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PROCEDURES AND CRITERIA FOR ASSESSING SEISMIC SOIL LIQUEFACTION AT NUCLEAR POWER PLANT SITES

A. INTRODUCTION

This regulatory guide has been developed to provide guidance to license applicants on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation. It discusses conditions under which the potential for such response should be addressed in safety analysis reports. The guidance includes procedures and criteria currently applied to assess the liquefaction potential of soils ranging from gravel to clays.

In 10 CFR Part 100, "Reactor Site Criteria," § 100.23, "Geologic and Seismic Siting Criteria," sets forth the principal geologic and seismic considerations that guide the NRC in its evaluation of the suitability of a proposed site. In addition, 10 CFR 100.23(d)(4) discusses several siting factors that must be evaluated and requires that the potential for soil liquefaction be evaluated in addition to several other geologic and seismic factors.

Safety-related site characteristics, including those related to the response of soils to earthquakes, are identified in Regulatory Guide 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)." Regulatory Guide 4.7, "General Site Suitability Criteria for Nuclear Power Stations," discusses major site characteristics that affect site suitability. Procedures and methods of site investigations are described in Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants." Guidelines for laboratory testing are given in Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants."

Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," defines investigations related to seismicity, faults, and vibratory ground motion.

The scope of this guide is limited to evaluation of the behavior of soils subjected to earthquake shaking. It specifically excludes nonseismic failure of sensitive clays, failure under static loads (such as flow slides in loose point bar deposits), and soil response to machine vibrations and blasting. The selection or synthesis of appropriate ground motion records to use for a response analysis is not included.

The technical basis for this regulatory guide is contained in NUREG/CR-5741 (1999). NUREG/CR-5741 was developed to compile current and state-of-the-art techniques for evaluating earthquake-induced liquefaction. It summarizes the process of acquiring and using geological, geophysical, geotechnical, and other kinds of relevant information that support design considerations with respect to the liquefaction potential of soils. A report from the 1996 NCEER and 1998 NCEER/NSF workshops summarized evaluation procedures for liquefaction susceptibility in the past 30 years and recommended state-of-the-art evaluation practices (Youd et al., 2001).

The information collections contained in this draft regulatory guide are covered by the requirements of 10 CFR Parts 50 and 100, which were approved by the Office of Management and Budget (OMB), approval numbers 3150-0011 and 3150-0093. The NRC may not conduct or sponsor, and a person is not required to respond to, a request for information or an information collection requirement unless the requesting document displays a currently valid OMB control number.

B. DISCUSSION

GENERAL

Every site and plant facility is unique, and therefore requirements for analysis and investigations vary. It is not possible to provide evaluation procedures for seismic soil liquefaction potential to deal with every contingency. In circumstances that are not specifically addressed in this guide, prudent and sound engineering judgment should be exercised, with due regard to the degree of conservatism needed to provide reasonable assurance of the adequacy of the design and construction of nuclear facilities.

In the present state of the art, the interpretation of available geotechnical data and results of analyses of soil response to seismic loading are tempered by (1) documented experience and empirical knowledge of conditions under which seismically induced ground failure has or has not occurred in the past, and (2) application of a degree of conservatism that will compensate for uncertainties and thus assure the safety of the facility under seismic loading. With advances in the state of the art, improvements in methodology will be adopted to minimize the uncertainties.

The mechanisms of liquefaction and the early development of analysis techniques for seismic response of saturated cohesionless soil deposits and the physical mechanisms of liquefaction are discussed by Casagrande (1936); Shannon & Wilson (1972); Castro (1975); and Finn and Martin (1975). Empirical techniques based on field performance data were subsequently developed and promoted. These include publications by Seed (1976, 1979a, 1979b), Finn (1981), and Seed and Idriss (1971,1982). Hynes (1988) extended the state of knowledge on liquefaction mechanisms with regard to large-particle sized soils; Kaufman (1981), Puri (1984), Walker and Stewart (1989), Koester (1992), and others examined mechanisms of earthquake-induced liquefaction of sands containing fines and both plastic and nonplastic mixtures of silt and clay.

DEFINITIONS

During an earthquake, soils may undergo either transient or permanent reduction in undrained shear resistance as a consequence of excess pore water pressures or disruption of the soil structure accompanying cyclic loading. Such strength degradation may range from slight diminution of shear resistance to the catastrophic and extreme case of seismically induced liquefaction, which is a transient phenomenon.

The word "liquefaction" means a change in state from a solid to liquid. As applied to a soil, the term refers to a change from a solid or stable assemblage of soil particles to a complete or substantially complete suspension of the solid particles in a fluid, such that the suspension has a very low shear strength. Some practitioners restrict the use of the term liquefaction to describe flow failure, as is observed to occur in a failed slope when driving shear stresses caused by an earthquake remain higher than the post-failure shear strength of the soil materials (Castro and Poulos, 1977). Others use the term initial liquefaction to describe the buildup of pore water pressure in laboratory tests within an undrained soil specimen to a level equal to the total confining stress. Liquefaction-induced ground failure, in the extreme sense of surface manifestation, may take the form of flow failure, lateral spread as mentioned, and ground oscillation (Youd, 1993).

In this guide, the term seismically induced liquefaction includes any drastic loss of undrained shear resistance (stiffness and/or strength) resulting from repeated rapid straining, regardless of the state of stress prior to loading. The term is interchangeably applied to the development of either excessive cyclic strains or complete loss of effective stress within an undrained laboratory specimen under cyclic loading (sometimes referred to as initial liquefaction).

SITE CHARACTERIZATION

Preliminary assessments are to be made to determine whether the site is clearly likely or not likely to liquefy in response to earthquake shaking. The following information is essential to an initial assessment of the potential for earthquake-induced ground failure.

1. Geomorphology of the site.
2. A soil profile, including classification of soil properties and the origin of soils at the site, and a three-dimensional subsurface soil stratigraphy should be developed to support the site conceptual model.

3. Water-level records, representative of both current and historical fluctuations.
4. Evidence obtained from historical records, aerial photographs, or previous investigations of past ground failure at the site or at similar (geologically and seismologically) nearby areas (including historical records of liquefaction, topographical evidence of landslides, sand boils, effects of ground instability on trees and other vegetation, subsidence, and sand intrusions in the subsurface).
5. Seismic history of the site.

Significant steps in the site investigations, as related to the evaluation of the potential for seismically induced ground failure, occur in both the initial exploration phase and the detailed exploration phase. The initial phase should include borings with Standard Penetration tests (SPT) or Cone Penetration Tests (CPT) complemented by SPT disturbed sampling for measuring penetration resistance and for obtaining samples for soil classification and water content determination.

Although the SPT is the routine method of determining penetration resistance for the purpose of evaluating the potential for seismically induced ground failure, because of the available data base for correlating SPT values with seismic response, CPT soundings are increasingly used since they provide better stratigraphic detail for penetration resistance and soil characteristics and their results are more repeatable and consistent. CPT results should be verified using published (e.g., Robertson and Wride, 1998) and site-determined correlation with SPT values and relevant soil parameters.

For the in situ investigation of the strength of soils containing gravels or cobbles and their potential for liquefaction, the tool of choice should be the Becker Hammer Tool or shear wave velocity measurement. Liquefaction resistance and residual strength of gravelly soils should be estimated by converting Becker Hammer Penetration Test (BPT) blow counts to equivalent SPT blow counts (e.g., Harder and Seed, 1986). Recommendations with respect to BPT and shear wave velocity measurement performance and result interpretation are discussed in Youd et al., 2001.

During the initial field investigations, the following conditions would be cause to suspect a potential for seismically induced ground failure: (1) low penetration resistance as measured by SPTs or CPTs in sands and finer grained soils, BPTs in gravels or cobbles, low shear wave velocities in various soil deposits, (2) artesian head conditions or excess pore pressures, (3) persistent inability to retain soil samples in conventional sampling devices, (4) saturated zones of granular soils with impeded drainage, and (5) the presence of any clean, fine sand below the ground-water table. Any of these occurrences should be noted on the boring log and described in the report of the site investigations. If deposits are identified that show a potential for seismically induced ground failure that would affect the safety of the facility, they should be explored in detail to define their thickness and areal extent, and further investigations should be conducted to define the threat that those deposits may pose to the facility.

The detailed exploration phase following the initial phase of subsurface investigations should include surveys and undisturbed sampling borings to (1) refine the preliminary interpretation of the stratigraphy and the extent of potentially liquefiable soils, (2) measure in situ densities and dynamic properties for input to dynamic response analyses, and (3) recover undisturbed samples for laboratory testing when site soils are not adequately represented in the available data base.

SCREENING TECHNIQUES FOR EVALUATION OF LIQUEFACTION POTENTIAL

On the basis of the information obtained during the initial site investigations, an early evaluation of the potential for seismically induced ground failure can be made in some cases. The goal of these evaluations is to determine whether the site is clearly safe or whether soils clearly will liquefy. If the results of the screening evaluation are unclear, however, more detailed analysis should be conducted. The following paragraphs discuss some screening techniques for evaluation of liquefaction potential.

Earthquake-induced liquefaction is most commonly observed in (but not restricted to) the following types of soils: (1) fluvial-alluvial deposits, (2) eolian sands and silts, (3) beach sands, (4) reclaimed land, and (5) uncompacted hydraulic fills.

Cohesive soils with fines content greater than 30 percent and fines that either (1) are classified as clays based on the Unified Soil Classification system or (2) have a Plasticity Index (PI) greater than 30 percent should generally not be considered susceptible to liquefaction.

Sands that have dual Unified Soil Classification system designations such as CL-ML, SM-SC, or GM-GC are potentially liquefiable (Youd, 1998). Other designations involving the "C" description, if the clay content is greater than 15 percent by weight and the liquid limit is greater than 35 percent and occurs at natural water contents lower than 90 percent (Wang, 1979), can be considered nonliquefiable.

Some gravelly soils are potentially vulnerable to liquefaction (Coulter and Migliaccio, 1986; Ishihara, 1984; Harder, 1988; Andrus and Youd, 1987; Andrus and Youd, 1989; Andrus et al., 1992) when (1) their voids are filled with finer particles or (2) they are surrounded by less porous soils where their drainage is impeded and they may be vulnerable to cyclic pore pressure generation or liquefaction or both.

Liquefaction resistance can be roughly correlated with geologic age, depositional environment, and prior seismic history (Youd, 1998, after Youd and Perkins, 1978). Most liquefaction risk is associated with recent Holocene deposits and uncompacted fills. There have, however, been a few observed cases of liquefaction of Pleistocene and even Pre-Pleistocene deposits. Particular caution should be used in dealing with very loose types of these soils (e.g., dune sands, talus) and with extremely loose collapsible soils (e.g., loess).

If it can be demonstrated that any potentially liquefiable soil types present at a site (1) are currently unsaturated (e.g., are above the water table), (2) have not been saturated previously (e.g., are above the historic high water table), and (3) cannot reasonably be expected to become saturated, such soils can be considered to pose no potential liquefaction hazard. Youd (1998) has summarized historical data that relate water table depth to liquefaction susceptibility.

Potentially liquefiable soils may not pose a liquefaction risk to the facility if they are insufficiently thick and of limited lateral extent. If, however, they are very loose and continue laterally over a sufficient area, they can represent hazardous planes of weakness and sliding and may thus pose other hazards with respect to translational site instability, lateral spreading, or related ground displacement. During an earthquake, a loss of bearing capacity may occur beneath shallow foundations of structures supported on relatively stable strata above the liquefiable soils. The reduction in the lateral capacity and the addition in the lateral loads to the deep foundations may also occur if the soils surrounding deep foundations liquefy (Martin and Lew, 1999). When suitably sound lateral containment is provided to eliminate potential sliding on liquefied layers, potentially liquefiable zones of finite thickness occurring at any depth may be deemed to pose no significant sliding risk but may, however, be susceptible to differential settlement and ground oscillation. Where considerations of such differential settlement or relative displacement between structures or components of the facility have safety significance, quantitative analyses of the settlement and the deformations, including effects of earthquakes, will be required even though the soils involved may be judged seismically stable to liquefaction effects.

Liquefaction hazard maps from probabilistic methodologies can sometimes be used as a screening technique to identify potentially liquefiable soils (e.g., NUREG/CR-6622, 1999).

PROCEDURES FOR EVALUATING LIQUEFACTION POTENTIAL

General

If the geologic site evaluation indicates the presence of potentially liquefiable soils, the resistance of these soils to liquefaction or significant strength loss to cyclic pore pressure generation should be evaluated.

Factor of Safety Against Liquefaction

Liquefaction susceptibility can be expressed in terms of a factor of safety against the occurrence of liquefaction as:

$$FS_{againstliquefaction} = FS = \frac{CRR}{CSR}$$

where CRR (cyclic resistance ratio) is the available soil resistance to liquefaction, expressed in terms of the cyclic stresses required to cause liquefaction, and CSR (cyclic stress ratio) is the cyclic stress generated by the design earthquake.

The above definition of the factor of safety is used in empirical procedures for the evaluation of liquefaction potential. Interpretations of the factor of safety, FS, are provided by Marcuson, Hynes, and Franklin (1990) for level ground conditions, Tokimatsu and Yoshimi (1983) for sandy soils, Evans (1987), and Hynes (1988) for gravelly soils. A series of scaling factors to calculate the factor of safety against liquefaction were provided in Youd et al., 2001.

Analytical Methods

These methods typically rely on laboratory tests to determine either liquefaction resistance or soil properties that can be used to predict the development of liquefaction. Various equivalent linear and non-linear computer methods are used with the laboratory data to evaluate the potential for liquefaction. Analytical methods need accurate measurements of constitutive soil properties. If reliable screening procedures have not ruled out the possibility of liquefaction, a comprehensive laboratory testing program should be conducted. Guidelines for laboratory testing are described in Regulatory Guide 1.138, “Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants.”

Laboratory Cyclic Strength Testing

The cyclic tests used to evaluate the response of soils to earthquake shaking should correctly simulate the loading to which the soil would be subjected in situ. Cyclic simple shear tests may best reproduce the straining in a soil specimen caused by upwardly propagating earthquake waves. Various configurations of cyclic simple shear, cyclic triaxial, large-scale shake table, and cyclic torsional shear apparatuses have been employed to study liquefaction resistance (Woods, 1981; Wood, 1982; and Department of the Army, 1986).

Research studies have demonstrated that laboratory-determined cyclic triaxial strengths (in fact, strengths determined from any unidirectional loading test) are higher than those expected to produce equivalent effects in the field (Seed, 1976). Research has also shown that estimation of field liquefaction resistance from laboratory test results may not be possible by universal application of simple factors, e.g., gradation, density, and soil type (Koester, 1992).

Physical Modeling

In physical modeling by the use of a centrifuge, a small-scale physical model of a soil deposit is subjected to an increased acceleration field such that the stress level caused by self weight in the model would be the same as the corresponding stress level in the prototype. Numerous studies by many researchers have demonstrated the benefits of centrifuge modeling for seismic simulation studies and liquefaction potential evaluation (Whitman, Lambe, and Kutter, 1981; Schofield, 1981; Scott, 1983; Arulanandan, Anandarajah, and Abghari, 1983; Steedman, 1984; Coe, Prevost, and Scanlan, 1985; Hushmand, Scott, and Crouse, 1988; Ketchman, Ko, and Sture, 1991; and Arulanandan and Scott, 1993).

Empirical Procedures

Empirical methods have become widely used in routine engineering practice. Procedures for carrying out a liquefaction assessment using empirical methods are in Youd et al. (2001); Seed and Idriss (1971); Seed (1983); National Research Council (1985); and BSSC (1991).

Recommended relationships between corrected SPT penetration resistance and the liquefaction resistance CRR (normalized for overburden stress) are discussed in Youd et al.

(2001). The measured SPT blow count is modified for various parameters to yield the corrected SPT penetration resistance. Modifications for CRR include those for earthquake magnitudes greater than 7.5 and other parameters (Youd et al., 2001).

CPT-derived estimations of CRR follow procedures discussed in Robertson and Wride, (1998) and recommended by NCEER Workshop (Youd et al., 2001). BPT blowcounts are converted to equivalent corrected SPT values (Harder and Seed, 1986), which in turn are used to estimate CRR. By comparing the calculated equivalent, uniform, earthquake-induced cyclic stress ratio, CSR, with the uniform cyclic stress ratio necessary to fully trigger liquefaction, CRR, the factor of safety against "triggering" of liquefaction can be calculated.

A simplified procedure using shear wave velocity (Vs) as an index to assess the liquefaction potential by Andrus and Stokoe (2000) was recommended by the NCEER Workshop (Youd et al., 2001). Shear wave velocity and CRR are directly influenced by void ratio, effective confining pressures, stress history, and ages of the deposits. The relationship between overburden stress corrected shear wave velocity and CRR was also provided in Andrus and Stokoe (2000).

The following table lists the applicability of various field tests for assessment of liquefaction potential (modified after Youd et al., 2001).

	Test Type			
Feature	SPT	CPT	BPT	Vs
Past measurements at liquefaction sites	Abundant	Abundant	Sparse	Limited
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Partially drained, large strain	Small strain
Quality control and repeatability	Poor to good	Very good	Poor	Good
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	Primarily gravel	All
Soil sample retrieved	Yes	No	No	No

C. REGULATORY POSITION

1. SITE CHARACTERIZATION

1.1 General

The initial phase of the site characterization program should include borings with their locations and depths chosen so that the site geology and foundation conditions are sufficiently defined in lateral extent and depth to provide the data for the designs for all structures and excavations. This phase should include borings with SPT tests or CPT tests for determining penetration resistance and soil characteristics for measuring classification and water content determinations.

CPT should be the tool of choice for initial site characterization studies in support of liquefaction potential assessment. The CPT results should be used to select localities and depths for subsequent SPT borings or boreholes and other sampling efforts. Coverage of the site with CPT and SPT borings should be adequate to (1) establish general soil conditions, distribution of soil types, homogeneity, and ground-water elevation; (2) identify soils that might liquefy; and (3) assist in specifying the locations of additional borings and geophysical surveys aimed at detailed seismic response evaluation.

For the in situ investigation of the strength of soils containing gravels or cobbles and their potential for liquefaction, the tool of choice is the Becker Hammer Tool or shear wave velocity measurement.

1.2 Performance of Standard Penetration Tests

The procedures and apparatus used in performing SPTs should conform to applicable published industry consensus standards and should be described in sufficient detail to permit the evaluation of the results by the NRC staff. Because of the importance of the details of field procedure, field tests made for the evaluation of seismic stability of soils should be performed under a quality assurance program, in accordance with the requirements of Appendix B, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants,” to 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities.”

1.2.1 Adjustment of "N" Values

Procedures used for adjusting or correcting "N" values from SPTs for various parameters for use in the evaluation of liquefaction potential must be discussed and justified. Adjustments for the “N” values are discussed in the literature in Youd et al. (2001).

1.2.2 SPT Correlations

In evaluating the seismic stability of site soils on the basis of correlation of SPT data, where correlations or comparisons are made with other than raw measured data (i.e., with parameters such as relative density or overconsolidation ratio derived or computed from the "N" values), the method of derivation should be described in sufficient detail that the staff can retrace the derivation. When derived parameters are used in correlations, the "N" values should also be provided.

1.3 Performance of Cone Penetration Tests

For CPT-based evaluations, sufficient soil sampling using boreholes or the SPT adjacent to CPT soundings should be performed to obtain soil samples for confirmation of soil type and for soil index testing. CPT soil characterization charts (e.g., Robertson and Wride, 1998) for estimating liquefaction resistance should be used in conjunction with laboratory-measured soil index tests.

1.4 Performance of Becker Hammer Penetration Tests

Uncertainties should be minimized in the in situ testing of gravelly soils using BPT tests. Recommendations, such as in Youd et al. (2001), should be followed closely with respect to BPT performance and results interpretation.

1.5 Seismic Wave Velocity Measurements

The shear wave velocity method cannot detect a thin liquefiable layer if measuring intervals are too large; it also does not provide samples for soil classification. Sufficient boreholes should be drilled and sufficient in situ tests should be conducted to detect and delineate thin liquefiable layers, nonliquefiable clay-rich soils, and silty soils above the groundwater table that might become liquefiable should the groundwater table rise. Weakly cemented soils that might have high Vs value can also be detected by CPT and SPT methods (Youd et al., 2001).

2. SCREENING TECHNIQUES FOR EVALUATING LIQUEFACTION POTENTIAL

Based on the initial site characterization, an early evaluation of the potential for seismically induced ground failure should be made. The liquefaction hazard evaluation should address three basic questions: (1) Are potentially liquefiable soils present? (2) If so, are they saturated or could they become saturated at some future date? and (3) If so, are they of sufficient thickness or lateral extent to pose a risk to the survival or function of the project? These evaluations should make assessments as to whether the site is clearly safe or if soils clearly will liquefy. If the results of the screening evaluation are unclear, more detailed evaluations should be conducted.

3. PROCEDURES FOR EVALUATING LIQUEFACTION POTENTIAL

3.1 General

If evaluations of the site investigations indicate the presence of potentially liquefiable soils, the resistance of these soils to liquefaction must be evaluated. It should also be determined whether the potentially liquefiable soils should be removed, whether remedial action should be undertaken, whether further field and laboratory investigations are needed, or whether detailed stability and deformation analysis could demonstrate that an acceptable margin of safety is maintained for the design structures even if liquefaction is assumed to occur.

3.2 Factor of Safety Against Liquefaction

The factor of safety, FS, is used in empirical procedures for the evaluation of liquefaction potential. Interpretations of the FS should be used with caution. In general:

1. Soil elements with low FSs ($FS \leq 1.1$) would achieve conditions wherein soil liquefaction should be considered to have been triggered. Conservative undrained residual strengths, S_r, from laboratory and field tests should be assigned to these zones for further stability and deformation analyses.

2. Soil elements with a high FS ($FS \geq 1.4$) would suffer relatively minor cyclic pore pressure generation and should be assigned some large fraction of their (drained) static strength, obtained from laboratory tests, for further stability and deformation analysis.

3. Soil elements with intermediate FSs ($FS \approx 1.1$ to 1.4) should be assigned strength values between the values appropriate to conditions 1 and 2 above for further stability and deformation analyses. In strongly contractive soils, the possibility of progressive failure or deformation should be considered and mobilization of undrained residual strengths should be assumed.

For computing the FS with respect to seismically induced ground failure, cyclic resistance corresponding to stress levels causing 5 percent peak-to-peak cyclic strain or 2-1/2 percent strain in compression, whichever occurs first, should be used. These criteria refer to data obtained in consolidated-undrained, stress-controlled, cyclic triaxial tests. When other methods of test are used, consistent criteria should be applied.

3.3 Analytical Methods

Analytical methods rely on accurate measurements of constitutive soil properties. Many of the material properties and other input parameters used in the analyses as discrete values are actually known only within some range of values. Some factors that should be addressed, related to input parameters, assumptions, and methods of analyses, are listed below.

3.3.1 Uncertainty in Geotechnical Input Parameters

The normal variability in soil and rock materials is such that geotechnical engineering parameters, such as soil types, layer thicknesses, and soil strengths, are usually known as ranges of values rather than as discrete values. To ensure that reliably conservative results are obtained from the analyses, values representing the conservative side of the range of an input parameter ordinarily are used. If a mean value is used, or if it is not known which side of the range is the more conservative, analyses should be performed to determine the sensitivity of the analysis to that variable.

3.3.2 Seismic Input Motions

Earthquake magnitude and duration, as well as the epicentral distance, are the primary factors in determining liquefaction potential at a site. The character and form of the earthquake motion are significant to soil response analyses and directly affect the results. In dynamic analyses of soil or soil-structure systems, the earthquake normally must be specified as an acceleration versus time record, which is event-specific. Analyses should be performed using an ensemble of records such that as a whole they conservatively represent the frequencies, amplitudes, and duration of shaking that are critical to the foundations and earth structures of the facility. In an area that lacks seismic strong motion records, seismic wave attenuation relationships should be carefully chosen to reflect the regional attenuation difference between, for example, the western and eastern United States, in the process of calculating of peak ground acceleration. Non-linear soil responses should also be taken into consideration for seismic wave amplification.

3.3.3 Time Step in Digitization of Earthquake Record

Time intervals used in digitization of ground motion records should be small enough to adequately represent the highest frequencies that are significant to the analysis. In general, 200 samples per second is an appropriate sampling rate.

3.3.4 Ground-Water Level

For analysis, ground-water tables or pool elevations should be assumed to lie at the maximum or the minimum of the range that can be expected at the site of the facility, in the direction that is most critical to the analysis. In analysis of seismically induced ground failure, the higher water levels normally will be the most critical. Normal water levels should be chosen in accordance with Regulatory Guide 1.135, "Normal Water Level and Discharge at Nuclear Power Plants," and water levels governed by flood conditions should be chosen in accordance with Appendix A to Regulatory Guide 1.59, "'Design Basis Floods for Nuclear Power Plants."

In the absence of additional supporting data from the project site itself, extrapolation of data regarding water table elevations from adjacent sites will not, by itself, usually suffice to demonstrate the absence of liquefaction hazard, except in those cases in which a combination of uniformity of local geology and very low regional water tables permits very conservative assessment of water table depths. Preliminary geologic site investigations should also address the possibility of local water tables or locally saturated soil units at the site.

3.4 Laboratory Cyclic Strength Testing, Physical Modeling, and Use of Data

3.4.1 Effects of Sample Disturbance

In laboratory tests to determine the response of granular soils to cyclic stresses, the use of undisturbed samples of good quality is preferred to the use of reconstituted materials. However, research and experience has shown that the effects of disturbance during sampling include an increase in density in the case of loose sands, and a decrease in density in the case of dense sands. To minimize the changes of density and consequent degradation of strength in samples of relatively dense sands, it is advisable to use carefully controlled and executed field sampling methods in order to obtain samples of the best quality possible. A less desirable alternative is the use of test specimens that are reconstituted to their in situ densities. If this method is used, care must be taken to ensure that there is no mixing of materials from different strata. Marcuson and Franklin (1979) have reviewed techniques and apparatus commonly applied to sample granular soils.

3.4.2 Equivalence Between Earthquake Stress History and Periodic Stress History in the Laboratory

The stress excitation produced at the site by an earthquake resembles in many respects a random motion, and dynamic stress analyses are performed with excitation inputs resembling as nearly as possible the expected earthquake motions at the site. Laboratory cyclic load testing is conventionally performed using uniform periodic load cycling. To make a valid comparison between stresses in the field and soil response in tests with uniform periodic loads, it is necessary to develop an equivalence between the quasi-random stress record obtained from a dynamic analysis and some number of uniform load cycles at some load or stress level. The equivalence is material-dependent and should be computed on a case-by-case basis using the results of cyclic material response tests having appropriate ranges of cyclic stress and number of applied load cycles. The ranges in these test parameters should cover the ranges of interest in the field so that extrapolation is not required.

3.4.3 Overconsolidated Soils

Any adjustment of laboratory-derived cyclic strength values to account for overconsolidation or lateral earth pressure conditions should be adequately supported by appropriate field and laboratory investigations to demonstrate the existence of those conditions.

3.4.4 Application of Cyclic Triaxial Test Data

The cyclic triaxial test does not accurately model the stress conditions in situ. Caution should be exercised when using laboratory-obtained soil cyclic strengths. There should be appropriate downward adjustments of cyclic stress values obtained from triaxial tests as appropriate. The rationale behind the adjustment and the data supporting its magnitude should be presented and referenced. Laboratory cyclic tests should be used only to establish parametric effects on cyclic strength behavior.

3.4.5 Physical Modeling

Centrifuge model testing permits correct modeling of the stresses and strains through the increased gravity field. However, particle sizes in the model to compensate for scaling effects are incorrect. Further, the tests do not replicate in situ soil conditions and loading history. Such tests, therefore, should only be used for verification of theories, parametric studies, verification of numerical analysis, or study of soil response phenomena.

3.5 Empirical Methods

Empirical methods have become widely accepted in routine engineering practice. The Discussion section of this guide discusses the use of SPT, CPT, BPT, and Vs tests to evaluate liquefaction potential, while Regulatory Position 1 discusses procedures that should be followed in the performance of these tests.

3.6 Probabilistic Approach

If a probabilistic method is sufficiently formulated to assess the potential for seismic soil liquefaction that takes into account the uncertainties in earthquake load, earthquake resistance, and method of liquefaction evaluation, systematically and quantitatively, the method can be incorporated and integrated into the analysis and design processes. The NRC staff will assess the methodology to ascertain its appropriateness.

4. FINAL EVALUATION OF SEISMIC STABILITY

In evaluating the potential for seismically induced ground failure at a nuclear facility site, the NRC staff will review the methods of analysis or evaluation used, the assumptions, the input parameters, and the original data obtained in the field and laboratory investigations. The exposition of these factors in the Safety Analysis Report should be adequate to permit the staff to perform an independent analysis and to verify the analysis presented.

In the final evaluation of the site's safety with respect to seismic stability of soils, the collective information and the studies applied should be considered as a whole in determining whether the margin of safety is acceptable. An acceptable margin of safety applicable to all cases cannot be fixed. It should be determined on a case-by-case basis by using engineering judgment, considering (1) the degree of conservatism in the input parameters, (2) the assumptions in the analysis, (3) the analytical methods used, (4) the extent of and reliability of the data base for the input parameters, (5) the definition of failure used, (6) the critical mode of failure, (7) the definition of factor of safety used, (8) the safety significance of the problem, (9) case history evidence of field performance under similar soil conditions, and (10) the degree of consistency in the results of the analyses and correlations.

D. IMPLEMENTATION

The purpose of this section is to provide guidance to applicants regarding the NRC staff's plans for using this regulatory guide.

Except when an applicant proposes an acceptable alternative method for complying with the specified portions of the NRC's regulations, the methods described in this guide reflecting public comments will be used in the evaluation of applications for construction permits, operating licenses, early site permits, or combined licenses submitted after January 10, 1997. This guide would not be used in the evaluation of an application for an operating license submitted after January 10, 1997, if the construction permit was issued before that date.

REFERENCES

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REGULATORY ANALYSIS

A separate regulatory analysis was not prepared for this Regulatory Guide 1.198. The regulatory analysis prepared for Draft Regulatory Guide DG-1105, “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites” (March 2001), provides the regulatory basis for this regulatory guide as well. DG-1105 was issued for public comment as the draft of this present regulatory guide. A copy of DG-1105, including the regulatory analysis, is available for inspection and copying for a fee at the U.S. Nuclear Regulatory Commission Public Document Room, 11555 Rockville Pike, Rockville, MD; the PDR’s mailing address is USNRC PDR, Washington, DC 20555; telephone (301)415-4737 or 1-(800)397-4209; fax (301)415-3548; e-mail <PDR@NRC.GOV>.