and vibrated specimens) follow a similar trend.

Stress focus plots of cyclic strength and CRR for pluviated clean sands

Trends and deviations in geotechnical data with confining stress are sometimes easier to see when the data are plotted in log-log plots. These log-log plots are termed stress focus plots from Olsen (1994). If data fit as a straight line on a log-log stress focus plot, then that data is well fitted by a simple exponential curve. Stress focus plots were used in this study as a framework for investigating confining stress effects on cyclic strength and CRR. The Vaid et al. (1985) and Vaid and Thomas (1994) data are plotted on stress focus charts ($\log_{10}$ cyclic strength plotted versus $\log_{10}$ effective confining stress, atm) in Figure 5. This figure shows that at confining stresses of 400 kPa and greater, the cyclic strength envelope is well fitted with an exponential curve (plots as a straight line on a log-log plot).

These data follow the stress focus concept, namely that the cyclic strength curves for different relative densities tend to converge as confining stress increases. This point (or zone) of convergence is termed the stress focus. As shown by Olsen (1994), the location of the stress focus is a function of soil type and mineralogy. The slope of a line in a stress focus plot corresponds to the inverse of the exponent used by Olsen (1984) to describe K_σ: $K_\sigma = (\sigma'_v)^{f-1}$. Consider generalized stress focus cyclic strength lines for a very loose, medium dense and dense sand. As density increases, the cyclic strength at a confining stress of 1 atm, CRR_1, increases. As density increases, the slope, $1/f$, of the stress focus cyclic strength line increases. The corresponding K_σ curves are determined as: $K_\sigma = (\sigma'_v)^{f-1}$ As density increases, the exponent f decreases and K_σ decreases, resulting in a more severe reduction to CRR.

Gravelly Soils, Hynes (1988 and 1996)

Hynes (1988) conducted 15-in. diameter cyclic triaxial tests on moist tamped gravel specimens from Mormon Island Auxiliary Dam, constructed to relative densities of about 40 and 64 percent. Similar tests were conducted at WES on Ririe Dam gravels (Sykora et al. 1991) compacted to a relative density of about 45 percent, and Success Dam gravels compacted to a relative density of about 50 percent (U.S. Army Engineer District, Sacramento 1996). Banerjee et al. (1979) conducted tests on well-compacted Oroville Dam gravels (relative density of about 84 percent, 12-in. diameter, specimens constructed in 6 layers with surface vibrator applied for compaction). Cyclic strengths for this study were compared at N = 10 load cycles.

Hynes (1996) observed that the cyclic strength envelopes for these gravels have an upward turn at low confining stress, less than 3 atm. Hynes (1996) proposed that this behavior is caused by a stress history that the sample construction process builds into the specimen, causing it to behave as an overconsolidated (oc) soil; oc material has greater axial and horizontal stiffness, is more resistant to volume change, and thus is more resistant to development of residual excess pore pressures when subjected to cyclic loading. In a stress focus chart, over-consolidation is indicated by a steepening of the cyclic strength line as confining stress decreases. Over-consolidation results in reduced values of K_σ.

The K_σ values interpreted from the gravel data depend upon how the CRR_1 is estimated (also the failure criterion used such as maximum pore pressure response and cyclic strain level, and membrane compliance corrections applied). The most conservative K_σ values are obtained by straight-line projection of the cyclic strength-effective confining stress Mohr-Coulomb envelope to estimate CRR_1. The resulting K_σ values for Oroville gravels are : $K_\sigma = 0.54$ at 2 atm and 0.2 at 8 atm confining stress. The most optimistic K_σ values are obtained by ignoring the apparent "overconsolidated" data at 2 atm, and then projecting the cyclic strength envelope to the origin. This results in K_σ values of 0.50 at 8 atm and 0.44 at 14 atm for Oroville gravels. The range of K_σ interpretations for the gravel data collected in this study are plotted in Figure 6, with the Seed and Harder (1990) curve for reference.

Except for the most optimistic interpretations, these gravel data generally plot below the data for clean and silty sands (resulting in a more severe K_σ reduction to CRR_1).

Byrne & Harder (1991), Pillai & Byrne (1994) and Pillai & Stewart (1994)

Byrne and Harder (1991) selected K_σ values for clean sands from previous work to develop a recommendation for the clean sands and gravels present at Terzahgi Dam, Canada. Their data set included the work by Vaid and his colleagues at the University of British Columbia at Vancouver (UBC), as well as clean sand data from Seed and Harder (1990). This clean sand curve is plotted in Figure 7. Pillai and Byrne (1994) estimated K_σ values for Duncan Dam foundation materials. At Duncan Dam, the foundation contains a water-laid unit of very fine sand (D_{50} = 0.2 mm) with 5 to 8 % fines (typical gradations shown in Figure 11a). Foundation soils were frozen in situ to extract high quality samples for laboratory testing. Pillai and Byrne (1994) and Pillai and Stewart (1994) report constant values of CRR for confining stresses of 2 to 12 atm from cyclic triaxial laboratory results, and also constant values of CRR for confining stresses of 2 to 6 atm from cyclic simple shear laboratory results. If CRR is constant regardless of confining stress (for a fixed relative density), then K_σ is equal to one.

The in situ freezing procedure resulted in undisturbed samples with a wider range of void ratios and slightly lower average than the values measured in situ by other means (including gamma-gamma logging). When these samples were reconsolidated to effective confining stress levels of 2, 4 and 6 atm, they densified. The CRR values at 10 cycles correspond to a relative density of about 55 percent, and indicate K_σ values of about one.

For the liquefaction analysis of the fine sand unit in the Duncan Dam foundation, CRR was estimated to be constant with confining stress in the fine sand, with CRR = 0.12 (earthquake magnitude = 6.5). Pillai and Byrne (1994) and Pillai and Stewart (1994) state that the N-values measured in the unit at various effective confining stress levels (various overburden depths) were converted to $N_{1,60}$ values using an energy correction measured for the SPT equipment and Gibbs and Holtz (1957) relative density relationships to determine N_1 for a given relative density (known from the gamma-gamma logging and undisturbed samples). CRR_1 values from Seed's liquefaction chart (Seed et al. 1985) were inferred from the $N_{1,60}$ values. K_σ values were estimated as the ratios of CRR (equal to 0.12) to CRR_1 values. This procedure combines changes in CRR due to confining stress with changes due to densification, and requires considerable confidence in the relationships used to determine $N_{1,60}$ for a given relative density. However, in this case, the resulting CRR is about the same as would be obtained if densification effects are treated separately and C_N and K_σ corrections are determined for fixed values of relative density.

Gibbs and Holtz (1957) performed chamber tests on a coarse sand (D_{50} = 1.5 mm) with zero fines and a fine sand (D_{50} = 0.3 mm) with 14 percent fines. Gibbs and Holtz (1957) performed chamber tests on dry, moist and saturated coarse sand specimens and dry and saturated fine sand specimens. They used confining stresses of 0 (self-weight), 10, 20 and 40 psi (0, 0.68, 1.36 and 2.72 atm). For the Duncan Dam analysis, it appears that $N_{1,60}$ values were linearly interpolated from the measured value of $N_{1,60}$ = 10 for the fine sand at 1 atm and D_r = 30

% , to the estimated value of $N_{1,60} = 19$ at $D_r = 65$ % , using Gibbs and Holtz (1957) relationships as a guide.

Marcuson and Bieganousky (1976) performed SPT chamber tests to determine C_N corrections and relative density relationships for fine to coarse sands with low fines contents. They used two fine sands, Reid-Bedford Model sand and Ottawa sand, with gradations similar to Duncan Dam. Marcuson and Bieganousky (1976) used effective confining pressures of 10, 40 and 80 psi (0.68, 2.72 and 5.44 atm). Their relationship between $N_{1,60}$ and D_r for fine, submerged sand is similar to the Gibbs and Holtz (1957) relationship for dry coarse sand. The Marcuson and Bieganousky (1976) data indicate that C_N values decrease (a more severe reduction from N to N_1) as grain size and relative density decrease (Hynes-Griffin and Franklin 1989). Skempton (1986), Olsen (1994) and Olsen and Mitchell (1995) provide summaries of data and estimated C_N corrections. The potential range in C_N values is large.

It is difficult to compare the K_σ values from the Duncan Dam study with other sources since: a) the data reduction process couples densification due to increased confining stress with confining stress effects on cyclic strength and penetration resistance; and b) the resulting confining stress corrections are highly site specific. In this case, the confining stress and densification effects appear to cancel each other, so CRR would be found to be constant with depth using procedures that treat densification and confining stress separately.

Consensus K_σ Curve (Harder 1996, Youd and Idriss 1998)

The K_σ correction was an issue discussed at the Salt Lake City Workshop (Youd and Idriss, editors, 1998). Published K_σ trends indicate a wide range of possible K_σ relationships. After a presentation by Harder (1996) and considerable discussion by the participants, a consensus K_σ relationship was selected as the clean sand curve from Byrne and Harder (1991, Figure 7). It roughly corresponds to the average for all tested soil types and relative density levels, and serves as a lower bound for medium dense to loose sands and silty sands. The data that support the consensus K_σ curve correspond generally to medium dense clean sands. The consensus K_σ curve may be unconservative for gravelly soils or very dense soils, and may be very conservative for loose sands and silts.

Summary K_σ Curves for Different Soil Types

Clean Sands. The K_σ data for clean sands are grouped in Figure 8. The data from Vaid show a consistent trend that K_σ decreases as relative density increases for loose to medium dense clean sands. However, the CRR_1 values determined from the laboratory tests are significantly less than would be estimated for a clean sand at that relative density based on blowcount correlations.

Gravels. The limited data for gravels

indicate lower values of K_σ than for sands and silty sands. (A lower value of K_σ means a more severe reduction in cyclic strength.) However, the range of possible values of K_σ is large, depending on the procedure used to estimate CRR_1 . The K_σ boundary for the clean sand data is the upper bound for interpretation of the gravel data. The K_σ values are minimum when CRR_1 is estimated from straight-line extrapolation in a shear strength vs. confining stress plot, with samples at lower confining stresses behaving as overconsolidated material. The K_σ values are maximum when CRR_1 is estimated from straight-line extrapolation in a shear strength vs. confining stress plot, ignoring "overconsolidated" data.

Hydraulic Fill and Foundation Soils. The K_σ and cyclic strength data for hydraulic fills are shown in Figures 9 and 10. Data from loose samples of silty sand formed by wet pluviation (Fort Peck Dam shell and foundation material) have K_σ values of about 0.9 to 0.95 at confining stresses of 6 to 8 atm. Values of K_σ from tests on undisturbed samples of hydraulic fill from Sardis, USFD and LSFD generally plot below the earlier Seed curves. The USFD data form a lower bound to the K_σ data set for hydraulic fill material. Figure 9 also shows K_σ data for silty, sandy foundation soils (primarily alluvial) from Sardis, Fort Peck, Arcadia and Enid Dams.

Soil Mixtures (Silty, Sandy Foundation Soils). Figures 11 and 12 show K_σ and cyclic strength values from cyclic triaxial tests on undisturbed samples of silt, carved from block samples of the foundation materials at Enid Dam. Also shown in these figures are K_σ values for foundation soils at Arcadia Dam. The Arcadia samples were undisturbed, thin-wall shelby tube samples. For Enid silt, the maximum value of K_σ results from including densification of the sample during consolidation. The minimum value comes from comparing cyclic strengths at a constant void ratio. The Enid and Arcadia points plot close to the hydraulic fill data from USFD, LSFD and Sardis Dam. Consequently, the USFD data also form a lower bound K_σ curve for silty, sandy mixtures.

Stress History, Undisturbed samples, Reconstructed Specimens and CRR_1

Stress History Effects The database of cyclic strength and K_σ values assembled for this study indicate that K_σ is very sensitive to the stress history of the soil. The effect of the in situ stress history on cyclic strength can be obscured due to sample disturbance unless special measures, such as freezing (Singh et al. 1982, Tani and Yasunaka 1988), are taken to minimize disturbance. Reconstruction of specimens in the laboratory may introduce an artificial stress history, as is caused by moist tamping, or result in a near virgin state, as is the case with wet or dry pluviation.

Undisturbed and Moist-Tamped Specimens Increased cyclic strength caused by stress history effects introduced during laboratory construction of a specimen may not simulate in situ conditions unless a similar stress history exists in the field. Samples constructed by moist tamping may approximately simulate a stress history for rolled-fill materials (which are known to perform very well under cyclic loading, Seed et al. 1977). However, for cohesionless soils, high quality undisturbed samples obtained by in situ freezing are needed to quantify past stress history effects on cyclic strength and determine the appropriate K_σ correction (Singh et al. 1982).

Undisturbed and Pluviated Specimens Sample construction by wet or dry pluviation may simulate deposition of hydraulic fill or foundation deposits beneath a dam, typically recent alluvium, colluvium or lacustrine deposits. However, the in situ deposit may have some stress history and aging effects that are not simulated by freshly constructed laboratory samples. These in situ stress history and aging effects would be most apparent at low levels of confining stress. An elevated cyclic strength at low confining stress leads to a more severe K_σ correction. If high quality undisturbed samples are used to determine cyclic strength as a function of confining stress, as was the case for LSFD and USFD, then the cyclic strength may be somewhat elevated at low confining stress and the K_σ correction less than one. If recently constructed (pluviated) laboratory tests are used for relative scaling of strength to extend field correlations, then the K_σ correction may be close to one, and cyclic strength at high confining stresses may be overestimated. This can be observed by comparing the K_σ correction for LSFD and USFD with that for Fort Peck Dam.

CRR_1 from Laboratory Tests Compared with CRR_1 from Penetration Tests Cyclic strengths of undisturbed samples correspond fairly well with cyclic strengths inferred from penetration tests (examples: USFD and LSFD data; data from Japan on samples excavated using in situ freezing reported in Prakash and Dakoulas 1994). Cyclic strengths of samples prepared by wet or dry pluviation are generally much lower than expected for a soil at that relative density (examples: data from Vaid and others for clean sands; comparison of cyclic strength of reconstructed specimens and undisturbed specimens from Fort Peck Dam). The cyclic strength data for gravels (Folsom, Ririe and Oroville Dams) were compared with an estimated CRR_1 for soils at that relative density. Estimation of CRR_1 from the laboratory gravel data in stress focus plots leads to values that are significantly less than expected for a given relative density. This underestimation of CRR_1 can also be observed in the Fraser River sand data; the CRR_1 values from the laboratory tests on freshly constructed pluviated specimens are much less than would be expected for a material at that relative density.

The data for freshly deposited materials (Fort Peck Dam, Vaid) indicate a straight line relationship on the stress focus plot but the CRR_1 values are well below (80 to 150 percent too low for Fort Peck) those expected for a material at that relative density. Consequently, if specimens are constructed by wet or dry pluviation, the results should be similar to the observations from Fort Peck and Vaid's work, and are not necessarily representative of field conditions; these results may also greatly overestimate K_σ . The Duncan Dam laboratory data contradict this trend; these data were obtained from cyclic tests on undisturbed samples of fine, clean sand that had been frozen prior to sampling. The freezing process has been shown to preserve stress history effects on cyclic strength (Singh et al. 1982). The Duncan Dam laboratory data indicate K_σ equal to one for confining stresses ranging from 2 to 12 atm.

The stress focus theory described shows that the cyclic strength envelopes for a given gradation and mineralogy merge at very high confining stress, regardless of stress history and relative density. In the stress range of interest for dams, however, cyclic strength envelopes can be very sensitive to these factors. The cyclic strength chart from Seed et al. (1983) shows blowcount values at sites that have and have not liquefied. For the sites that did liquefy, the process of liquefaction in the field may have the effect of redepositing the affected soils, thus erasing past stress history, and resulting in a near virgin deposit. For the sites that did not liquefy, the past stress history may be preserved and possibly added to by the earthquake shaking.

3. Conclusions

1. Laboratory cyclic strength tests on relatively loose soil samples reconstituted by dry or wet pluviation result in high K_σ values (nearly linear strength envelope with intercept near zero, little reduction in CRR as confining stress increases).

2. The CRR_1 values for reconstituted, pluviated specimens are too low by about a factor of two when compared to CRR_1 values from tests on high quality undisturbed specimens.

3. The CRR_1 values for reconstituted, pluviated specimens are too low by about a factor of two when compared to CRR_1 values estimated from penetration tests for soils at about the same relative density as the reconstituted specimens.

4. The K_σ data base developed in this study from laboratory tests on undisturbed samples of fine sands, silts and silty sands, including hydraulic fill and water-laid foundation deposits, indicates K_σ values ranging from a minimum of 0.55 to 0.45 (LSFD and USFD), and a maximum of 1 to 0.9 (Duncan Dam) for confining stresses of 4 to 6 atm.

5. Experience at USFD and LSFD indicates that although material type is variable in hydraulic fills, the method of deposition results (for practical purposes) in a uniform relative density and a uniform cyclic strength.

6. It is hypothesized that stress history (including aging effects) is an important factor in determining appropriate values for K_σ . Pluviation in the laboratory results in near-virgin specimens, but the cyclic strengths of these specimens does not correspond to values from undisturbed

samples and inferred from penetration tests. Moist-tamping results in specimens with considerable stress history that may or may not correspond well to field conditions.

7. Direct measurement of K_σ in the laboratory will require testing high quality undisturbed specimens. Pluviated, reconstituted specimens may overestimate K_0 . Moist-tamped, reconstituted specimens may underestimate K_σ .

8. Stress Focus theory, originally developed for interpreting CPT cone resistance, was investigated as a framework for interpreting the non-linear relationship between cyclic strength and confining stress. The data indicate a locus for a stress focus boundary for liquefaction resistance which varies with soil type. The stress focus format simplifies the mechanics of relating CPT measurements to soil properties.

4. Recommendations

1. Before conducting a costly, complex laboratory testing program for site-specific values of K_σ , parametric dynamic analyses and liquefaction evaluations should be conducted to determine whether a detailed laboratory testing program is warranted.

2. Initially, the analyses should consider the most optimistic K_σ values. If the dam or site is judged unsafe with these values, remedial construction is indicated.

3. If the dam is found to be safe, with factors of safety against liquefaction greater than 2, the analyses should then be performed with the lower bound K_σ values.

4. If the dam is still judged to be safe with these values, for example with factors of safety against liquefaction greater than 1.5, then further laboratory testing or analysis may be unnecessary to demonstrate adequate seismic performance.

5. However, if the analyses (with the lower bound K_σ values) indicate factors of safety less than 1.5, then a laboratory testing program is recommended to quantify the appropriate K_σ relationships for this site.

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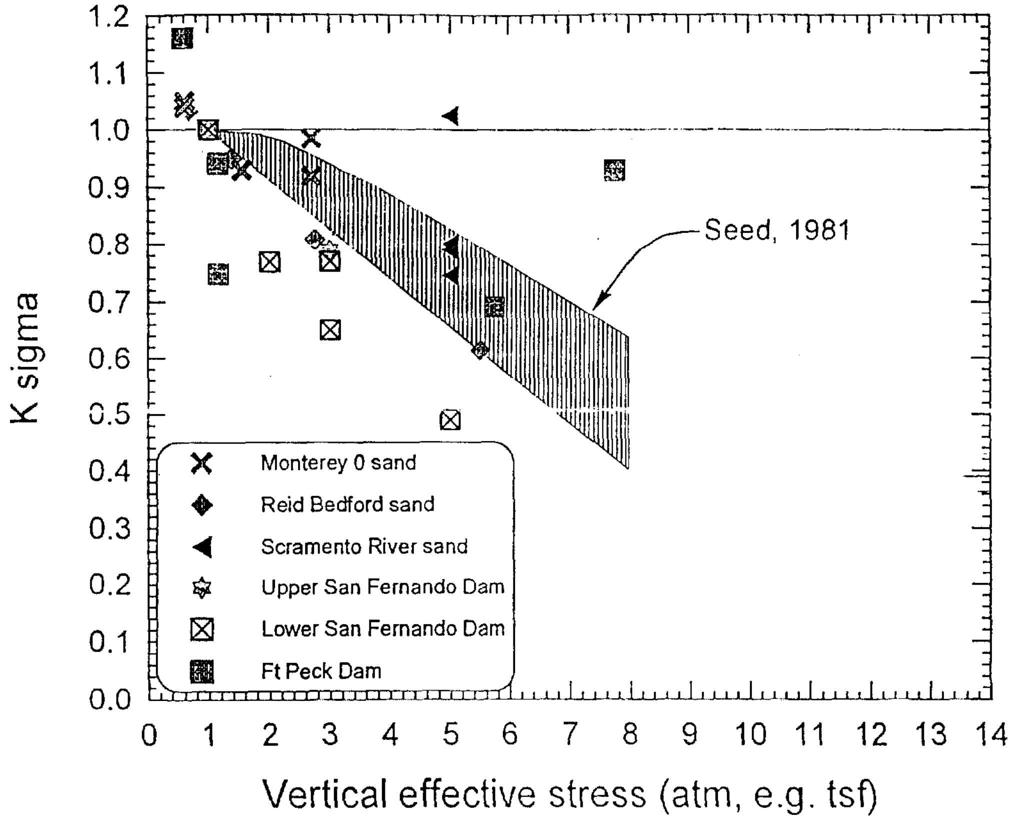

Figure 1 Relative reduction in cyclic stress ratio causing liquefaction with increase in confining stress, K_σ (Seed and Idriss 1981)

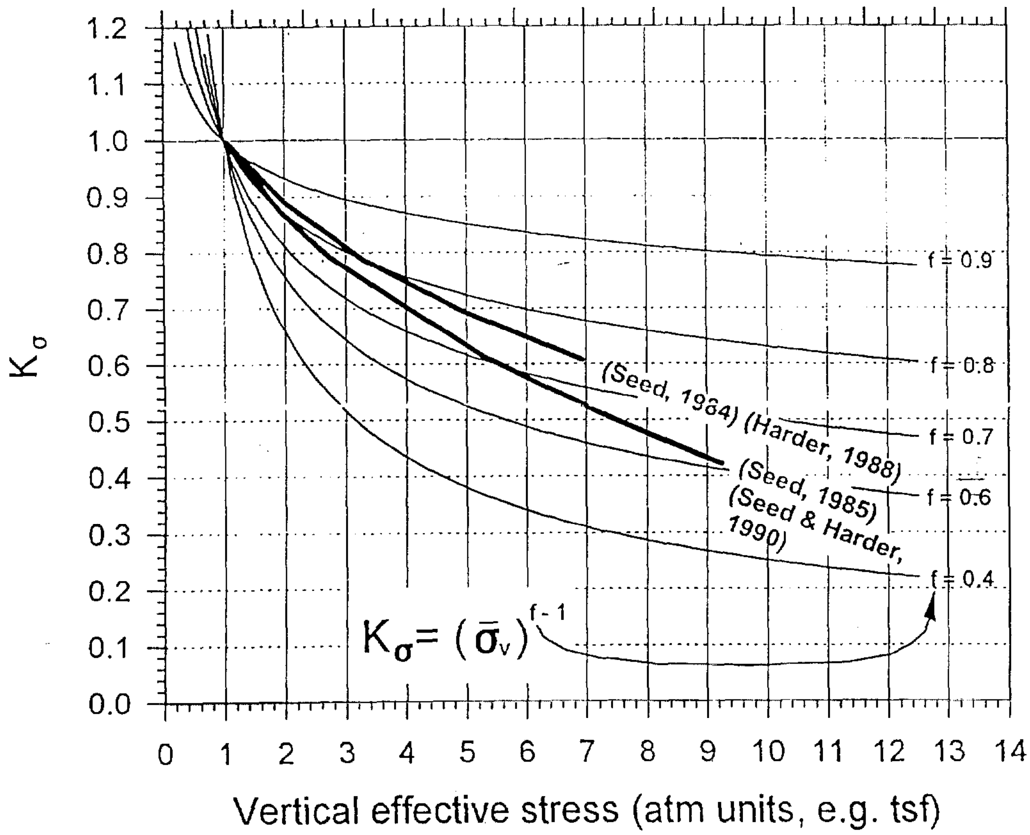

Figure 2 Contours of liquefaction stress exponents on the K_σ chart

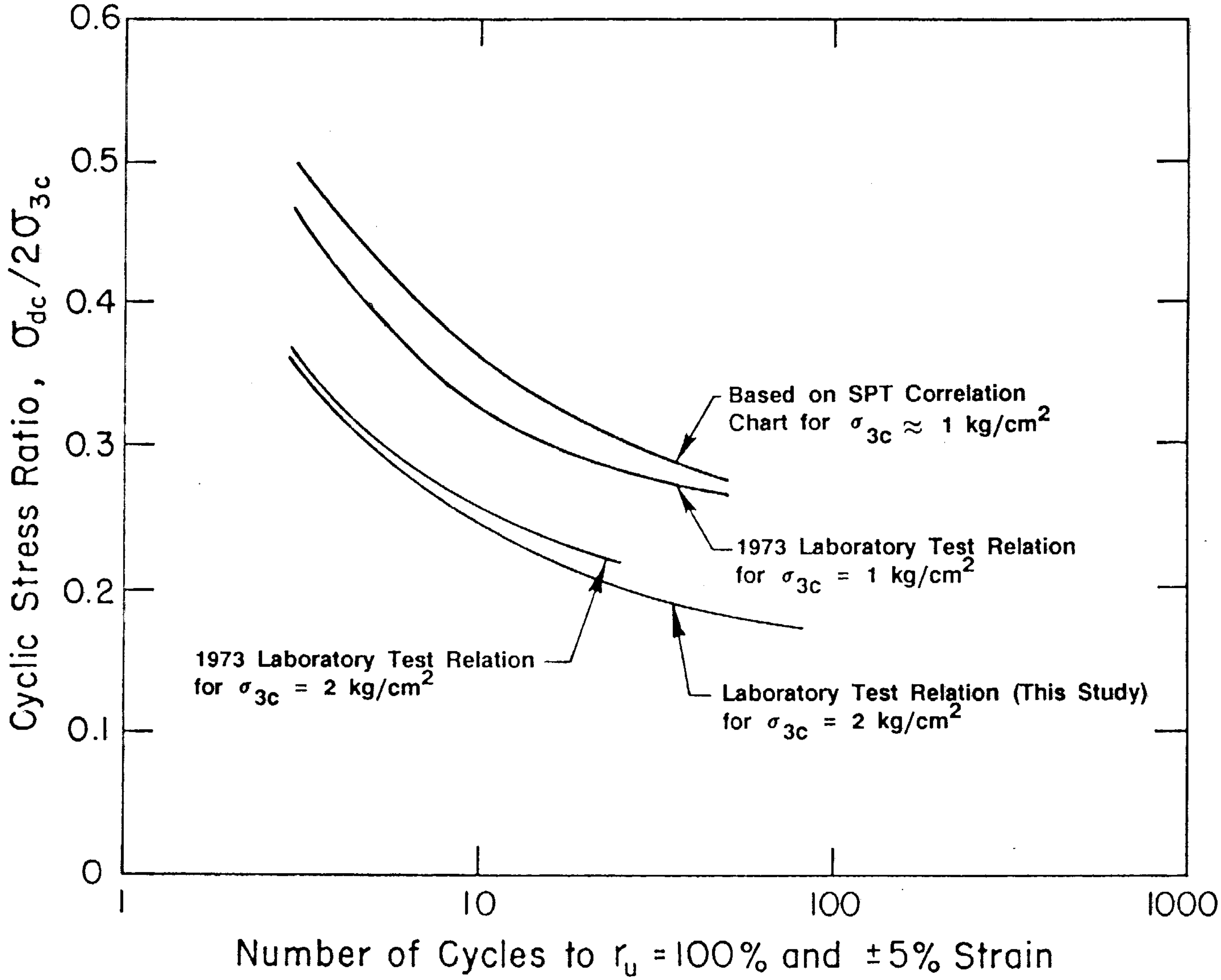

Figure 3 Comparison of results of laboratory cyclic load tests with data determined from field case studies (Seed et al. 1989)

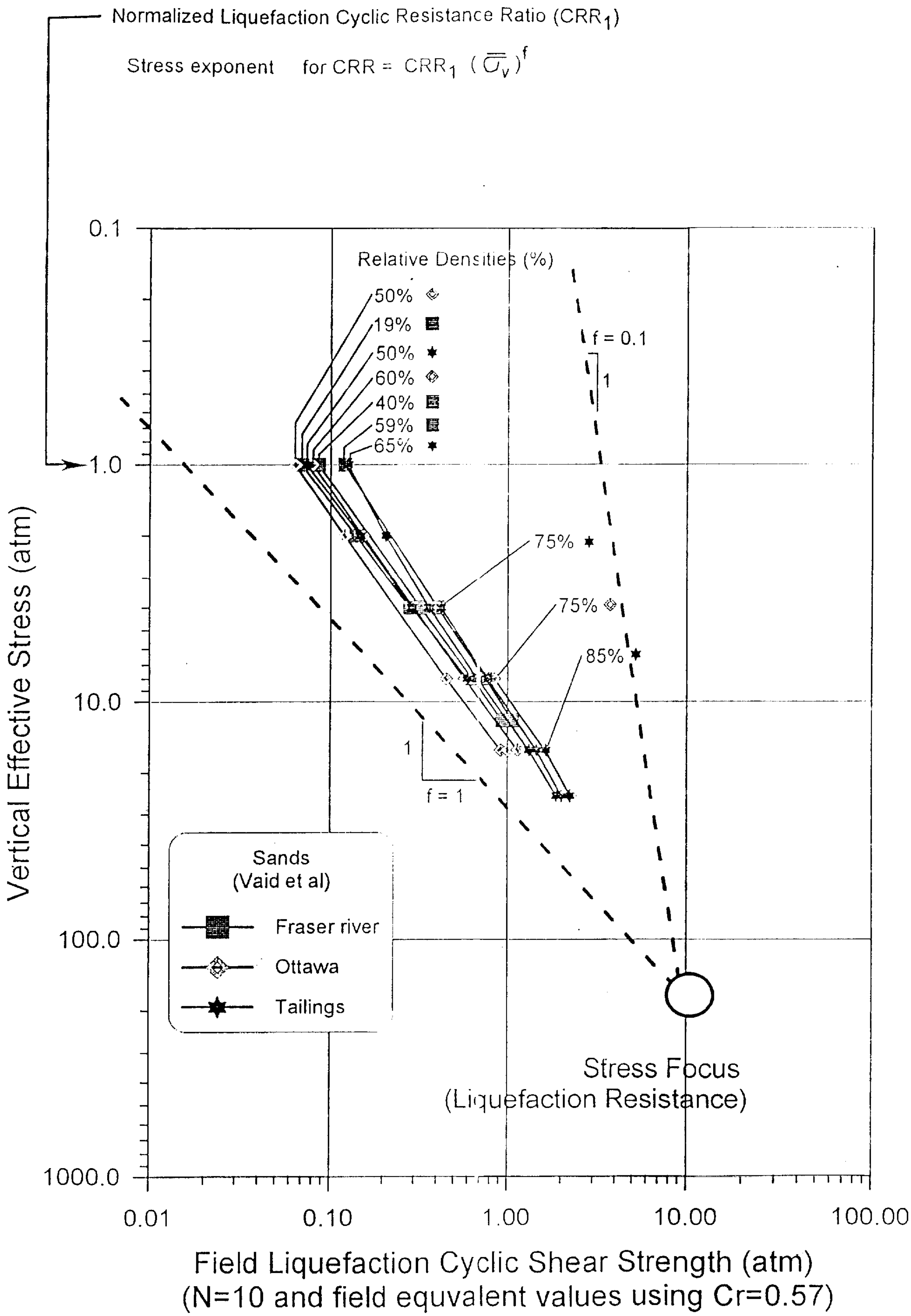

Figure 4 Stress focus plot of cyclic strength for tailings, Ottawa and Fraiser River sands (after Vaid et al. 1985 and Vaid and Thomas 1994)

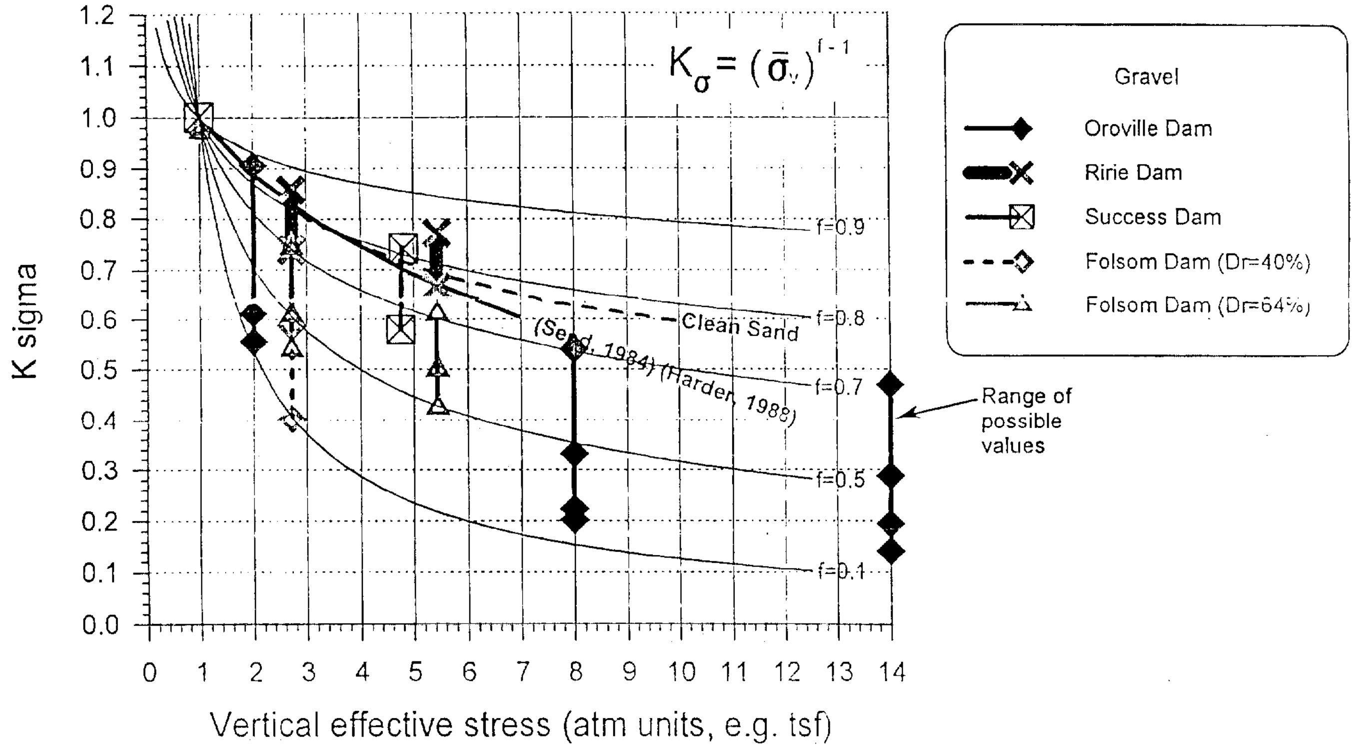

Figure 5 K_σ and CRR stress focus data for gravels

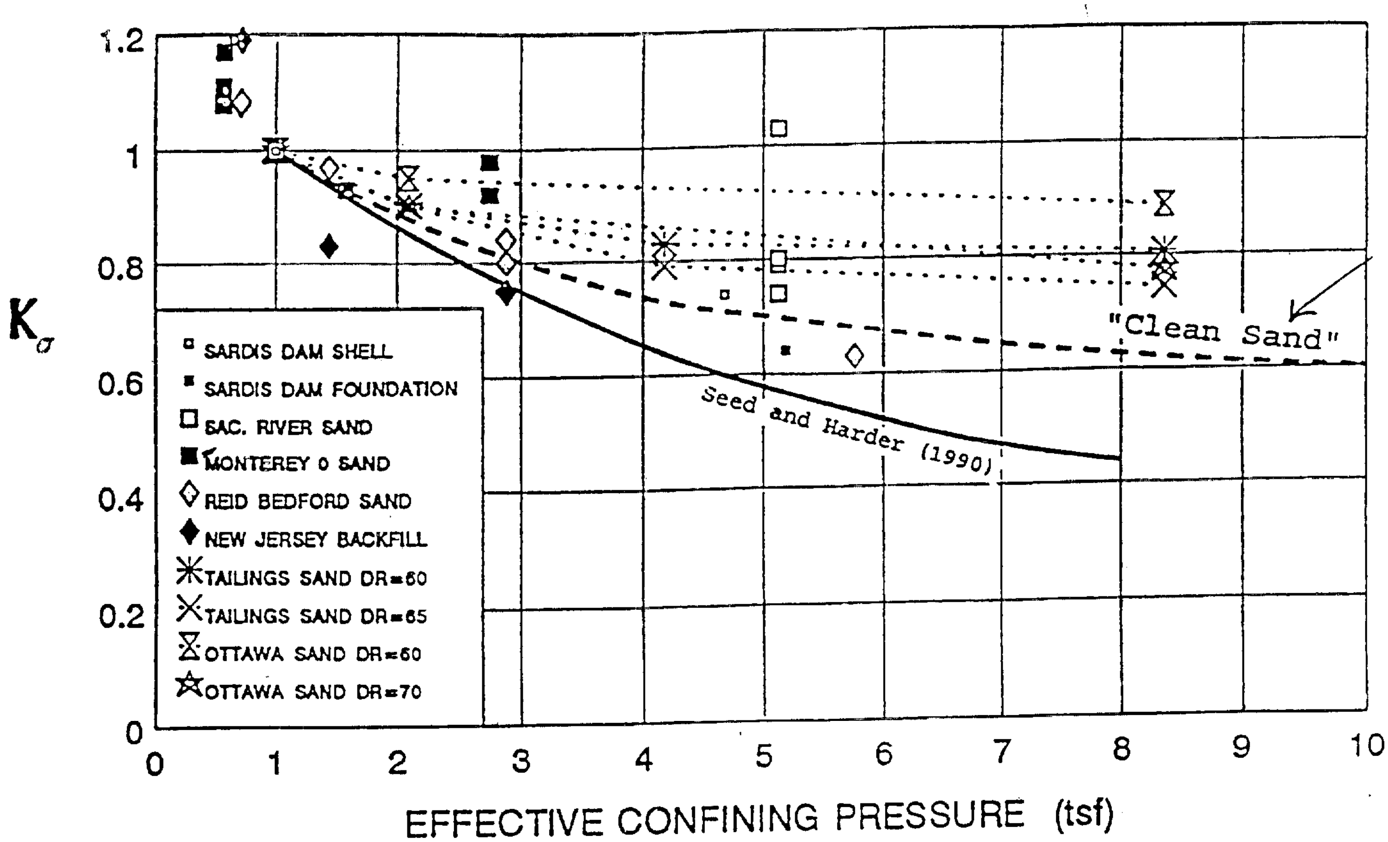

Figure 6 Recommended K_σ values (Harder 1996)

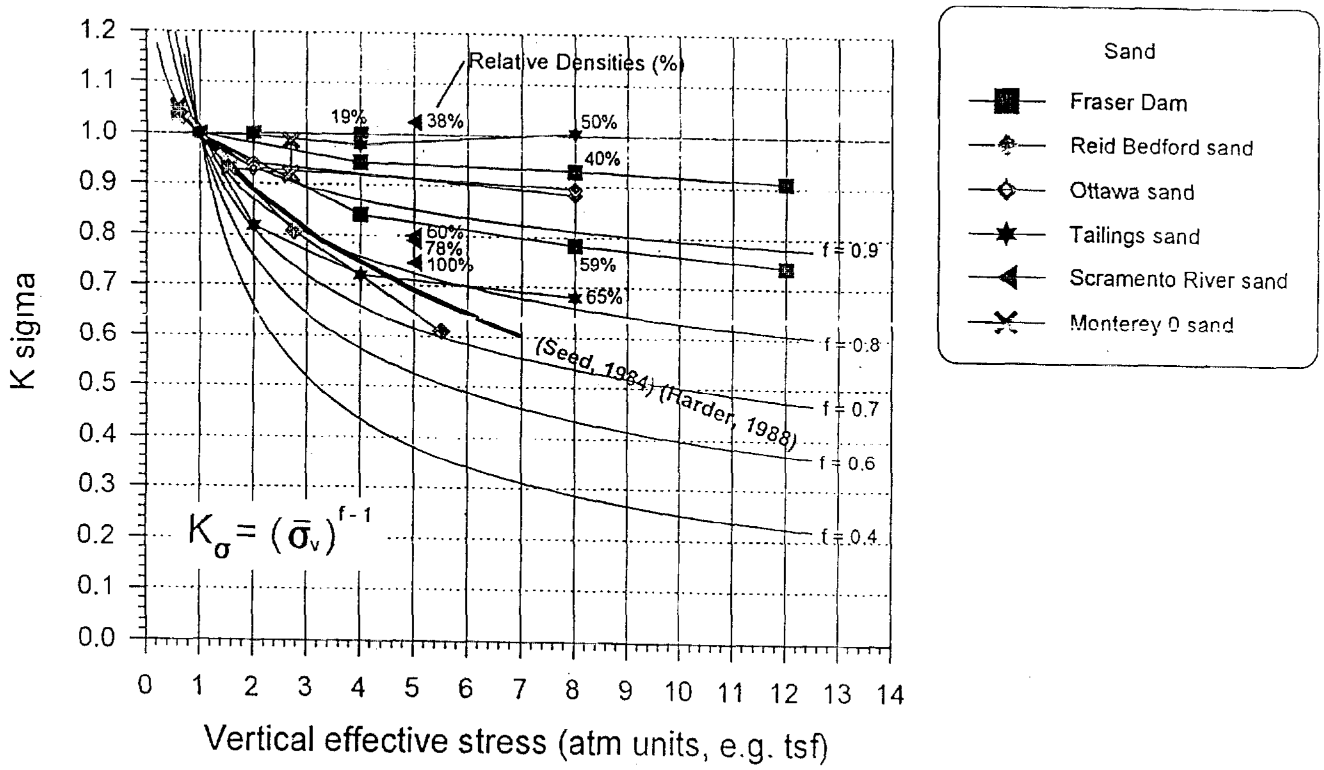

Figure 7 K_σ and CRR stress focus data for clean sands

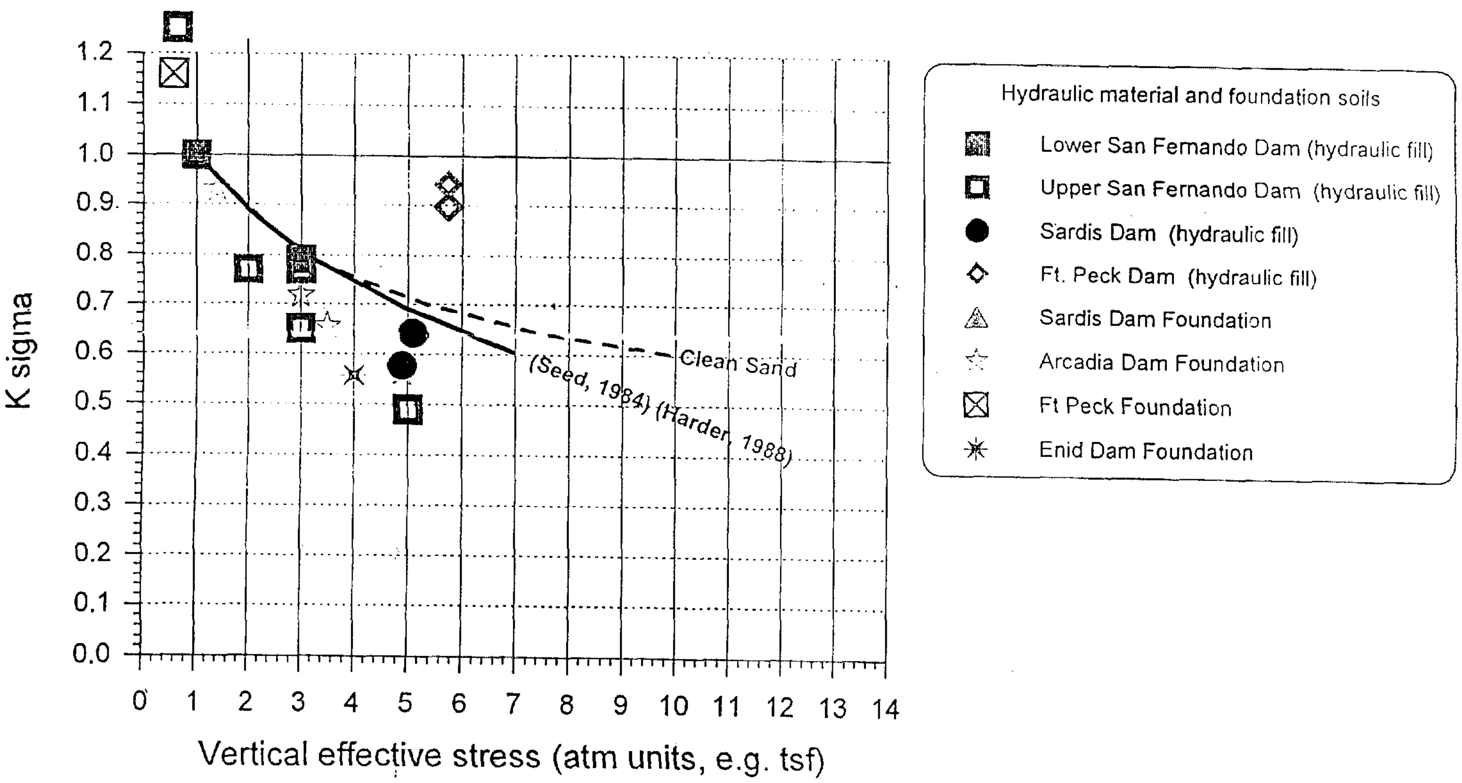

Figure 8 K_σ and CRR stress focus data for silty sands and hydraulic fills

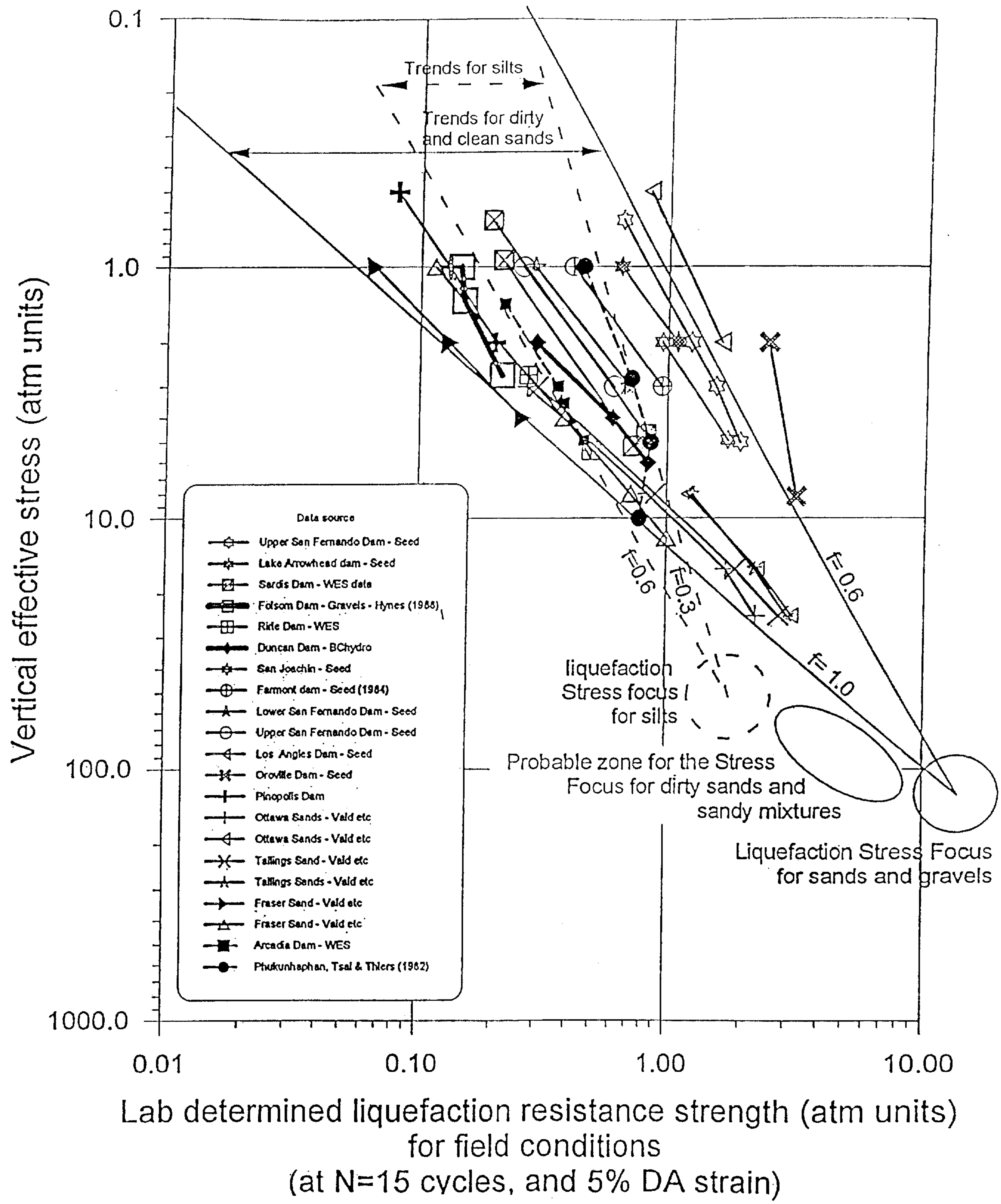

Figure 9 Trends of liquefaction resistance in stress focus format