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Liquefaction Potential Evaluation for Arcadia Dam

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SYNOPSIS The paper presents the studies performed as part of the liquefaction potential evaluation for Arcadia Dam. The evaluation was performed by the Tulsa District, Corps of Engineers, using a modification of the "Simplified Procedure" developed by H. B. Seed. A discussion of the various decisions, judgements and procedures used to adapt the required studies to the site specific conditions is presented along with a description of drilling, sampling, sample handling, and laboratory testing. The most significant finding from the evaluation is that a useful relationship exists between standard penetration test (SPT) blow count values, laboratory cyclic shear strength and soil grain size. This relationship enabled the SPT data obtained in the silty sands and silts present at Arcadia to be used with the simplified procedure which is based on blow count data obtained in relatively clean sands.

INTRODUCTION

A liquefaction potential evaluation of the embankment-foundation system for Arcadia Dam, Oklahoma was undertaken as part of the seismic design of the structure. The dam site is located on the Deep Fork River about 12 miles northeast of Oklahoma City, Oklahoma, within zone 2 on the Seismic Risk Map of the United States, Algermissan, S.T. (1969). Zone 2 has potential for moderate earthquake damage and therefore, required an evaluation of the seismic hazards. This paper presents a general discussion of the case studies involved in a liquefaction evaluation as applied to an earthfill dam on a sand foundation. The primary purpose of the paper is to provide a discussion of the various decisions, judgements and procedures used to adapt the required studies to the site specific conditions. The evaluation was performed by the Tulsa District, Corps of Engineers, using state-of-the-art procedures. A detailed report on the evaluation was published by the Tulsa District (1982) and contains a complete record of all information beyond the scope of this paper.

GENERAL DESCRIPTION OF EMBANKMENT AND FOUNDATION

Embankment construction started in October, 1982, and is scheduled for completion in December, 1985. The dam will be a compacted, zoned earthfill embankment with an impervious clay core flanked by random shells. Maximum height of the embankment will be about 85 feet above the flood plain. Total length will be about 5300 feet with the flood plain accounting for approximately 1500 feet. Overburden in the flood plain averages 80 feet in depth and consists of interbedded alluvial deposits of clays, silts and sands. The clays are lean and silty, and account for approximately 65 percent of the foundation soils. The non-plastic silts and sands exist in layers and lenses varying in thickness from a few inches to several feet and classify predominantly as silty sands (SM). Soils outside the flood plain reach did not present a liquefaction hazard and will not be discussed in this paper. A section of the embankment and foundation is shown in figure 1.
SELECTION OF APPROPRIATE METHOD FOR LIQUEFACTION EVALUATION

The seismic evaluation of the embankment-foundation system was performed following recommended procedures from a Corps of Engineers publication titled, "Earthquake Design and Analysis for Corps of Engineers Dams," ER 1110-2-1806, (1977). This publication requires state-of-the-art procedures be used as appropriate. A search of the literature showed that compacted earthfill dams on non-liquefiable foundations are resistant to serious damage from earthquake shocks of the magnitude expected in seismic risk zone 2. Seed, Makdisi, and De Alba (1978). The seismic safety of the dam, therefore, depends primarily on the liquefaction potential of the embankment foundation. The problem was how best to determine the liquefaction potential of the foundation soils. The methods available for evaluating the liquefaction potential of the foundation soils range from empirical approaches to sophisticated analytical procedures involving finite element modeling of the embankment-foundation system. Due to the complexity of the alluvial deposits at Arcadia, the finite element procedures are very difficult to apply. Many of the advantages of this detailed analytical approach are offset by the simplifications that must be made in modeling the complex foundation conditions and determining the dynamic properties of each soil type. For these reasons, less complex methods were determined to be more appropriate for this evaluation. A modification of the simplified procedure developed by Seed and Idriss (1971) was used for this study. Professor Seed served as a consultant throughout the study and provided considerable guidance as to how the simplified procedure could be modified to evaluate the liquefaction potential of an earth dam foundation.

REASONS FOR MODIFYING SIMPLIFIED PROCEDURE

As is usually the case, none of the available techniques for evaluating liquefaction potential, including the simplified procedure used for this study, could be directly applied to the problem. The simplified procedure was developed using data obtained from sites that have been subjected to earthquakes where the liquefaction characteristics of the soils have been noted. The cases studied during development of the procedure were limited to sites with relatively level ground (implying little or no initial horizontal shear stresses present) and to sites where the potentially liquefiable soils were relatively clean sands. Two modifications were made in the published procedure to account for site specific conditions at Arcadia. The first of these modifications accounted for the initial horizontal shear stresses induced in the foundation by the dam embankment and the second deals with the high silt content in the potentially liquefiable soils.

SIMPLIFIED PROCEDURE FOR LIQUEFACTION EVALUATION

The simplified procedure for evaluating the liquefaction potential at the Arcadia dam site included the following steps.

(1) Determine the stresses induced by the design earthquake.

(2) Determine the cyclic shear strengths of the foundation soils by either field tests (SPT) or laboratory tests.

(3) Determine the liquefaction potential by comparing the shear stresses induced by the earthquake with the cyclic shear strength of the foundation soils.

DETERMINATION OF STRESSES INDUCED BY DESIGN EARTHQUAKE

The first step in determination of stresses induced by the design earthquake is determination of the design earthquake itself and the intensity of the resulting ground shaking at the site. The procedures involved selecting a design earthquake are beyond the scope of this paper but the importance of the required seismic studies should not be overlooked since the magnitude of the design earthquake has a significant effect on the liquefaction potential. The seismic studies for Arcadia resulted in a design earthquake with a magnitude mb=5.6 and a peak site acceleration of \( \ddot{a}_{\text{max}} = 0.12 \text{ g} \).

Once the design earthquake has been determined, the stresses induced in the foundation soils by this earthquake can be determined. The simplified procedure gives a reasonably accurate assessment of the stresses developed during an earthquake for level ground conditions. These stresses are expressed as a ratio using the following formula:

\[
\tau_{\text{eq}} = 0.65 \times \sigma'_0 \times \ddot{a}_{\text{max}} \times r_d
\]

Where:
- \( \tau_{\text{eq}} \) = earthquake induced stress
- \( \sigma'_0 \) = total vertical pressure at depth being studied
- \( g \) = acceleration due to gravity
- \( \ddot{a}_{\text{max}} \) = peak ground surface acceleration due to design earthquake
- \( r_d \) = depth reduction factor (see fig. 2)
Before these stresses could be used in the liquefaction evaluation of the foundation soils it was necessary to make corrections for the initial shear stresses induced by the dam embankment. The presence of initial shear stresses can have a major effect on the response of the soil to a superimposed cyclic stress condition. In general, the presence of initial shear stresses tends to reduce the rate of pore pressure generation due to earthquake shaking. The magnitude of the initial shear stresses were determined at various points in the foundation using finite element procedures. Figure 3 shows the relationship used to correct the calculated stresses induced by the earthquake for the presence of initial static shear stresses.

![Fig. 3 Correction For Initial Shear Stresses (after Seed, 1981)](image)

The correction was applied by dividing the earthquake induced stresses by the correction factor corresponding to the static stress ratio at the point in the foundation being studied. Once the correction is made, the stresses induced by the design earthquake are plotted versus depth as shown in figure 4. In applying the simplified procedure to the dam foundation, it was assumed that the peak ground surface acceleration at points on the surface of the embankment would be the same as that developed on level ground beyond the toes of the embankment. It was not immediately clear whether this assumption was appropriate or not so a limited dynamic analysis of the ground response to the earthquake was also performed. The computer program SHAKE was used for this analysis. Schnabel, Lysmer, and Seed (1972). The results of the computer analyses were similar to those of the simplified procedure. Accordingly it was appropriate to use the stated assumption concerning ground surface acceleration for the case being studied. However, for other cases, a final conclusion on the ground response under earth dams should not rely solely on the simplified procedure unless it is clear that varying the ground surface acceleration at different points on the dam will not have a significant effect on the potential for liquefaction.

![Fig. 4 Stresses Induced by the Design Earthquake](image)
DETERMINATION OF CYCLIC SHEAR STRENGTHS

The second step in the simplified procedure is to determine the cyclic shear strength. The cyclic shear strength is defined as the cyclic shear stress causing liquefaction in the number of stress cycles corresponding to the design earthquake. This strength can be determined from SPT data or by means of an appropriate laboratory test program. Both SPT and laboratory tests were used for this study. In order to obtain valid repeatable data from the above tests it is mandatory that proper procedures for drilling, sampling, sample handling, and testing be followed. Due to the importance of these activities a description of the procedures used for each activity is presented. Cone penetration tests (CPT) were used to verify and extend the information found during the SPT program and a brief description of the CPT program is also presented.

Standard Penetration Testing

The standard penetration test (SPT) is sensitive to several factors. To assure valid repeatable results, the following special equipment and procedures were used for this study.

Equipment. An automatic drop hammer with a free falling weight was used for the SPT program. This type of equipment assured that the same amount of energy would be imparted to the drill rods with each blow. Since the liquefaction evaluation using SPT results is based on blow count obtained using the rope and cathead type of equipment, it was necessary to correct the field blow counts obtained using the automatic hammer because a free falling hammer imparts more energy to the drill rods. Kovacs, Evans and Griffith (1977), Kovacs, Salomone and Yokel (1980).

Drilling Procedures. The drilling fluid level was maintained above the level of the water table at all times. The weight and viscosity of the drilling fluid were controlled so that the cuttings would be effectively removed from the hole. Cuttings were cleaned from the hole by stopping rotation at the required depth and maintaining circulation until the final cuttings were removed. Circulation time after reaching required depth was about 30 seconds per 10-foot hole depth. To avoid unnecessary disturbance of the material being sampled, both the drill rotation speed and the circulation pressure were controlled. The hole was cleaned out between every SPT sample with a 4-inch fishtail bit fitted with upward deflectors on the circulation ports. To prevent disturbance of the material being tested, slow withdrawal rates of about 0.3 feet per second were used near the bottom of the hole when removing tools after cleanout. Procedures of the SPT sampling itself were standard.

SPT Results. For the purposes of this study, only SPT blow count data on materials with a plasticity index (PI) less than 4 was used. The results are plotted in figure 5.

![SPT Blow Count Versus Depth](image)

Fig. 5 SPT Blow Count Versus Depth

These results have been corrected for overburden pressure as described by Seed and Idriss (1971) but they have not been corrected for silt content. The site specific correction for silt content is described in a later paragraph titled Correlation Between SPT Blow Count, Silt Content, and Laboratory Cyclic Tests Results.

Cone Penetration Testing

The CPT investigations were designed to verify and extend the soil information found during the SPT program. The CPT soundings were performed using a truck mounted electronic cone penetrometer. Fugro, Inc., Consulting Engineers and Geologist, conducted the investigations. The general procedures used for the CPT investigations of the Arcadia Site have been described elsewhere, Sangerlat (1972). The results of the CPT program were similar to the SPT data in that both records indicated essentially the same layering of material types and penetration resistances when adjacent holes were compared. However, due to the very complex layering and range of values for both fines content and plasticity of the soils being studied, it was decided to use only SPT results as input to the liquefaction evaluation. The CPT sounding were used primarily to confirm that all low density areas of the foundation had been sampled.

Undisturbed Sampling and Sample Handling

Quality undisturbed samples for cyclic testing are very difficult to obtain and require special techniques. The following equipment and procedures were used for this study.
equipment. Undisturbed samples were obtained using a hydraulically operated fixed piston sampler with a 7/8-inch I.D. by 3-inch O.D. stainless steel sampling tubes. A noncorroding sample tube is necessary because the noncohesive samples must be transported and stored in the sample tubes until they are removed for testing.

Rilling Procedure. Reaming and cleaning of holes was performed after each push using the same procedure described for the SPT program. If, for any reason, the rig was lifted during a push, the sample was considered o be disturbed by the action of the sampler piston and as discarded.

Sample Handling Procedure. All samples were withdrawn from the hole and handled in such a manner that vibration and disturbance were absolutely minimized. Upon withdrawal from the hole, the tube was suspended vertically from the hoisting cable and a perforated expanding packer was installed firmly against the bottom of the sample with a porous stone and a disc of filter paper between the sample and the packer. If problems were encountered with part of the sample falling from the tube upon removal from the hole, the packer was installed while the lower end of the tube as still submerged in the mud. After installation of the bottom packer, the sampler head and piston assembly were carefully removed and the tube placed in vertical rain racks and allowed to drain for 24 hours. The drainage water was collected by placing a jar under the tube. After placing the tube in the vertical drain rack, the top of the sample was cleaned to remove contaminated material and drilling mud, and the distance to the top of the sample was measured. After drying had been completed, this measurement was repeated. All measurements were recorded on the rilling logs. If a minimum of 250 ml of water had rained from the sample, the sample was to be frozen in the freezer boxes provided for this purpose. None of the samples used for this study drained absolutely minimum while transporting the unfrozen samples. A final measurement to the top of the sample as taken upon arrival at the laboratory to determine if any settlement occurred during shipment. The care taken during shipment was effective in that no settlement occurred during transport.

Laboratory Testing

General. The laboratory testing program included cyclic triaxial tests, cyclic simple shear tests, monotonic triaxial tests, relative density tests, and classification tests. Unlike the cyclic shear tests, the relative density and monotonic triaxial tests do not provide a measure of dynamic strength. They do, however, provide a qualitative measure of liquefability. An outline of the testing program and its purpose follows.

Selecting Samples for Testing. All samples were x-rayed at the laboratory before removal from the sampling tubes. The negatives were used in selecting locations for taking specimens from the tubes. The procedure used in extracting the samples involves using a tube cutter with stiffening collars to cut an approximate 8-inch sample length, then vertical extrusion with a hydraulic cylinder.

Cyclic Triaxial Tests. The purpose of this testing was to determine the stress conditions causing liquefaction in undisturbed soil samples. Test procedures were in accordance with widely accepted procedures. Seed and Peacock (1971); EN 1110-2-1906 (1980). Twenty-three isotropically consolidated, stress controlled, cyclic triaxial tests were performed on representative foundation samples. The initial effective confining pressure was varied from 1 to 3 tons per square foot but it had negligible effect on the cyclic strength. The cyclic load was chosen such that failure would occur between 2 and 100 cycles, with failure defined as 5 percent double amplitude strain. Figure 6 shows test results for all material types plotted as the log of the number of cycles to 5 percent double amplitude strain versus the applied cyclic shear stress ratio $\sigma_d/\sigma_3$, where:

$$\sigma_d = \text{cyclic deviator stress},$$

$$\sigma_1 - \sigma_3, \text{ and}$$

$$\sigma_3 = \text{initial effective confining pressure}.$$

![Fig. 6 Cyclic Triaxial Tests Results](image-url)
The test results show a relatively narrow range of strengths regardless of the material type, therefore, only one design curve was selected for use in the liquefaction study. The lower bound curve is shown so that the minimum strength value can also be determined. The design strength is the stress ratio that will produce liquefaction in the number of uniform cycles that are equivalent to the stress-time history that would be produced by the design earthquake. The appropriate number of cycles depends on the magnitude of the design earthquake and is five for this analysis. The stress ratio at which the design line intersects five cycles is selected as the design strength. The design strength determined from the cyclic triaxial tests must be corrected before being used in the analysis because the cyclic triaxial test does not adequately reproduce the in-situ stress conditions present during an earthquake. This correction factor (cr) will vary between approximately 0.55 and 0.70 depending on the relative density of the specimen. The value used for this study was conservatively chosen as cr equals 0.57 and is applied by multiplying this value by the strength determined from the cyclic triaxial tests.

Cyclic Simple Shear Tests. The purpose of this testing was to verify and extend the information found with the cyclic triaxial test program. The cyclic simple shear test more accurately approximates field stress conditions and therefore does not require the cr correction used for the cyclic triaxial tests. Seven cyclic simple shear tests were performed. Since varying the initial effective confining pressure made little difference in the cyclic strength determined from the cyclic triaxial test program, each of the seven samples were tested at an initial effective vertical confining pressure of 2.5 psf. As in the cyclic triaxial test, the cyclic load was chosen such that failure would occur between 2 and 100 cycles with failure defined as a 5 percent double amplitude strain. Figure 7 shows test results for all material types plotted as the log of the number of cycles to 5 percent double amplitude strain versus the applied cyclic shear stress ratio $\tau/\sigma_v$, where:

\[ \tau = \text{cyclic horizontal stress}, \]
\[ \sigma_v = \text{initial effective vertical confining pressure} \]

The test results show a narrow range of strengths. The design curve was selected as an average value. The lower bound curve is shown so that the minimum strength value can also be determined. The design strength is then selected as the stress ratio at which the design line intersects the number of uniform cycles that are equivalent to the stress-time history as produced by the design earthquake.

Correlation Between SPT Blow Count, Silt Content and Laboratory Cyclic Test Results

The simplified procedure uses the empirical relationship shown in figure 8 for comparing SPT blow count data to the cyclic stress causing liquefaction.

![Fig. 8 Correlation Between Stress Ratio Causing Liquefaction in the Field and SPT Blow Count (after Seed, 1981)](http://ICCHGE1984-2013.mst.edu)
this chart is based on surveys of areas where liquefaction has or has not occurred and the standard penetration resistance of the deposit is known. The soils on which this chart was based were relatively lean sands as opposed to the silty sands and silts present at the Arcadia dam site. Because of this difference in soil properties, the relationship between low count values obtained at Arcadia and cyclic strength causing liquefaction would not necessarily be the same as that shown in figure 8. A significant portion of the cyclic laboratory testing program described above was undertaken to develop a site-specific correlation between SPT blow count, silt content and laboratory cyclic strength. Using figure 8, the results of the cyclic testing were converted to low count values and plotted along with SPT results, see figure 9.

Relative Density Tests. Tests results from 14 relative density tests on undisturbed piston samples show a minimum relative density of 70 percent and an average relative density of 80 percent. Relative density values cannot be applied to the simplified procedure, but they were performed as part of the liquefaction potential evaluation in order to have a value to compare with other studies. Liquefaction is unlikely to occur in silts or sands as dense as those tested, but due to inaccuracies in relative density testing of undisturbed samples the above results are considered to be inconclusive.

Monotonic Triaxial Tests. Three stress-controlled R tests with pore pressure measurements were performed on material that would be subject to liquefaction. All three tests exhibited a dilative response. A dilative soil will not liquefy so as to produce a flow slide because dilatancy will reduce the built-up pore pressure and increase the effective strength of the material, allowing only limited strains. Castro (1975). These tests, like the relative density tests, were not used with the simplified procedure but they provide additional evidence that liquefaction is unlikely.

DETERMINATION OF LIQUEFACTION POTENTIAL

The final step in the simplified procedure for determining the liquefaction potential is to compare the shear stresses induced by the design earthquake with the cyclic shear strength of the foundation soils. The SPT results were corrected for fines content and plotted versus depth along with the results of the laboratory tests which had been converted to blow count values. The stresses induced by the design earthquake were calculated both under the centerline of the dam and near the toes of the embankment. These stresses were also converted to blow count values and plotted together with the results of the field and laboratory tests. All of the above values are plotted on figure 10.

First International Conference on Case Histories in Geotechnical Engineering
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http://icchge1984-2013.mst.edu

431
Since the cyclic strength values represented by the SPT and laboratory test results are greater than the stresses induced by the design earthquake, the foundation is considered safe against liquefaction.

ADDITIONAL CONSIDERATIONS FOR SEISMIC SAFETY

Although not a part of the liquefaction potential evaluation, the following features were built into the embankment to increase the seismic safety. The large freeboard available at Arcadia is a major factor since the freeboard is still highly improbable should an earthquake occur with the pool at the level of the uncontrolled spillway (500-year recurrence interval) since the freeboard at this level is 24.5 feet. Another factor increasing the seismic safety of the embankment is that the outlet works structure is founded on rock and would not be impaired functionally by an earthquake. Finally, the sand drain incorporated into the embankment increases the seismic safety by controlling any seepage caused by earthquake induced cracks.

CONCLUSIONS

Due to the complexity of the soil stratigraphy, a simplified procedure based on empirical data was determined to be the most appropriate method of evaluating the liquefaction potential. It was necessary to make several modifications to the published procedure to account for site specific conditions. The most significant modification was the correction developed to account for the silt content of the foundation soils. In addition to the factors directly evaluated, several defensive design features were incorporated into the structure to increase the seismic safety of the embankment. The combined studies verified the adequacy of the embankment and foundation with respect to seismic stability. The studies also demonstrated the importance of recognizing the applicability of state-of-the-art methods and of the importance of developing procedures that are valid for site specific conditions.

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