The Donald M. Burmister Lecture

Department of Civil Engineering and Engineering Mechanics
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Seismic Design and Analysis of Embankment Dams: The State of Practice

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This Lecture traces the development of the state of practice in seismic design and analysis of embankment dams, starting in the mid 1960's and continuing through 2000. The key issue, evaluation of earthquake -induced liquefaction of saturated loose cohesionless material, will be discussed, with emphasis on key components of the problem. Remediation methods for fixing existing embankment dams judged to be unsafe, should the "design" earthquake occur, are summarized. Gaps in our knowledge are identified and he will sketch a vision regarding future developments in permanent deformation analyses, site characterization, remediation and ground motions.

About The Speaker

Dr. William F. Marcuson III is President of the American Society of Civil Engineers (ASCE) and is one of the nation's leading civil engineers. He holds degrees in civil engineering from The Citadel, Michigan State University and North Carolina State University. He has received five national awards from ASCE, including the Norman Medal, civil engineering's oldest honor. In 1995 he was honored by the National Society of Professional Engineers as their Federal Engineer of the Year. His career included research and administrative positions at the U. S. Army Engineer Waterways Experiment Station, where he served as Director of the Geotechnical Laboratory for nearly 20 years, prior to his retirement in 2000. He is the only engineer to be named the Corps of Engineer's Engineer of the Year twice (1981 and 1995), and he was honored by the Corps as their Civilian of the Year in 1997. He was elected to the National Academy of Engineering in 1996 for his contributions to the design and analysis of embankment dams.

The late Prof. Donald M. Burmister (1895-1981) is one of the pioneers in the field of Soil Mechanics and Geotechnical Engineering. He established the Soils Laboratory at Columbia University in 1933. He was a faculty member for 34 years before retiring in 1963. During his tenure at Columbia University, he investigated earthworks and foundations for over 400 projects. Most notably among these were the Brookhaven National Laboratory, the Throgs Neck, Tappan Zee and Verrazano Narrows Bridges, the First New York World Fairs at Flushing Meadows, and the reconstruction of the White House in 1950. He has developed several soil testing methods and his soil classification system is still widely used. He also contributed to the first use of digital computer in conjunction with his theory of the layered pavement system.

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Seismic Design and Analysis of Embankment Dams: The State of Practice

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ABSTRACT: This paper traces the development of the state of practice in seismic design and analysis of embankment dams, starting in the mid 1960s and continuing through today. The key issue, evaluation of earthquake-induced liquefaction of saturated loose cohesionless material, is discussed, with emphasis on key components of the problem. Remediation methods for fixing existing embankment dams judged to be unsafe, should the "design" earthquake occur, are summarized. Gaps in our knowledge are identified and we have sketched our vision regarding future developments in permanent deformation analyses, site characterization, remediation, and ground motions.

KEY WORDS: cohesionless material, dams, deformation analysis, earthquakes, embankments, liquefaction, remediation

INTRODUCTION

The purpose of this paper is to sketch the state of the art in geotechnical earthquake engineering, how it got there, and the authors' vision of where it is headed in the next 20 years or so. Since our experience is primarily with the seismic engineering of large embankment dams, these discussions will concentrate mostly on embankment dams and their foundations, though the principles and methods apply as well to other kinds of engineered structures.

We observe at the outset that there is one major problem in seismic safety of embankment dams: liquefaction of either embankment or foundation soils. While there are other earthquake-induced phenomena the engineer needs to consider, their importance pales in comparison to that of liquefaction, so our emphasis will be mostly on how to deal with liquefaction problems.

ORIGINS OF MODERN GEOTECHNICAL EARTHQUAKE ENGINEERING

Into the mid-1960s, the state of practice in seismic analysis of earth dams was essentially the seismic coefficient method—that is, the use of a static analysis of the factor of safety against sliding with the addition of a horizontal force (equal to the product of a seismic coefficient and the weight of the potential sliding mass) to represent the effect of seismic shaking. This was also called a pseudo-static analysis. The selection of a seismic coefficient was based on judgment and experience of slope behavior during past earthquakes, and these coefficients were embodied in seismic design codes (e.g., the design criteria established for Japanese practice by the Japanese Committee on Large Dams (Japanese Society of Civil Engineers, 1960)). There

were two serious weaknesses in this approach: it lacked a rational basis for choosing the seismic coefficient, and it did not take into account possible loss of strength and stiffness of the soil as a result of shaking. There was a general awareness among engineers that shaking of sands could result in a quick condition—i.e., quicksand—but no methods of analysis were available.

The first, to our knowledge, specific suggestions for a rational basis for selection of seismic coefficients were made by Ambraseys (1960), who proposed coefficients based on elastic response analyses of the embankments. Figure 1 (Seed and Martin, 1966) shows his recommended coefficients based on maximum elastic response and root-mean-square response, and damping of 20 percent of critical, together with bands representing typical U.S. practice and the Japanese code.

R. V. Whitman, working with the late D. W. Taylor, first proposed the important concept that the effects of earthquakes on embankment stability should be assessed in terms of the deformations they produced rather than on a factor of safety against slope failure. The concept was described in detail in two memoranda dated April 14 and May 20, 1953, and signed by Taylor, to the Corps of Engineers. Over the next two decades, a number of earthquake engineering pioneers were developing this deformation concept.

The late Nathan Newmark, who served on an advisory board with Taylor for the Corps of Engineers, further developed Whitman's concept and presented a method of analysis based on it in his Rankine Lecture of 1965 (Newmark, 1965). The method as presented by Newmark assumes rigid-plastic behavior of the embankment during the earthquake, and that the time history of the earthquake
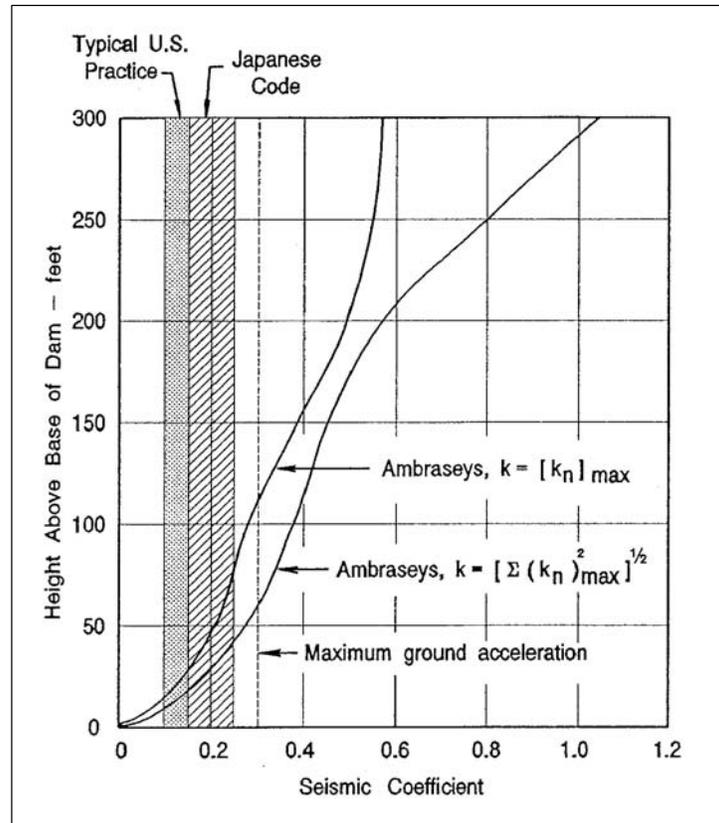
Fig. 1. Seismic coefficients suggested in 1966 for use in
pseudostatic analysis (after Seed 1966)

motions is known. The sliding mass is idealized as a rigid block on an inclined plane, which slides on the plane whenever the shearing resistance of the contact is overcome by inertial force due to shaking.

Two of the present authors (Hynes-Griffin and Franklin, 1984) used the Newmark analysis method together with viscoelastic response analyses, and the suggestion that a limited deformation could be tolerated, to propose a rational basis for choosing a seismic coefficient. Based on the assumptions that there is no liquefaction or major strength loss due to shaking or movement in the dam or foundation, and that an upper bound sliding displacement of one meter in a large dam would not result in catastrophic release of the reservoir, they recommended a (highly conservative) seismic coefficient equal to one-half of the peak base acceleration as a screening tool. However, if either a less conservative evaluation is desired, or a different value of the tolerable displacement is appropriate, the curves of Figure 2 can be used to select a different coefficient.

Two events in 1964 made it clear that there was an urgent need for methods of analysis of seismically induced liquefaction. The Good Friday earthquake in Alaska and the Niigata, Japan, earthquake of June 16 produced

widespread liquefaction of foundation soils and severe damage to structures. Photographs of overturned apartment buildings (Figure 3) and sand boils (Figure 4) in Niigata became quite famous.

Rapid progress was made in the 1960s on the development of methods to estimate the dynamic response of dams and foundations, and the potential for soil liquefaction. At the center of these developments were the late Professor H. Bolton Seed and his associates and students at the University of California at Berkeley. Professor Seed's genius was multi-dimensional; his influence probably owed as much to his talents for organization and leadership as to his technical and analytical ability. He orchestrated a group of researchers of diverse talents to produce an integrated approach to the earthquake safety problem.

Professor Seed was a pragmatist. His approach was aimed at achieving a workable engineering solution to a problem using the knowledge available at the time, and refining the solution as new knowledge became available. It was often inelegant. It was marked by assiduous collection, careful analysis, and insightful interpretation of data from past earthquakes and historical records of performance; the use of imperfect, but widely available, test methods (the cyclic
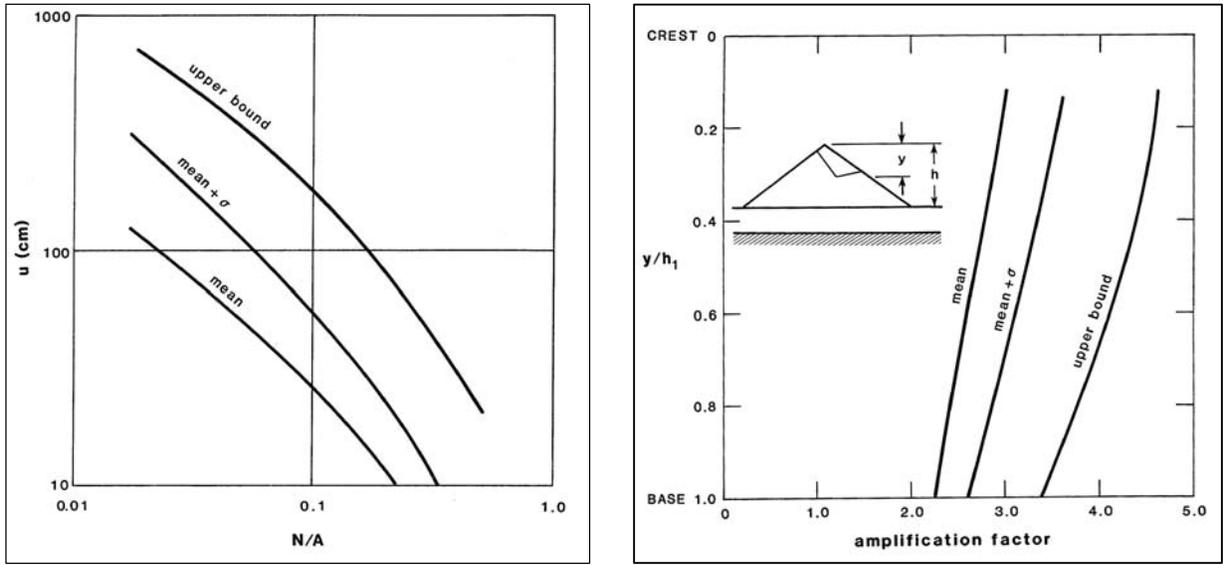
Fig. 2. (a) Permanent displacement u of a sliding block versus seismic coefficient N/A; (b) Amplification factors for linearly viscoelastic embankments at resonance (Hynes-Griffin and Franklin 1984)

Fig. 3. Overturned buildings due to liquefaction, Niigata, 1964
(Photo from National Information Service for Earthquake
Engineering, University of California, Berkeley)

Fig. 4. Effects of sand boils due to liquefaction, Niigata, 1964. (Photo from National Information Service for Earthquake Engineering, University of California, Berkeley)

triaxial test, the Standard Penetration Test), "calibrated" with empirical correction factors to match the results with observational data; and by simplified methods of analysis also "calibrated" to correct for errors introduced by the simplifications.

The perfect example of the Seed approach was the "Simplified Procedure for Evaluating Soil Liquefaction Potential" (Seed and Idriss, 1971). This procedure used a compilation of case histories in which sandy soils, under essentially level ground, had been shaken by earthquakes, with either the occurrence or non-occurrence of liquefaction, as indicated by surface manifestations such as sand boils, foundation failures, large displacements, and the like; and where Standard Penetration Test (SPT) values were available. (The same concept was independently developed and presented by Whitman (1971).) In this procedure, the shear stress on a horizontal plane in the soil

was estimated by analyzing the overlying soil as a rigid body, acted upon at the base by a horizontal force to accelerate it to the value at the ground surface. The non-rigidity of the soil was adjusted for by a depth correction factor, r_d, obtained by averaging values from numerous linear viscoelastic, one-dimensional analyses of the ground response during earthquake shaking. A further correction factor, c_r, was applied to account for the difference in cyclic triaxial test and field conditions and was a function of relative density. When points representing either occurrence or non-occurrence of liquefaction are plotted on a graph of stress ratio versus SPT blow count, N, points of liquefaction occurrence and non-occurrence are seen to occupy different regions of the graph (Figure 5) (Seed et al., 1983).

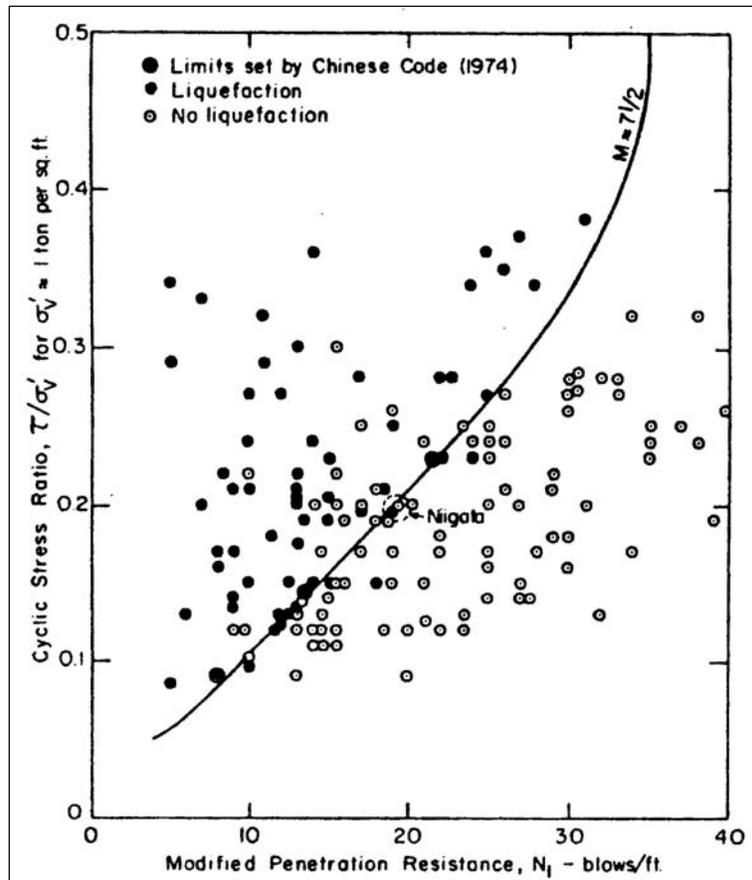The "Simplified Procedure" was refined and modified over the next decade and a half as new information was

Fig. 5. Plot of stress ratio versus SPT blow count, adjusted for overburden
pressure, showing occurrences of liquefaction (closed circles) and
non-liquefaction (open circles) (Seed et al. 1983)

gathered and methods of analysis were improved (Castro, 1975; Marcuson and Bieganousky, 1977a, 1977b; Seed,1981; Seed et al., 1981; Harder and Seed, 1986; Seed et al., 1985; Seed et al., 1988). Corrections and adjustments were made for the magnitude of the earthquake, the effect of overburden pressure on N-values from the SPT, fines content of the soil, the effects of SPT test apparatus and procedure on the energy input by the SPT hammer, and anisotropy of stress conditions.

For more precise evaluation of the liquefaction potential of soils, the approach developed by the Berkeley group used laboratory tests on "undisturbed" samples of sand obtained usually by means of piston tube samplers (though a preferred method was hand-carving of specimens, in those rare cases where direct access could be obtained.). The test specimens were subjected to cyclic loading under triaxial stress conditions. To cover an adequate range of stress conditions, a large number of test specimens was required.

As studies of the method progressed, however, it became more and more apparent that sample disturbance could not be ignored. Additionally, even if actual particle rearrangement were prevented, the sequence of stress changes during sampling, storage, and preparation for testing produced unavoidable changes in the properties of the sample. One approach to this problem was to freeze the soil in situ, while allowing free drainage at the freezing front to avoid disturbance due to expansion of freezing pore water (Yoshimi et al., 1977, 1978). While Yoshimi's method was to pull the frozen column out of the soil, others have used rock coring methods to obtain samples. The specimen could be kept in a frozen condition until the appropriate stress conditions were applied in the triaxial test device. Experiments at the University of California, Berkeley (Singh et al., 1979) suggested that this method produced near-perfect preservation of in-situ properties. However, because of the expense and level of effort it required, and the limitation of its use to free-draining (i.e., clean, cohesionless) soils, the method never came into widespread use.

Growing appreciation of the shortcomings of "undisturbed" sample testing led engineers to prefer in situ test methods (SPT, CPT) for evaluation of soil liquefaction potential, even for use in analyses by the more

sophisticated finite element methods. This approach generally uses empirical correlation of test values with liquefaction resistance, though some theoretical methods of analyzing the test values (e.g., expanding cavity theory for the CPT) are finding limited use.

It was after the Borah Peak earthquake of 1983 that the phenomenon of liquefaction of gravelly soils was forcefully brought to the attention of earthquake engineers who had largely ignored previous evidence of it, in the widely accepted belief that soil liquefaction occurred only in sands (Youd et al., 1985). Reexamination of data from the Alaska 1964 Good Friday earthquake revealed that liquefaction of gravel was largely responsible for the sliding of the Port of Valdez (Coulter and Migliaccio, 1966). Other documented occurrences were in the upstream shell of Baihe Dam during the 1976 Tang Shan earthquake (Liu et al., 1980) and gravels in sand boils resulting from the Fukui earthquake of 1948 (Ishihara, 1971). Extensive laboratory studies of gravel liquefaction were done at the Waterways Experiment Station (WES) by Hynes (1988).

The 1976 Tang Shan earthquake, which produced evidence of gravel liquefaction, also produced liquefaction of fine-grained soils at many sites in China (Wang, 1984), and a general awareness among U.S. earthquake engineers that liquefaction potential of fine-grained soils could not be ignored. Laboratory studies of fine-grained soils liquefaction were done at the University of Colorado at Denver and at WES by Koester (1992) and elsewhere by others.

The rapid development of the electronic digital computer revolutionized earthquake engineering research and development, as well as practice, during the 1960s. By 1967 there were probably 25 to 50 computer programs used in the United States for limit equilibrium analysis of slope stability (Whitman and Bailey, 1967). New analysis tools, e.g., the Finite Element Method (FEM) (Zienkiewicz, 1971) and the Fast Fourier Transform (FFT) (Cooley and Tukey, 1965), were incorporated into computer codes for dynamic analysis. This made possible the application of numerical analysis methods to problems that would have been computationally intractable using older calculation methods. Dynamic analyses could now be done using recorded time histories of earthquakes as input.

Very likely the most widely used of all dynamic analysis codes is the program SHAKE (Schnabel et al., 1972), and improved versions of it, which uses a one-dimensional model of the soil column and operates in the frequency domain. Non-linearity of the soil properties is adjusted for by using an "equivalent linear" formulation, which matches soil moduli and damping values to strain levels. An early 2-D code was QUAD-4 (Idriss et al., 1973), which was used to analyze dam cross-sections. It operated

in the time domain, but also used the "equivalent linear" method. These programs and others (LUSH, FLUSH) came out of the Berkeley group, and became universal standards. In the years since these programs were developed, many other methods of dynamic analysis of soil systems have followed.

It was recognized in the 1960s that instrumentation to record strong ground motions, and earth dam response, was of great importance. That decade was a period of great expansion of the United States' network of strong-motion instrumentation. In 1971, the Lower San Fernando Dam, near Los Angeles, California, suffered a massive upstream slide (Figure 6) as a result of liquefaction during the Magnitude 6.5 San Fernando earthquake, and loss of the reservoir was narrowly averted. By that time, the means were in place to collect an unprecedented body of data, both in terms of the number of records obtained and their geographical distribution. The California Institute of Technology lists more than one hundred records of ground motion from the San Fernando earthquake in their catalog of strong-motion records. The 1960's expansion of the strong-motion instrumentation network was a development of tremendous importance, since it provided a data base upon which meaningful statistical analysis could be exercised. On the other hand, this meant that the statistical correlations obtained from this data base were strongly biased, representing as they did mostly a single earthquake.

The Upper San Fernando Dam also moved a measurable amount during the 1971 San Fernando event, giving us an excellent case history of (1) earthquake-induced liquefaction in the Lower Dam, and (2) earthquake-induced soil softening and consequent limited movement in the Upper Dam, together with the best ground motion data base for any earthquake up to that time (Seed et al., 1975a,b).


WHERE WE ARE NOW

Liquefaction Resistance of Soil

From a practical point of view the primary geotechnical earthquake engineering issue related to embankment dams is liquefaction of either embankment or foundation soils. This issue can be further focused on engineered dams constructed on what has turned out to be liquefiable alluvial deposits. Estimation or measurement of the capacity for a soil to resist development of pore pressures leading to liquefaction has been the subject of intense research for more than thirty years. Over the years, various techniques have evolved to estimate in situ liquefaction resistance; the techniques of choice are empirically based on field occurrence data. This section will briefly summarize currently applied practice for estimation of in situ liquefaction resistance, quantified by the ratio of

(a) View from upstream
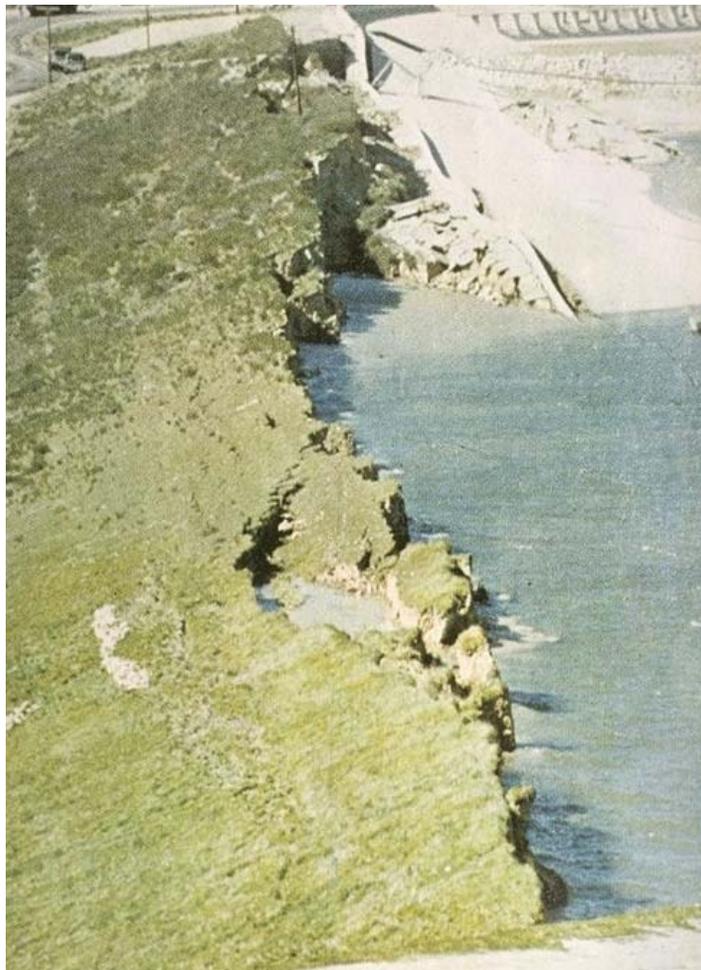
(b) At the crest

Fig. 6. Lower San Fernando Dam following the earthquake of 1971
(Photos from National Information Service for Earthquake
Engineering, University of California, Berkeley)

cyclic shear strength to pre-earthquake effective overburden stress; this is the Cyclic Resistance Ratio, CRR.

In 1996 T. Leslie Youd chaired a workshop to review developments and gain consensus regarding the state-of-knowledge, state-of-practice and future research needs for assessing liquefaction hazards (Youd and Idriss, 1997; Youd et al., 2001). The workshop was attended by 20 researchers and practitioners from the U.S., Canada, and Japan. To keep the workshop focused, the scope was limited to the evaluation of routine liquefaction resistance. Remarkably, the participants reached consensus recommendations on:

- use of the SPT, CPT and shear wave velocity measurements for evaluation of liquefaction resistance,
- use of the Becker penetration tests for gravelly soil,
- magnitude scaling factors, and
- correction factors K alpha and K sigma.

Probabilistic analysis and seismic energy considerations were also reviewed. A major recommendation made as a result of the workshop was the modification of the so-called "liquefaction curve" near the origin on the corrected blow count vs. cyclic stress ratio plot from Seed et al., 1985 (Figure 7). Originally the curve went through
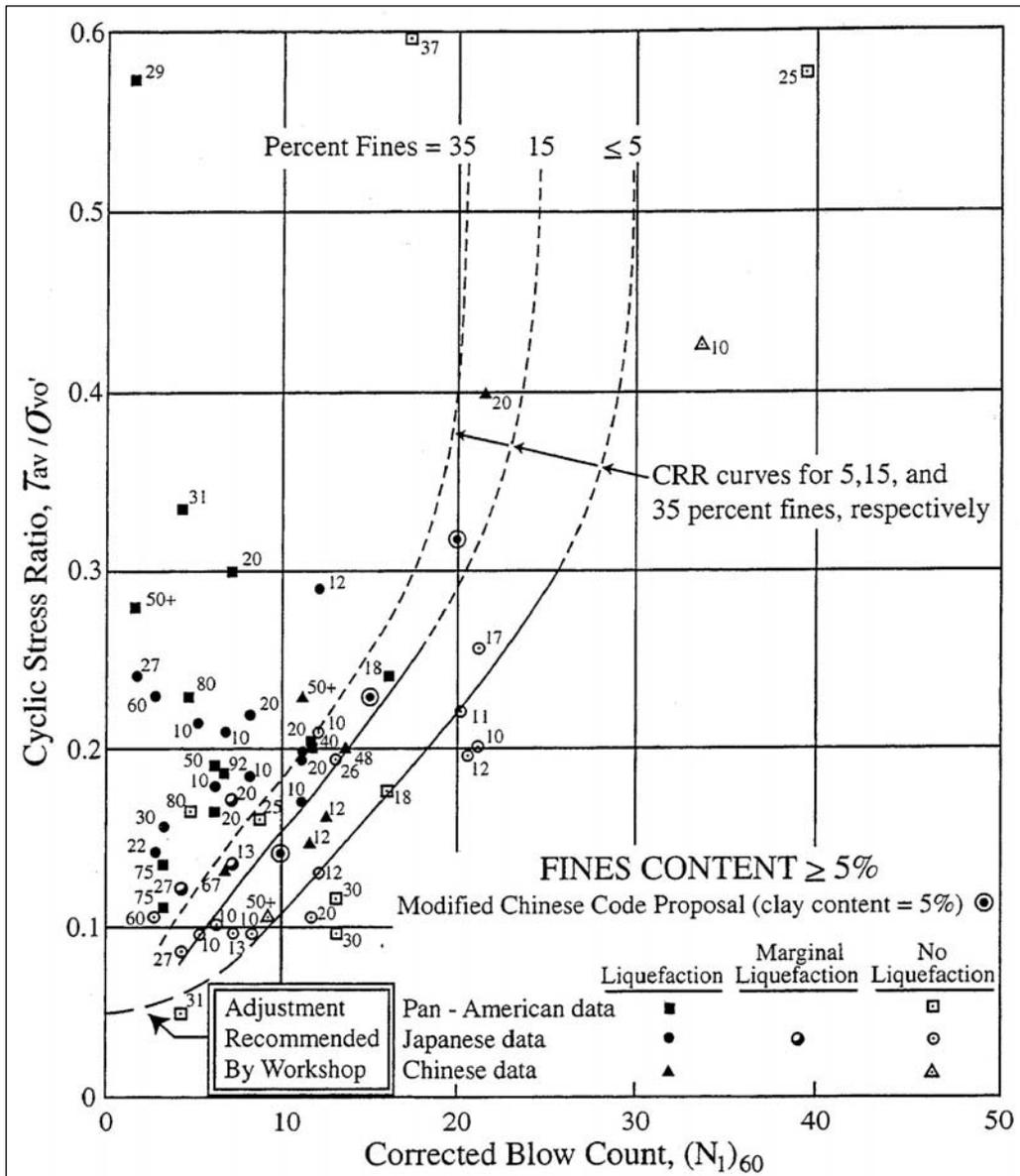

Fig. 7. Curves recommended for calculation of CRR from SPT data
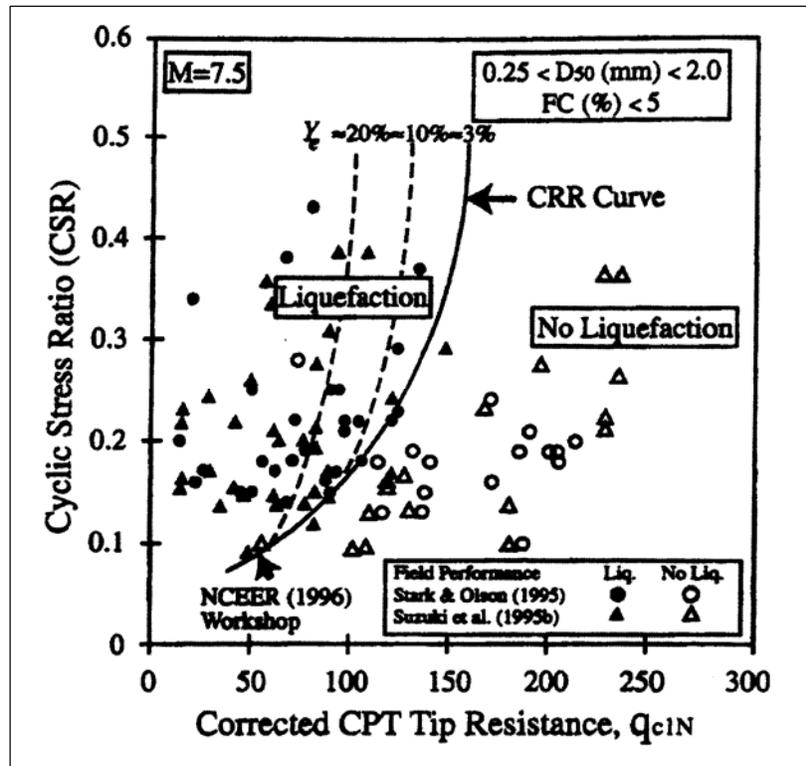(Youd and Idriss, 1997; Youd et al., 2001)

Fig. 8. Curves recommended for calculation of CRR from CPT data
(Youd and Idriss,1997; Youd et al., 2001)

the origin and the recommendation of the workshop was to modify the curve and terminate it at a stress ratio of 0.05, as implied by the threshold strain concept. Other important recommendations to come out of this workshop include: (a) standard definitions of the terms Cyclic Stress Ratio and Cyclic Resistance Ratio, and (b) the acceptance of an empirical data base and a so-called "liquefaction curve" for CPT tip resistance (Figure 8) along with methods to normalize CPT tip values. Since most of our earthquake data comes from magnitude 7.5 earthquakes, magnitude-scaling factors were suggested for use when concerned with earthquakes other than magnitude 7.5.

K alpha and K sigma received heated discussion and recommendations for K sigma emerged. The consensus was that K alpha needs more research. A summary of this workshop was also published by ASCE and the reader is referred to Youd et al. (2001) for more information.

Earthquake Ground Motions

One of the largest unknowns in a liquefaction analysis is the earthquake ground motion. In general, an acceleration time history for the dam site is needed at either bedrock or a (possibly notional) rock outcrop. This can be convolved through a site response analysis to obtain the motions that will affect the dam. Selection of the earthquake to be used in analysis is, in general, best done as a team effort

involving the engineer, the seismologist, and the geologist. The team needs to identify each potential earthquake source that could affect the dam site, the greatest earthquake each source can produce, and time histories representing the resulting attenuated ground motions at the dam site. It is not always apparent by inspection which one of those time histories is most critical, so analyses with multiple input earthquakes is often necessary.

Probabilistic seismic hazard analysis (PSHA) has become quite fashionable in recent years, and is useful for estimating structural response (National Research Council, 1988). For critical structures, such as an embankment dam whose failure would be catastrophic and cause loss of life, the Corps of Engineers uses deterministic approaches to obtain earthquake parameter input into a liquefaction analysis. If remediation is called for then a PSHA is often used to get a sense of the urgency of the required fix.

Dynamic Stress and Deformation Analysis

It is not sufficient to determine that materials are potentially liquefiable; one must estimate the amount of permanent deformation the dam is likely to experience and determine whether or not that deformation is tolerable, given such conditions as filter zone and core geometry, allowable freeboard and downstream development. A hierarchy of constitutive models is available for analysis of

the dynamic response of embankment dams to earthquake loading. The dynamic response models range from the relatively simple equivalent linear model to complex elastic-kinematic hardening plasticity models. Detailed critical assessments of these models may be found in Finn (1988a,b) and Marcuson et al. (1992). To estimate permanent deformations engineers today turn to nonlinear methods of analysis.

During seismic shaking, both transient and residual porewater pressures are generated in saturated soils. The residual porewater pressures are due to plastic deformations in the soil skeleton. These persist until dissipated by drainage or diffusion, and therefore they exert a major influence on the strength and stiffness of the soil skeleton. Nonlinear, elastic-plastic constitutive models of soil behavior under cyclic loading are typically based on a kinematic hardening theory of plasticity using either multi-yield surfaces or a boundary surface theory with a hardening law giving the evolution of the plastic modulus. These constitutive models are complex and some incorporate parameters not usually measured in field or laboratory testing. Soil is generally treated as a two-phase material using partially coupled or fully coupled equations for the soil and water phases. The coupled equations and the more complex constitutive models can make heavy demands on computing time (Finn, 1988b).

Validation studies of the nonlinear, elastic-plastic models suggest that, despite their theoretical generality, the quality of response predictions is strongly stress path dependent (Saada and Bianchini, 1987; Finn, 1988b). When loading paths are similar to the stress paths used in calibrating the models, the predictions may be good. As the loading path deviates from the calibration path, the prediction becomes less reliable. In particular, the usual method of calibrating these models, using data from static triaxial compression and extension tests, does not seem adequate to ensure reliable estimates of response for the dynamic cyclic shear loading paths that are important in many kinds of seismic response studies. It is recommended that calibration studies of elastic-plastic models for dynamic response analysis should include appropriate cyclic loading tests, such as triaxial, torsional shear, or simple shear tests. The accuracy of pore pressure prediction in the coupled models is highly dependent on the accurate characterization of the soil properties. It is difficult to characterize the volume change characteristics of loose sands and silts that control porewater pressure development because of the problems of obtaining and testing undisturbed samples representative of the field conditions. As a check on the capability of these models to predict porewater pressure adequately, it is advisable to use them to predict the field liquefaction resistance curve as derived from normalized Standard Penetration Test data (Seed et al., 1986).

Nonlinear methods used in current engineering practice to evaluate the seismic response and deformation of embankment dams include DYNAFLOW (Prevost, 1981), DIANA (Kawai, 1985), DSAGE (Roth, 1985), DYNARD (Moriwaki et al., 1988), FLAC (Cundall and Board, 1988), DYSAC2 (Muraleetharan et al., 1988, 1991), TARA-3 (Finn et al., 1986), and SWANDYNE 4 (Zienkiewicz et al., 1990a,b). Most of these programs are proprietary to their developers.

DYNAFLOW is a fully coupled nonlinear dynamic analysis procedure. The constitutive model of DYNAFLOW is based on the concept of multi-yield surface plasticity. The initial load and unload (skeleton) stress-strain curve obtained from laboratory test data is approximated by linear segments, and the curves for loading, unloading, and reloading follow the Masing criteria (Masing, 1926). The procedure can include anisotropy. The program allows dissipation and redistribution of porewater pressures during shaking. Validation of the program has been by data from centrifuge tests and case histories. The results of these forensic studies provide remarkable insight about internal damage to embankments from pore pressure generation and deformations.

DSAGE is one of the constitutive model options in the program FLAC, which is based on the explicit finite difference method for modeling nonlinear static and dynamic problems. The program uses an updated Lagrangian procedure for coping with large deformations.

DYNARD uses an explicit finite difference method for Lagrangian nonlinear analysis allowing large strains and displacements. It analyzes the deformation and response of earth structures to the simultaneous effects of gravity and seismic shaking using undrained strength and degradable undrained soil moduli. The cyclic and nonlinear behavior of soils is incorporated in the analysis by a 2-D bounding surface model, similar to that of Cundall (1979) and Dafalias and Hermann (1982).

DYSAC2 is a fully coupled nonlinear dynamic analysis procedure. The constitutive model is also based on bounding surface plasticity. The program has been validated in a preliminary way using the results of centrifuge model tests (Muraleetharan et al., 1993).

SWANDYNE 4 is a general-purpose elastic-plastic computer code that permits a unified treatment of such problems as the static and dynamic nonlinear drained and undrained response analyses of saturated and partially saturated soils to earthquake loading. The formulations and solution procedures, upon which the computer code is based, are presented in Zienkiewicz et al. (1990a,b).

The direct nonlinear approach is based on direct modeling of the soil nonlinear hysteretic stress-strain response. The U.S. Army Engineer Waterways Experiment Station (WES) has worked with the direct nonlinear dynamic

effective stress analysis methods of Finn for more than 10 years. This approach is in the program TARA-3 (Finn et al., 1986), which is proprietary to Finn. WES had extensive experience using this method in practice.

The objective in direct analysis is to follow the stress-strain curve of the soil in shear during both loading and unloading. Checks are built into the TARA-3 program to determine whether or not a calculated stress-strain point is on the stress-strain curve and apply corrective forces to bring the point back on the curve if necessary. To simplify the computations, the stress-strain curve is assumed to be hyperbolic. This curve is defined by two parameters, the shear strength and the in situ small strain shear modulus. The response of the soil to uniform all-around pressure is assumed to be nonlinearly elastic and dependent on the mean normal effective stress.

The response of the soil to an increment in load, either static or dynamic, is controlled by the tangent shear and tangent bulk moduli appropriate to the current stress-strain state of the soil. The moduli are functions of the level of effective stress, and therefore, excess porewater pressures must be continually updated during analysis and their effects on the moduli taken progressively into account. These pressures are modeled in TARA-3 using the Martin-Finn-Seed porewater pressure model (Martin et al., 1975).

Remediation

From an embankment dam perspective, the problem is not how to site, design and construct a new dam. While it is true that many of the best dam sites in many parts of the world have already been used, it is also true that using existing engineering technology, engineers today are capable of designing and constructing new dams that will behave acceptably during the design earthquake. If, for example, there are loose alluvial sands in the foundation, you simply remove them. On the other hand, engineers are faced with existing dams, some of which were designed prior to 1960, founded on alluvial material that is potentially liquefiable. It is the seismic retrofit of these existing dams that are the key concern facing geotechnical earthquake engineers who are focused on embankment dams—the scope of this paper. In order to rehabilitate a deficient embankment dam to prevent potential seismic instability, one must either change the engineering properties of the dam and/or foundation, modify the geometry of the existing dam, or both.

If predicted permanent deformations are estimated to be small and tolerable the engineer's job is finished. On the other hand if the deformations are intolerable and the dam is not to be taken out of service, then the engineer is faced with the issue of seismic remediation. There are a wide variety of treatment methods in use; some of the more important ones are discussed below. Two excellent sources of information on methods of remedial treatment are the publications by the Japanese Geotechnical Society (1998) and the Japanese Port and Harbour Research Institute (1997).

Berms and Buttresses. Upstream and downstream berms and buttresses are used to increase the effective overburden pressure on the problem material and thus increase its liquefaction resistance. This increase in overburden also causes a small amount of consolidation and thus increases the density. Berms and buttresses are also used to increase the length of the failure surface, provide a counterweight to limit movement, and maintain a remnant section. The effectiveness of a berm is generally limited to a zone that is about as deep as the berm is thick. A berm or buttress can not reduce the factor of safety during day-to-day operation, and its presence is obviously verifiable. If coarse-grained soil or rock is available, berms and buttresses can, with some difficulty, be constructed on the upstream shell without lowering the pool.

Excavate and Replace. This method assures that the problem material is removed and replaced with a nonliquefiable material. Excavation and replacement offers the advantage of providing relative assurance that what was designed has actually been constructed in the field. It is often expensive and operationally difficult. Dewatering is almost always required and in many cases the reservoir must be lowered significantly. This approach is most useful when the problem material is near the ground surface. In addition to excavating liquefaction-prone material, the excavation-and-replace method can also be used after an earthquake to remediate shallow cracking.

In-situ Densification. When excavation and replacement are ruled out for some reason, in-situ densification can sometimes be used to decrease the potential for liquefaction by decreasing the void ratio of the problem material. The method includes vibro-techniques, dynamic compaction, compaction grouting, and displacement techniques. In-situ densification is most effective when the material to be improved is close to the ground surface and has limited fines content. To date this approach has not been used under an existing dam except in cases where most of the embankment over the foundation zone to be densified has been temporarily removed. Foundation densification will not be uniform and could adversely change the dam-foundation interface. Cracking might occur which could increase the risk of piping. Verification of the amount of improvement and of the spatial variability of the improvement is required.

In-situ Strengthening. While somewhat similar to in-situ densification, in-situ strengthening forms a composite material that is strong enough to ensure stability. Soil nailing, stone columns, and methods of deep soil mixing are examples. Some of these methods may also cause consolidation and increase the strength of the soil around

the feature, but this increase in strength is generally ignored in stability analyses. In-situ strengthening is generally most effective when the potentially liquefiable material is confined to a relatively thin layer but it can be implemented for thick deposits in the case of deep soil mixing. Caution, as discussed under "In-situ densification," is suggested if used under an existing structure. Conventional grouting of the foundation through the embankment has not been used for the purpose of strengthening for two reasons. One is the possibility of hydraulic fracture in the embankment. The other is that it is not easy to determine to what extent the grout has penetrated the zones needing improvement.

Increase Freeboard. An increase in freeboard may be used when the seismic analysis indicates that the dam is marginally stable and/or only small earthquake-induced deformations are probable. Obviously, this approach decreases the probability of overtopping associated with settlement or slumping of the crest.

Drainage. This approach provides for relief of seismically induced pore water pressure. Techniques include strip drains, stone columns, and gravel trenches. Gravel trenches can be used to intercept migrating elevated pore pressure plumes. Analysis of drains is problematic; however, because accurate and reliable in situ permeabilities are extremely difficult to obtain. Care must be used in placement of stone columns and gravel trenches so that stone crushing is minimized and permeability remains high. Stone column spacings should be sufficiently small to dissipate pore pressures to a low level during the earthquake, and to prevent the occurrence of high hydraulic gradients that could carry large amounts of fines into the gravel drains. Potentially, stone columns could be flushed periodically to maintain their performance. It is inevitable that extra drains added for remediation will reduce the length of flow lines and thus will increase the seepage gradients under static pool conditions. Hence, even if the drains are designed as filters with respect to the adjacent material, the static safety of the dam will be somewhat reduced and more water will filter through the dam.

Combinations. Various combinations of the approaches can be used.

Validation. Validation of the rehabilitated dam's performance under design loads is still an issue. The Lake Champlain South Dam, improved with stone columns in the toe prior to the 2001 Nisqually, Washington earthquake, experienced about 0.16 g. No cracks, deformations, or evidence of piping were found in or around the dam after the earthquake (Hausler and Koelling, 2003). Field validation of the current practice will require a larger empirical data base of strongly-tested rehabilitated dams.

Good judgment requires that absolutely nothing be done to increase the likelihood of failure during normal static operating conditions such as increasing the likelihood of failure by piping, or hydraulic fracturing. One must also verify that any intended densification and/or drainage are achieved. Verification of densification is usually achieved by the use of a test section where before-and-after SPT, CPT, and/or shear wave velocity measurements are used to assess the effects of the densification technique attempted. If the technique is found successful in the test section, the same before-and-after index tests are used to verify densification of the improved zone.

Site Response

The dynamic response of a given site to a given earthquake may be estimated using several methods. The site may be idealized as consisting of horizontal layers of infinite extent and analyzed using SHAKE. SHAKE is a 1-D wave propagation code that has seen routinely used in evaluating site response for the last 30 years. If the site does not lend itself to 1-D analysis, then 2-D methods such as QUAD-4, FLUSH, and other codes are routinely used. We prefer the use of QUAD-4 and other programs that operate in the time domain as opposed to programs such as FLUSH that operate in the frequency domain. Calculations in the time domain provide greater insight as to when and where liquefaction is triggered and how it propagates through and under a dam. Frequency domain methods rely on the principle of superposition, and thus on linearity of stress-strain relations, obviating the possibility of using true nonlinear relations.

Critical input parameters needed for site response analyses include the following:

- Depth to rock. This is important because this depth greatly influences the natural frequency of the site. Top of rock is not usually a uniform, level surface, and a representative value needs to be chosen for each column or section analyzed.
- Location of the water table and the maximum elevation the water table is likely to be at the time of the earthquake. Since liquefaction is often the critical issue, saturation is key and soil below the water table is routinely assumed to be saturated.
- Soil layer properties. Layer thickness, lateral extent and soil type are needed inputs for each layer.
- Drained and undrained (static) shear strengths.
- Soil stiffness and damping. Both shear modulus and damping, as functions of shear strain, are required.
- Density and relative density.

In a perfect world we would obtain and use all the above properties in 3-D. The world is not perfect, however. With the rapidly increasing power, capacity, and speed of computers, voluminous computation methods are less of

an issue. If we are to take full advantage of these computational tools, our R&D on soil properties needs to be accelerated.

Site Characterization

The purpose of a subsurface investigation is to identify and characterize the subsurface materials. Key parameters are the depths to rock and the water table along with the characterization of the soil stratigraphy and properties: soil types, strengths, permeabilities, etc. Before initiating a subsurface investigation, a review of the local, regional, and site geology is in order. The geological review gives good insight into what one might expect to find during a field site investigation and what tools might prove useful.

Geophysical Investigations. As a first step, a geophysical investigation should be conducted using surface methods. Surface seismic methods afford a quick, nonintrusive, and relatively inexpensive means of assessing material properties in preparation for more specific and accurate exploration and testing to detect regions of lower-velocity materials (an indicator of low liquefaction resistance). Typical land-based seismic geophysical methods include surface vibratory, surface refraction, and seismic borehole approaches (crosshole and downhole). The Spectral Analysis of Surface Wave (SASW) techniques developed by Prof. K. H. Stokoe II and his colleagues at the University of Texas are valuable (Stokoe and Santamarina, 2000; Stokoe et al., 1994, 2003; Tokimatsu, 1995). These methods are used to measure shear wave velocity, V_s, compression wave velocity, V_p, and Rayleigh wave velocity, V_t, as desired, as functions of depth within an embankment and the underlying foundation material. For engineering purposes, V_s is generally estimated as 1.025 (V_t).

Cone Penetration Test (CPT). From the results of geophysical investigation estimates of initial layering and thickness of layers at the site can be made. It is suggested that the geophysical investigation be followed by an appropriate number of CPT probes. We say "appropriate number" because it is impossible to determine where and how many CPT probes are required without some subsurface information in hand, such as that provided by the SASW study. The CPT data are useful in determining the soil profile at specific locations, including soil type and layering, layer thickness, soil strength and liquefaction potential. In some cases complex cones can be used to gather shear wave velocity data, which can be compared with the SASW data; and permeability data, which are useful in liquefaction evaluations.

The CPT has gained wide acceptance as a subsurface investigation tool for a variety of applications, particularly in soft clays and loose soils prone to earthquake-induced liquefaction. The electrical friction cone penetrometer (several variations, depending on instrumentation design) have replaced the earlier, mechanical version, due primarily to the ability to obtain continuous, direct measurement of the resistance of soil to tip penetration and side friction. These two parameters have been correlated with soil type and behavioral properties (Douglas et al., 1981; Olsen and Farr, 1986; Robertson and Campanella, 1983; and Olsen, 1994). One failing of the CPT is the inability to obtain physical samples, although special cones that can collect soil samples have been developed for hazardous waste investigations (Leavell et al., 1996). Sample collection slows down the field work and reduces the economy of using the CPT; on the other hand, borings usually require circulation of drilling fluid and much greater labor and time (and accordingly, expense) to advance through similar depths of investigation.

In deference to the large field performance data base on liquefaction potential that has evolved using the SPT, researchers have usually elected to convert CPT data into equivalent SPT values and take advantage of existing correlations, although this is changing. Several factors influence both the CPT and the SPT, including (Douglas et al., 1981):

- The SPT and CPT are similarly affected by certain soil properties, such that CPT results are directly relatable to SPT results for liquefaction potential.
- CPT profiles provide much finer resolution of stratigraphy than do SPT results (and liquefaction failure may occur in thin layers that could lead to sliding).
- The typically large variation of test results associated with the actual performance of an SPT is substantially avoided with the CPT, which is more automated. However, the CPT tool must be properly calibrated.

Olsen (1984) proposed normalization of measured CPT data to a function of the effective overburden stress, followed by conversion to continuous, normalized SPT equivalents. More recently, updated procedures to estimate liquefaction resistance directly from CPT measurements have been developed based on a combination of SPT and CPT measurements at liquefied sites, laboratory testing, and some theoretical reasoning (Olsen; Robertson and Wride; in Youd and Idriss 1997).

Standard Penetration Test (SPT). The SPT historically has been the most widely employed in situ test in this country and perhaps worldwide for preliminary subsurface investigation. The SPT is still in common use for preliminary in situ investigation of liquefaction potential as a result of its empirical correlation to field performance. However, the term "standard" is something of a misnomer, as there are many variations in practice. Other countries have also developed indigenous versions of the test, unconstrained by the U.S. "standard." The blow count from the SPT is greatly influenced by factors such as the

method of drilling, the type of hammer and anvil, the sampler design, and type of mechanism for lifting and dropping the hammer, the type of drill rod, and the length of rods (depth) (e.g., Kovacs et al., 1983). The SPT also suffers from the disadvantages that it yields only a discontinuous record of penetration, representing a two-foot interval by a single number, and split spoon samples are useful only for visual and laboratory classification purposes. When gravels or gravelly material are encountered and need to be investigated, the Becker Hammer is the tool of choice. Results of Becker Hammer testing are correlated back through SPT results to evaluate liquefaction potential (Harder and Seed, 1986).

Sampling. Finally the needed number of disturbed and undisturbed soil samples can be obtained at critical locations (horizontally and vertically) of materials for visual observation and laboratory analysis and testing. Disturbed samples, useful for visual classification, grain size analysis, water content determination, and determination of Atterberg limits, may be obtained by the SPT split spoon or by soil augers. Undisturbed samples are usually taken in loose cohesionless soils below the water table.

Soil samples are disturbed both mechanically and by changes in their effective stress state on removal from a deposit in situ and transportation to testing facilities. Marcuson and Franklin (1980) reviewed techniques and apparatuses that are still commonly applied to sample granular soils for liquefaction potential evaluation. They reported that fixed piston sampling operations tend to produce the best of tube samples when used in medium dense sands. Tube sampling was observed to densify loose sands and dilate dense sands. The implication is that cyclic strength test results on tube sampled specimens, if interpreted directly, would be unconservative in the case of sands that were loose in situ, and conservative in dense sands. As discussed earlier, in situ freezing, though costly, produces the best undisturbed samples of sands. Block samples, when they can be obtained, are superior to tube samples, but unavoidably have gone through an unloading stress path prior to sampling.

Fifty-four years ago, Hvorslev (1949) published his treatise on subsurface exploration and sampling of soils, which remains the standard reference for field investigations of this kind. Hvorslev's accomplishment was remarkable, as it is rare for a manual of technique to remain valid for half a century.

Gaps in Our Knowledge

Engineers are currently remediating earth dams to prevent damaging effects of potential earthquake shaking, with most of this remediation work taking place in the last two decades. The available seismic remediation and retrofit experience base is still growing. The current state of knowledge is deficient in at least four areas: soil properties, numerical techniques, construction techniques, and internal damage to the dam itself. There is also an urgent need for quantitative guidelines on site/material improvement. In situ permeability determination is particularly troublesome.

Soil Properties. All the problems associated with the determination of soil properties for the static and dynamic analysis of a dam exist when considering seismic remediation. In addition, the properties of the remediated or composite soils and their response to loading are not well understood.

Currently, we cannot accurately predict the residual strength of a liquefied deposit, and this significantly impacts the calculation of a post-earthquake deformed shape of a dam. Laboratory procedures have been established to determine the steady-state strengths of liquefied materials (Castro et al., 1985; Poulos, 1981). For these procedures to be useful, a good method for accurate (to within 0.01) and reliable in-situ void ratio determination is needed. Research in this area should be encouraged.

Numerical Techniques. No rehabilitated dam has yet been shaken by a strong earthquake. Thus, real verification is nonexistent. We have verified some of our numerical procedures using simplified problems and centrifuge experiments; however, case histories involving strong earthquakes and retrofitted dams are lacking. The static and dynamic interaction of the remediated material with the surrounding material is not well understood. Research needs to be conducted to determine how remediated "hard spots" in the foundation will interact with the dam during the earthquake.

There are currently at least four computer programs used in North America to predict large deformations: TARA-3, DYNAFLOW, FLAC, and DYNARD (Finn et al., 1986; Finn and Yogendrakumar, 1989; Prevost, 1981; Cundall, 1988; Cundall et al., 1988; Roth, 1985; Moriwaki et al., 1988). These computer programs are sophisticated, complex, and difficult to use. More importantly, the exact values of the deformed shape predicted by these codes are not trusted. Instead, they provide insight regarding the potential levels of deformation and internal damage to the embankment. Since remediation will limit seismic deformation (1 m or so), the static equilibrium of the calculated post-earthquake deformed shape is checkable by conventional stability analysis. However, a conventional slope stability analysis alone, with its rigid body assumptions, will not give an adequate picture of damage to the embankment. Today's use of the large deformation codes is mostly in this context. More verification and validation are needed.

Technical guidance needs to be developed so that input parameters to such models can be estimated from commonly used field procedures such as cone penetration test, Becker hammer test, and Standard Penetration Test (SPT). A remediation concept based on very large freeboard (i.e., 15 m) from either a lowered pool or added embankment (in order to have sufficient remaining embankment to retain the reservoir in the presence of large movements) would require a more accurate prediction of large deformations and resulting internal damage than can be done today.

Construction Technology. Current methods for in-situ densification are sometimes inadequate. For example, vibro-flotation works reasonably well in clean sands but not so well if the fines content is high. Dynamic compaction is limited to near-surface materials. Conventional grouting has not been a viable option, largely because engineers lack the ability to verify where the grout goes and resulting composite strengths and dynamic behavior. Jet and compaction grouting have been used. Research into construction techniques and field verification of what has been achieved regarding rehabilitation methods is encouraged.

Internal Damage. When an earth dam is subjected to seismic shaking or to fault movement in the dam's foundation with resulting large deformation, our ability to predict internal damage to the dam is poor. We do not know the effects of this internal damage and cracking on the defensive measures against piping that have been designed and built into the dam. Research on this topic is nonexistent. Today's remediation practice requires the consideration of the possibility that some of these defenses might be breached (i.e., How wide a transition or filter zone is wide enough?).

WHERE WE ARE GOING

It was Warren Buffet, we believe, who said, "The stock market presents a much clearer view thru the rear view mirror than thru the windshield." The same can be said for any subject, though even the past is not always seen with crystal clarity. Nevertheless, we can observe some trends, and assume that we can project them some distance into the future.

Dynamic Stress and Deformation Analysis

In the future, effective stress, time domain approaches will dominate our design and analysis methods. This is because these methods account for the time varying intensity of ground shaking, and provide insight as to where, when and in what order elements of a dam or foundation respond to excitation. These improved methods will model, in three dimensions, the entire sequence of events, including the construction, operation, maintenance, initiation of

earthquake shaking, liquefaction and final configuration of the dam-foundation-reservoir system. This will require improvements to be made in our modeling capability so that areas such as wing walls and wrap around sections can be more rigorously analyzed. These developments will parallel the structural research needed to develop and implement Performance Based Structural Design. As stated earlier, computer/numerical methods have and will continue to outstrip our advances in soil property input parameter determination.

Site Characterization

The CPT, other in-situ tests, and geophysical methods will become the tools of choice for site characterization, with sampling at critical locations identified by those methods. In general, the conduct of SPT, soil sampling, and lab testing will be limited to those critical locations and to sites with special problems. Much progress has been made on CPT tools and investigative techniques. The future CPT likely will carry multiple geophysical/mechanical tools, e.g., some form of the pressuremeter incorporated into a CPT rod string could also be used for measurement of soil mechanical properties. We believe that the key to improved spatial resolution in geophysical surveys will be development of ways to emplace dense arrays of sensors on the ground surface and underground, and use of tomographic methods to analyze the data.

Remediation

The most urgent and critical need in the area of embankment dam retrofit is for quantitative guidelines for soil improvement. We know that soil improvement works. That is, the soil improvement process improves the subsurface conditions. Currently index tests such as the SPT are used before and after soil improvement to qualitatively evaluate the work. We have no case history data to rely on as no rehabilitated embankment dam has been shaken be the design earthquake. Because this requirement is urgent, these guidelines will be developed in the next decade.

Earthquake Ground Motions

Seismologists will improve our design input motions. In addition to using empirically based attenuation relationships, we will develop more realistic design motions through improved understanding of the physics, the source and source mechanism, the travel path, and the local site conditions.

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