INFLUENCE OF SAMPLING, TESTING, AND ANALYTICAL METHODS ON FACTORS OF SAFETY AGAINST SEISMIC LIQUEFACTION

by

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ABSTRACT

A wide range of factors of safety can be computed by using different methods to evaluate the seismic liquefaction potential of sandy soils. Empirical approaches utilizing standard penetration test results (N-values) and laboratory testing-oriented approaches using relatively undisturbed or reconstituted samples can yield different values of resistance to shaking. Computed values of seismically induced shear stress can also vary significantly depending on method of computation. Discussion of a case history in which several methods of sampling, testing, and analysis were performed with varying degrees of rigor illustrates the effects of the different techniques on computed factor of safety against seismic liquefaction.

INTRODUCTION

The site under evaluation was that of a small nuclear power plant constructed on relatively uniform, clean river sands. Approximately the upper 20 feet consisted of hydraulic fill, overlying 100 to 130 feet of glacial outwash and fluvial deposits over bedrock. The soils were generally in a loose to medium dense condition, and were classified as fine to medium sand with a trace of silt. The liquefaction potential of the site soils was initially investigated in 1973 as part of the application for an operating license for the plant. In 1980 additional investigation was undertaken as part of a regulatory review of the plant's operational status. During these investigations, soil samples were obtained and tested and analyses performed with varying degrees of sophistication. The 1973 and 1980 sampling and testing programs and subsequent analyses of safety against liquefaction are described in the following sections.

SAMPLING AND TESTING PROGRAMS

1973 Investigations--The field exploration program in 1973 consisted of drilling and sampling from six test borings, located near critical structures within an area about 350 by 230 feet. Samples were obtained at five-foot intervals from ground surface to bedrock at approximately 130 feet, by means of standard penetration tests (SPT), driven Dames & Moore Type U samplers, and hydraulically advanced Osterberg piston samplers. Reasonable care was taken to minimize disturbance to the soil samples, particularly the piston samples intended for strength testing. Volume measurements of the piston samples were made immediately, and weights were recorded later at a site laboratory for determination of wet and dry densities. SPT blow counts (N-values) recorded in the field are shown in Figure 1 for the depths of interest. Figure 2 shows measured dry densities as a function of depth.

A series of stress-controlled dynamic triaxial compression tests was performed on representative samples obtained from within the upper 40 feet of the deposit, which were considered liquefaction susceptible. Due to disturbance during sampling and transportation, samples to be tested were reconstituted to densities representing the range of densities measured in the field. Confining pressures for triaxial testing were selected to approximate in situ effective overburden pressures. Cyclic triaxial testing was then performed until liquefaction, which for these tests was defined to occur at the number of uniform stress cycles causing 10% double amplitude shear strain. Figure 3 shows a summary of the cyclic liquefaction test results on reconstituted samples.

1980 Investigations--Due to renewed concerns on the safety of existing nuclear power plants, a comprehensive study of liquefaction susceptibility was undertaken again in 1980. Effects of sample disturbance on test results and the need for more extensive, controlled penetration tests were considered in designing this sampling and testing program. The field investigation consisted of sampling from five borings located near critical structures. Three of the borings provided SPT blow counts at five-foot intervals throughout the depth of the holes. In one boring on each side of the reactor containment were taken relatively undisturbed Osterberg piston samples, to be used for density determinations and laboratory cyclic triaxial strength testing. The split-spoon samples from the SPT holes were used for field classification and laboratory confirmation of index properties.

Drilling was performed with extreme care in order to minimize possible sample disturbance. The drilling rig was maintained level at all times to ensure vertical drilling, and a side-discharge bit was used to minimize disturbance to soil below the bit. A thick drilling mud was maintained above the groundwater level in the hole to prevent caving or bottom blow-up. Upon retrieval of piston samples, measurements of volume, weight and moisture content were made immediately in a field laboratory for later density determinations. Samples were then drained, frozen, and transported to the testing laboratory in

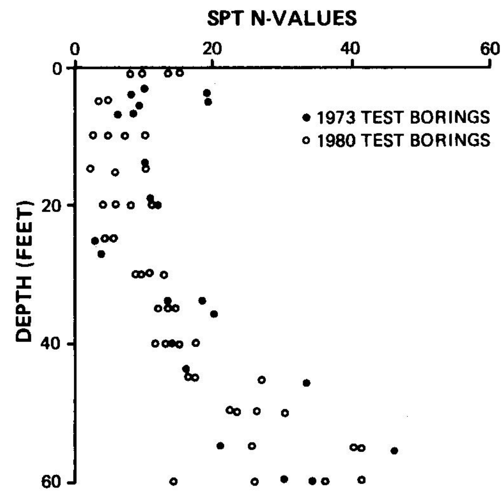

FIGURE 1
VARIATION OF SPT N-VALUES WITH DEPTH

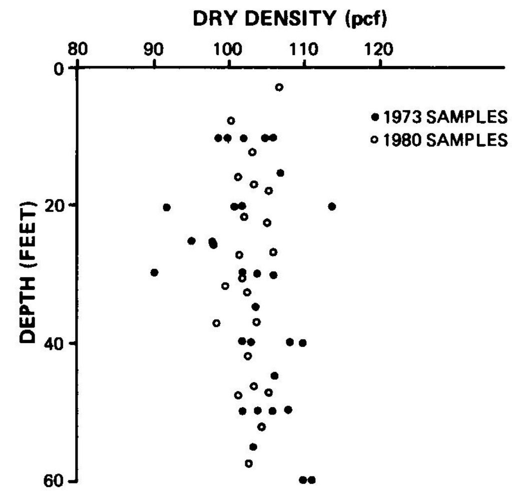

FIGURE 2
VARIATION OF DRY DENSITY WITH DEPTH

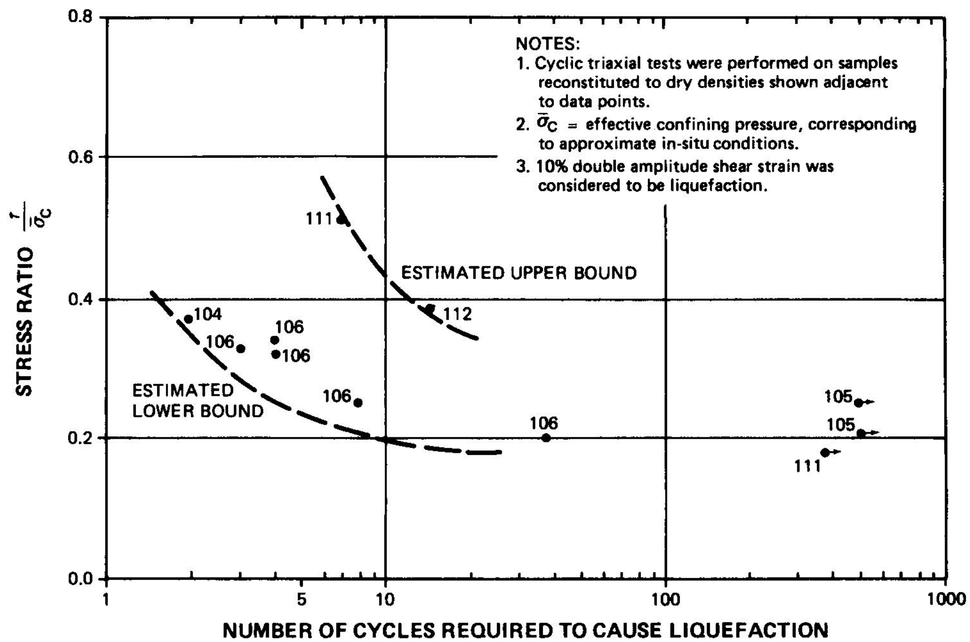

FIGURE 3
RESULTS OF 1973 CYCLIC TRIAXIAL TESTS

accordance with state-of-the-art sand sampling procedures (1). Further details of the sampling procedures followed are discussed in Reference 2.

Because research has shown that numerous variables in SPT technique can have significant effect on the resultant blow counts, every attempt was made to perform the tests in accordance with ASTM specifications, keeping in mind standard industry practice on which the empirical methods of liquefaction analysis are based. This adherence to standard practice included use of a flexible pull rope wrapped twice around the cathead, as well as calibration of the specified 140-lb hammer and 30-inch drop. Figure 1 shows variation with depth of the SPT-N values thus obtained.

In the laboratory dry densities were determined by drying portions of the frozen and thawed samples. Figure 2 shows measured dry densities as a function of depth. Cyclic shear strength tests were performed on the thawed samples, which were consolidated under confining pressures approximating in situ conditions. As in the 1973 testing, a 10% double amplitude shear strain criterion was used to define liquefaction. A summary of all cyclic liquefaction test results from the relatively undisturbed samples is presented in Figure 4. Further details of test procedures, effects of freezing and thawing of the samples, and test data are given in Reference 2.

LIQUEFACTION ANALYSES

Using the data obtained in the 1973 and 1980 investigations, two basic approaches were taken to evaluate the liquefaction potential of the saturated sand subjected to earthquake loading. In the first approach the cyclic shear stresses required to cause liquefaction (cyclic shear strengths) at various depths, were determined from the laboratory test data. Appropriate correction factors were used to convert the triaxial test data to field conditions, which more nearly correspond to simple shear conditions. The correction factor used in both the 1973 and 1980 analyses was 0.57 (3). The stress conditions anticipated in the field due to the design earthquake were evaluated by either a one-dimensional wave propagation analysis or by the Seed and Idriss simplified procedure (4). At a given depth, a factor of safety against liquefaction was calculated by dividing the cyclic shear stress required to cause liquefaction by the cyclic shear stress induced during the design earthquake.

The second approach, essentially an empirical one (3), used information available on the performance of various sand deposits during past earthquakes. The soil strengths were characterized by modified N-values which were then compared to the strengths of sand deposits known to have either liquefied or not liquefied under past earthquakes. Cyclic shear stresses during shaking were computed again by one-dimensional wave propagation analysis and by the simplified procedure. Liquefaction potential was assessed using empirical curves developed by

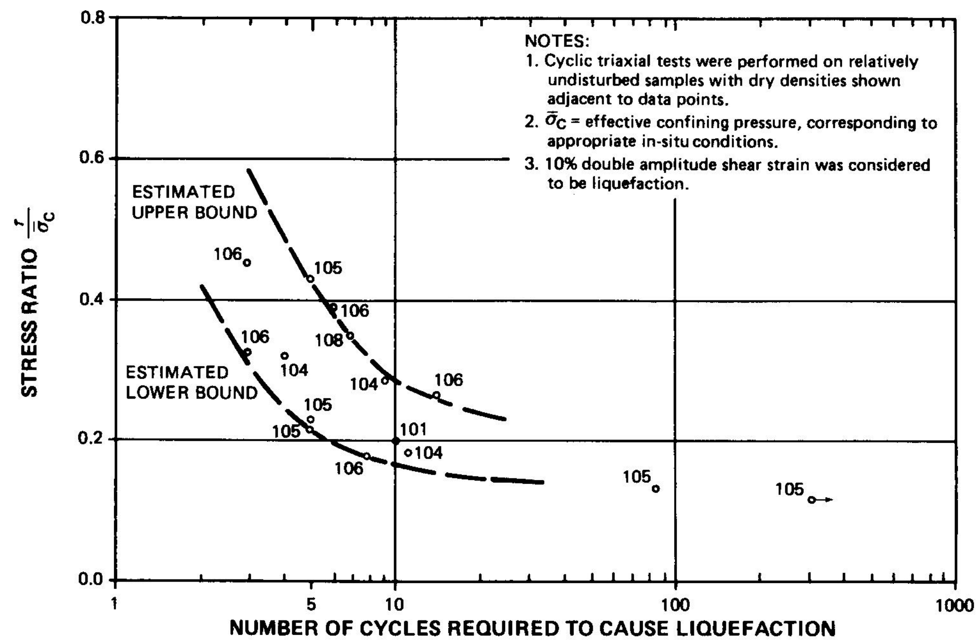

FIGURE 4
RESULTS OF 1980 CYCLIC TRIAXIAL TESTS

Seed. Although computation of numerical factors of safety was not the objective of this procedure, factors of safety can nevertheless be computed as the ratio of cyclic shear strengths based on modified blow counts to induced cyclic shear stresses.

DISCUSSION

The computed factor of safety for a given depth varied significantly, depending on design assumptions, data interpretation, and the methods selected for analysis. For purposes of this discussion, the effects of other uncertainties such as seismic design parameters and water table fluctuation were not considered, and the following parameters were assumed:

Design earthquake magnitude: 5.6
Strong motion duration of design earthquake: 15 seconds
Maximum horizontal ground surface acceleration: 0.12g
Number of equivalent uniform cycles: 5
Depth to water table: 10 feet.

Factors of safety computed by comparing four different evaluations of soil strength to two different calculations of seismically-induced stresses are shown in Table 1. For illustration purposes, conditions were evaluated at depths of 20 and 40 feet below ground surface. Using these methods of stress and strength determination, a range in factor of safety of 0.7 to greater than 3.9 was calculated for the soil at a 20-foot depth, and a range of 1.2 to 3.0 for the soil at a 40-foot depth.

The factor of safety was influenced by several factors affecting strength estimates and stress computations. In the approaches utilizing 1973 and 1980 blow count data to evaluate soil strength, the range in safety factor reflects the range in SPT N-values obtained at the site for that depth. Although a common approach might be to use a mean of N-values at a given depth as the design value, a very conservative approach might make use of the lowest N-value as representing a possible worst case strength evaluation. The use of the highest N-value, which may not be acceptable in a conservative approach, still represents a set of actual in-situ conditions.

Strength calculations based on 1973 cyclic triaxial testing indicate the influence of dry density of the tested samples on the range of stress ratios obtained. Prior to testing, the samples were reconstituted to dry densities representing the approximate range of densities measured at the site. The lower end of the range of densities is estimated by the lower bound curve in Figure 3, as samples reconstituted to less than about 104 pcf were too loose for testing. Using these data, a safety factor might be calculated by selecting a mean density for a given depth and interpolating a stress ratio from the test curves, or by more conservatively selecting the lower bound of stress ratio for a specified number of equivalent cycles. The lower bound approach might also be selected if there is evidence to believe that some densification has occurred during sampling, so that measured densities may be somewhat higher than the in situ values.

A narrower range of in situ dry densities was observed during the 1980 investigation, presumably due to the use of sampling and transportation methods that were believed to minimize disturbance. The variation in stress ratio resulting from tests on relatively undisturbed samples reflects less apparent dependence on density than in the 1973 tests, and more apparent influence from variation in soil fabric and confining pressure with depth. This is borne out by the observation that most of the upper bound data points in Figure 5 represent deeper samples, tested at higher confining pressures, while lower bound data points generally represent shallower samples (hydraulic fill) tested at lower confining pressures. While the number of tests performed is not adequate to make generalized conclusions, a significant difference in behavior of hydraulic fill and deeper soil samples is evident from the strength test data as well as from the SPT N-values. Whether a mean value, lower bound or upper bound stress ratio should be used in the analysis depends on the degree of conservatism required.

SUMMARY AND CONCLUSIONS

From this example, it is apparent that the resultant value of safety factor against liquefaction can vary widely depending on quantity, type and quality of strength data available, and the interpretation of that data to best represent in situ conditions. For the site conditions analyzed, the induced stress computed by one-dimensional wave propagation differed from that calculated by the simplified procedure, but only enough to slightly increase the safety factor. However, the shear stress computed from the one-dimensional analysis can itself vary, depending on input parameters such as shear moduli and damping coefficients and their variations with strain levels. The variability in each type of strength data generally had greater effect on the safety factor, depending on the extent of the variability and whether mean, lower bound or upper bound values were selected.

Evaluation of such data inevitably raises a question as to what degree of conservation is appropriate. Throughout the liquefaction analysis the engineer must exercise judgement concerning interpretation of soil data and selection of analytical technique to best represent the existing and potential conditions at the site. Interpretation of field and laboratory data can be particularly difficult at a site with loose clean sands, where the characteristics which make the deposit potentially susceptible to liquefaction also greatly increase the difficulty of obtaining relatively undisturbed samples. Potential for changed conditions at a site must also be considered, such as the presence of driven piles under a critical structure, which could increase soil densities and thereby provide additional safety against seismic liquefaction. A method of predicting increase N-values due to displacement piles can be found in Reference 5. The increased strength due to increased N-values can be accounted for in computing the factor of safety.

As in all engineering analyses, the consequences of potential failure must be taken into account in estimating safety factors. The most difficult question to answer therefore concerns what an acceptable factor of safety is. When human health and long-term safety are involved, new dimension is added to this question. Deterministic analysis, coupled with a probabilistic assessment and a clear understanding of the limitations of testing and analytical techniques and the reliability of field data, will indicate the real "factor of safety."

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TABLE 1

Calculated Factors of Safety

Methods of determining cycling shear stresses \ Methods of determining cyclic shear strengths	Depth (ft)	1973 SPT N-values	1980 SPT N-values	1973 cyclic triaxial tests on reconstituted samples	1980 cyclic triaxial tests on undisturbed samples
Simplified method	20	1.9-2.0	0.7-1.9	1.5-3.6[+]	1.3-2.6
	40	1.6	1.2-1.9	1.4-2.7	1.2-2.3
One-dimensional wave propagation analysis	20	2.0-2.1	0.8-2.0	1.6-3.9[+]	1.4-2.8
	40	1.8	1.3-2.1	1.6-3.0	1.3-2.6

Notes:

1. Factors of safety against liquefaction are calculated by dividing cyclic shear strengths by cyclic shear stresses.
2. Upper bound strength at 20 feet from 1973 lab tests is extrapolated to exceed the limits of available data in Figure 3.