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# Earthquake-Resistant Design of Earth Dams

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SYNOPSIS Lessons gained from observations of the field performance of earth dams during earthquakes are reviewed and used to illustrate the primary problems of concern. Defensive design measures which may be taken to mitigate the various hazards are reviewed and illustrated. Analytical approaches for evaluating seismic stability and the deformations of earth dams during earthquakes are discussed, together with recent developments which facilitate their implementation in special cases; situations which require careful consideration of special effects such as the three-dimensionality of the dam-valley system and pore pressure re-distribution following an earthquake are discussed and illustrated.

## INTRODUCTION

Since the near-failure of the Lower San Fernando Dam during an earthquake just north of Los Angeles in 1971--an event which necessitated the immediate evacuation of over 80,000 people whose lives were endangered--the design of earth dams to resist earthquake effects has assumed a position of much greater significance among design engineers. Prior to this event it was generally believed that earth dams were inherently resistant to earthquake shaking, but the major slide in the Lower San Fernando Dam (see Fig. 1), which involved the upstream shell, the crest of the dam, and 30 ft of the downstream slope, with a resulting loss of 30 ft of freeboard, provided dramatic evidence that this is not necessarily so. As a result, regulatory agencies became more stringent in their requirements for demonstration of adequate seismic stability, and design engineers responded by developing new and more convincing design approaches than had previously been used. Thus the past 10 years have seen a major change in interest and attitude towards this aspect of design.

The problem is not of limited interest. Many parts of the world are subjected to the potentially hazardous effects of earthquakes and stringent earthquake-resistant design criteria have already been adopted for the design of earth dams in many different countries.

As in all aspects of geotechnical engineering, the initial starting point in developing an understanding of the problem lies in observations of the field performance of structures during actual earthquakes. The significant lessons to be gained from such studies are summarized briefly in the following section.

## LESSONS FROM FIELD PERFORMANCE OF DAMS DURING EARTHQUAKES

The sequence of events associated with the performance of Hebgen Dam in the Hebgen Lake earthquake of 1959 first brought to the attention of design engineers the wide variety of damage which may result from earthquake shaking (Sherard, 1967). The various possibilities are listed in Table 1 and virtually all of these, with the exception of sliding along the base, were evidenced to some degree in the behavior of Hebgen Dam during this particular earthquake. A careful study of the behavior of this dam is an object lesson in earthquake-resistant design for engineers interested in this field.

TABLE 1. Possible Ways in which an Earthquake may Cause Failure of an Earth Dam

1. Disruption of dam by major fault movement in foundation
2. Loss of freeboard due to differential tectonic ground movements
3. Slope failures induced by ground motions
4. Loss of freeboard due to slope failures or soil compaction
5. Sliding of dam on weak foundation materials
6. Piping failure through cracks induced by ground motions
7. Overtopping of dam due to seiches in reservoir
8. Overtopping of dam due to slides or rockfalls into reservoir
9. Failure of spillway or outlet works

Other studies of failures and non-failures of dams shaken by strong earthquake motions also provide valuable insights into types of behavior, however (Seed et al., 1978). Of major importance for example is a review of the performance of earth dams which existed in close proximity to the San Andreas Fault in the San Francisco earthquake of 1906. At that time there were 33 dams within 35 miles of the fault (and 15 within

Fig. 1. View of Lower San Fernando Dam after Upstream Slope Slide in Earthquake of Feb. 9, 1971

five miles of the fault) on which a magnitude 8-1/4 earthquake occurred, so there can be little doubt that all of these dams were subjected to strong shaking for a prolonged period of time (over 1 min). Based on recent correlations of ground motions with distance in California earthquakes, it seems reasonably sure that all of these dams were subjected to ground motions having peak accelerations greater than 0.25g and those within five miles of the fault to motions with peak accelerations greater than about 0.6g. Yet significantly none of these old dams suffered any significant damage and there was certainly no evidence of slope instability. It is not possible to attribute this to the use of flat slopes for the dams (slopes varied typically from 1 on 2 to 1 on 3) or to the high quality of construction (most of the dams were not compacted with rollers but by moving livestock or by teams and wagons). However, a significant characteristic of all of the dams is that they were constructed of clayey soils on rock or clayey soil foundations. Only two of the dams were built largely of sand and for these structures the sand was apparently not saturated. It is reasonable to conclude therefore that dams built of clayey materials seem to exhibit high resistance against slope failures during earthquakes--but the field data provide no information on the possible behavior of saturated sands.

This important conclusion is reinforced by the study by Akiba and Semba (1941) of the performance of dams in the 1939 Ojika earthquake in Japan. As a result of this earthquake 12 cases of complete dam failures occurred, together with about 40 cases of reported slope failures. The main conclusions of this study were:

(a) There were very few cases of dam failures during the earthquake shaking, most of the failures occurring either a few hours or up to 24 hours after the earthquake.

(b) The majority of the damaged and failed embankments consisted of sandy soils and no complete failures occurred in embankments constructed of clay soils.

(c) Even at short distances from the epicenter, there were no complete failures of embankments constructed of clay soils; however, at greater epicentral distances there was a heavy concentration of completely failed embankments composed of sandy soils.

It is clear that these observations tend to confirm those concerning dams built of clay soils in the 1906 San Francisco earthquake. In addition, the Ojika earthquake experience provides clear evidence of the vastly superior stability of embankments constructed of clay soils under strong seismic loading conditions over those constructed of saturated sands--a fact fully in accord with Terzaghi's insightful considerations on this question. Moreover, the performance record clearly suggests that the critical period for an embankment dam subjected to earthquake shaking is not only the period of shaking itself, but also a period of hours following an earthquake, possibly because piping may occur through cracks induced by the earthquake motions or because slope failures may result from pore pressure redistribution.

More recently, important earthquakes in which dam performance was observed include: the 1968 Tokachi-Oki earthquake in which a large number of slope failures occurred at shaking levels of the order of 0.2g in dams constructed of loose volcanic sand (Moriya, 1974) and the 1971 San Fernando earthquake in which slope failures occurred in the Upper and Lower San Fernando Dams, both having sand shells (Seed et al., 1975b), while generally excellent performance was observed in 25 rolled earth fill dams at shaking levels between 0.2 and 0.4g (Seed et al., 1978). The latter event, together with the performance of hydraulic fill dams in Russia (Ambraseys, 1960), also showed that even dams constructed with hydraulically deposited sand shells can withstand levels of shaking up to about 0.2g from magnitude 6-1/2 earthquakes without detrimental effects.

The slide movements in the San Fernando Dams were of special importance since in both cases, field observations showed that the shaking induced by the earthquake caused a dramatic increase in pore-water pressure in the shells of the dams and, in the case of the Lower San Fernando Dam, a condition of liquefaction which led to a major slide resembling a flow slide, in the upstream shell. In both cases the slide movements were apparently associated with a loss of strength associated with these pore water pressure increases.

The general conclusions (Seed et al., 1978) which seem to follow from a close study of embankment dam performance during earthquakes are as follows

(a) Hydraulic fill dams have been found to be vulnerable to failures under unfavorable conditions and one of the particularly unfavorable conditions would be expected to be the shaking produced by strong earthquakes. However, many hydraulic fill dams have performed well for many years and when they are built with reasonable slopes on good foundations they can apparently survive moderately strong shaking--with accelerations up to about 0.2g from magnitude 6-1/2 earthquakes--with no harmful effects.

(b) Virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking, say with peak accelerations of about 0.2g with no detrimental effects.

(c) Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35 to 0.8g from a magnitude 8-1/4 earthquake with no apparent damage.

(d) Two rockfill dams have withstood moderately strong shaking with no significant damage and if the rockfill is kept dry by means of a concrete facing, such dams should be able to withstand extremely strong shaking with only small deformations.

(e) Dams which have suffered complete failure or slope failures as a result of earthquake shaking seem to have been constructed primarily with saturated sand shells or on saturated sand foundations and these types of dams require careful attention to ensure their seismic safety.

(f) Since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2g with no harmful effects, we should not waste our time and money analyzing this type of problem--rather we should concentrate our efforts on those dams likely to present problems either because of strong shaking involving accelerations well in excess of 0.2g or because they incorporate large bodies of cohesionless materials (usually sands) which, if saturated, may lose most of their strength during earthquake shaking and thereby lead to undesirable movements.

(g) For dams constructed of saturated cohesionless soils and subjected to strong shaking, a primary cause of damage or failure is the build-up of pore water pressures in the embankment and the possible loss of strength which may accrue as a result of these pore pressures. Methods of stability analysis which do not take these pore pressure increases and associated loss of strength into account are not likely to provide a reliable basis for evaluating field performance.

## DEFENSIVE DESIGN MEASURES

It may be noted that most of the potential problems which may develop as a result of earthquake action do not require analytical treatment but simply the application of commonsense defensive measures to prevent deleterious effects. Thus to prevent a dam being disrupted by a fault movement in the foundation may simply require the identification of potentially active faults and the selection of a site where such faults do not exist. Similarly the potential for settlement, slumping, or tectonic movements, all of which could lead to loss of freeboard, can be ameliorated by the provision of additional freeboard so that the loss of some portion would not have serious consequences. In short, many of the potentially harmful effects of earthquakes on earth and rockfill dams can be eliminated by adopting defensive measures which render the effects non-harmful. A list of such defensive measures would include the following (Seed, 1979):

(a) Allow ample freeboard to allow for settlement, slumping or fault movements.

(b) Use wide transition zones of material not vulnerable to cracking.

(c) Use chimney drains near the central portion of the embankment.

(d) Provide ample drainage zones to allow for possible flow of water through cracks.

(e) Use wide core zones of plastic materials not vulnerable to cracking.

(f) Use a well-graded filter zone upstream of the core to serve as a crack-stopper.

(g) Provide crest details which will prevent erosion in the event of overtopping.

(h) Flare the embankment core at abutment contacts.

(i) Locate the core to minimize the degree of saturation of materials.

(j) Stabilize slopes around the reservoir rim to prevent slides into the reservoir.

(k) Provide special details if danger of fault movement in foundation.

This list should not by any means be considered all-inclusive Occasionally special situations will require or provide the opportunity for unique defensive measures such as the double dam system which protects against release of water from the Los Angeles Dam. The dam itself is designed to withstand probably the strongest earthquake criteria ever established but in the very remote possibility of a release of water, the people living downstream are protected by a second dam half a mile downstream from the first which stores no water and is only required to function in the remote chance the main dam releases water. The space between the two dams is maintained as a park area. Many engineers may consider this resorting to extreme defensive measures but a combination of public pressures, political considerations and ready availability of both the downstream dam and park space dictated a highly acceptable solution to many concerned residents. I personally consider this system to provide one of the safest downstream environments of all dams with which I am acquainted.

On the other hand, situations may develop where, in spite of the utmost care in planning, actual events may change professional evaluation of the potential activity of faults. This would appear to be the case for the proposed Auburn Dam site in California. When plans for this dam were first developed in the 1960's, the area was considered seismically quiet and the dam-site immune from active faults. The occurrence of the Oroville earthquake in 1975 (State of California Department of Water Resources, 1977), some 60 miles to the north, led to a re-evaluation of this situation and the determination that potentially active faults exist very close to or possibly even across the proposed dam-site. Clearly this has led to a re-evaluation of the desirability of constructing a thin concrete arch dam at this location. Such occurrences point up the need for prudence in evaluating the potential seismicity of any dam site.

Defensive measures, especially the use of wide filters and transition zones, provide a major contribution to earthquake-resistant design and should be the first consideration by the prudent engineer in arriving at a solution to problems posed by the possibility of earthquake effects.

At the same time it is necessary to recognize that all reasonable steps should be taken to ensure that sliding such as that which occurred at the Lower San Fernando Dam, the Sheffield Dam or a number of dams in Japan does not invalidate the beneficial effects of defensive measures; and many members of the public, regulatory agencies and even leading dam engineers will readily acknowledge that relatively sophisticated analyses are warranted in many cases to provide guidance on the possibility of slide movements developing during earthquakes and their possible extent. Accordingly it is of interest to review the approaches taken to check on this possibility and evaluate their significance in the design procedure--if for no other reason than to provide some insight into their effects in producing safer designs. Early attempts in this direction are discussed in the following pages.

## PSEUDO-STATIC ANALYSIS PROCEDURES

For the past 40 years or more, the standard method of evaluating the safety of earth dams against sliding during earthquakes has been the so-called pseudo-static method of analysis in which the effects of an earthquake on a potential slide mass are represented by an equivalent static horizontal force determined as the product of a seismic coefficient, k, or $n_g$, and the weight of the potential slide mass, as illustrated in Fig. 2. Attempts by the author to determine the originator of this method have proved singularly unsuccessful but the earliest written version that I have found appears in a classical paper by Terzaghi (1950). Terzaghi described the method in the following words.

> "An earthquake with an acceleration equivalent $n_g$ produces a mass force acting in a horizontal direction of intensity $n_g$ per unit of weight of the earth. The resultant of this mass force, $n_gW$, passes like the weight W, through the center of gravity $O_1$ of the slice abc. It acts at a lever arm with length F and increases the moment which tends to produce a rotation of the slice abc about the axis O by $n_gFW$. Hence the earthquake reduces the factor of safety of the slope with respect to sliding from $G_s$, equation (1) to

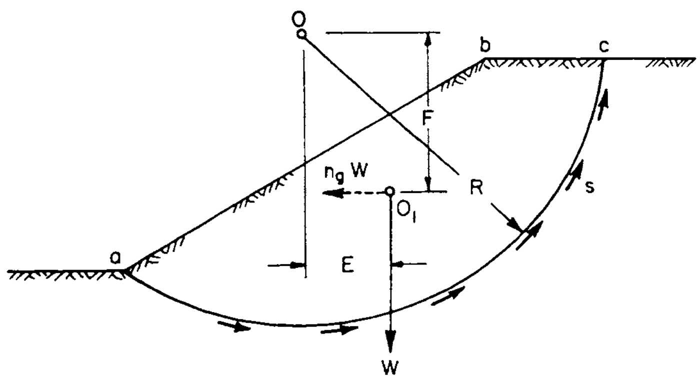


Fig. 2. Pseudo-static Method for Seismic Stability Analysis of Embankments (after Terzaghi, 1950)

$$G_s' = \frac{slR}{EW + n_g FW} \tag{2}$$

"The numerical value of $n_g$ depends on the intensity of the earthquake. Independent estimates (Freeman, 1932) have led to the following approximate values

| | |
|---|---|
| Severe earthquakes, Rossi-Forel scale IX | $n_g = 0.1$ |
| Violent, destructive, Rossi-Forel scale X | $n_g = 0.25$ |
| Catastrophic | $n_g = 0.5$ |

The earthquake of San Francisco in 1906 was violent and destructive (Rossi-Forel scale X), corresponding to $n_g = 0.25$.

"Equation (2) is based on the simplifying assumptions that the horizontal acceleration $n_g g$ acts permanently on the slope material and in one direction only. Therefore the concept it conveys of earthquake effects on slopes is very inaccurate, to say the least. Theoretically a value of $G_s' = 1$ would mean a slide, but in reality a slope may remain stable in spite of $G_s'$ being smaller than unity and it may fail at a value of $G_s' > 1$, depending on the character of the slope-forming material.

"The most stable materials are clays with a low degree of sensitivity, in a plastic state (Terzaghi and Peck, 1948, p. 31), dense sand either above or below the water table, and loose sand above the water table. The most sensitive materials are slightly cemented grain aggregates such as loess and submerged or partly submerged loose sand...."

In spite of Terzaghi's profound influence on virtually all aspects of soil mechanics, the seismic design of earth dams appears to have been one area where his advice went unheeded and literally hundreds of dams of all types have had their seismic stability evaluated using this approach coupled with seismic coefficients substantially smaller in magnitude than those which he advocated. In the United States, for example, seismic coefficients have typically ranged from 0.05 to 0.15 even in areas such as California where the strongest imaginable earthquakes may well occur; in Japan values have characteristically been less than about 0.2. Similar values have been used in highly seismic regions throughout the world, as shown by the design criteria listed in Table 2, and engineers were apparently convinced that such low values were all that were required to ensure an adequate level of seismic stability. No special consideration seems to have been given to the nature of the slope-forming or foundation materials and if the computed factor of safety was larger than unity, it has generally been concluded that the seismic stability question has been satisfactorily resolved.

This is certainly not in accordance with Terzaghi's conception of embankment behavior, specifically expressed in the words: "Theoretically a value of $G_s' = 1$ would mean a slide but in reality a slope may remain stable in spite of $G_s'$ being smaller than unity and it may fail at a value of $G_s' > 1$, depending on the character of the slope-forming materials." This statement clearly indicates that in Terzaghi's opinion a slope may be stable or unstable even if the computed factor of safety is greater than 1, but the optimistic position has generally held sway--that is, dams have been considered to have had adequate seismic stability so long as $G_s'$ remained equal to or greater than unity regardless of the nature of the slope-forming material.

A more detailed discussion of the reasons for the widespread use of this method has been presented elsewhere (Seed, 1979). A careful study of its merits, based on studies of earthquake-induced slides, shows that the method does not always predict failure, where failures have been found to occur, in embankments consisting of sandy soils or constructed on sandy foundations which show a marked loss of strength due to earthquake shaking (see for example, the failure of the Sheffield Dam in Fig. 3). Thus it cannot be used reliably for evaluating the possible performance of these types of dams (Seed et al., 1967, 1973).

On the other hand, based on a method of analyzing embankment deformations suggested by Newmark (1965) and applicable to soils which show no significant loss of strength due to earthquake shaking, (usually clayey soils, dry sands and some very dense cohesionless materials), it may be shown that (in cases where the crest acceleration does not exceed about 0.75g), deformations of such embankments will usually be acceptably small if the embankment can be shown to have a factor of safety of about 1.15 in a pseudo static analysis performed using a seismic coefficient of 0.15 (Seed, 1979).

This means that in deciding whether or not to use the pseudo-static method of analysis in any given case, the critical decision to be made by the design engineer is whether the soil is likely to be vulnerable to excessive strength loss or pore pressure development or not. This can be determined by tests. However, both field and laboratory experience indicates that clayey soils, dry sands and in some cases dense saturated sands will not lose substantial resistance

TABLE 2. Design Criteria for Selected Earth Dams (ICOLD Report)

| Dam | Country | Horizontal Seismic Coefficient | Minimum Factor of Safety |
|---|---|---|---|
| Aviemore | New Zealand | 0.1 | 1.5 |
| Bersemisnoi | Canada | 0.1 | 1.25 |
| Digma | Chile | 0.1 | 1.15 |
| Globocica | Yugoslavia | 0.1 | 1.0 |
| Karamauri | Turkey | 0.1 | 1.2 |
| Kisenyama | Japan | 0.12 | 1.15 |
| Mica | Canada | 0.1 | 1.25 |
| Misakubo | Japan | 0.12 | -- |
| Netzahualcoyote | Mexico | 0.15 | 1.36 |
| Oroville | USA | 0.1 | 1.2 |
| Paloma | Chile | 0.12 to 0.2 | 1.25 to 1.1 |
| Ramganga | India | 0.12 | 1.2 |
| Tercan | Turkey | 0.15 | 1.2 |
| Yeso | Chile | 0.12 | 1.5 |

Fig. 3. Failure of Sheffield Dam in Santa Barbara Earthquake of 1925

to deformation as a result of earthquake or simulated earthquake loading and thus pseudo-static analyses will generally provide an acceptable method of ensuring adequate performance for embankments constructed of these types of soil. In cases of doubt, however, a careful laboratory study will invariably provide the information from which an appropriate engineering decision concerning the applicability of the method can be made. It should also be noted that even some soils which might be vulnerable to the development of large pore pressures and some strength loss under conditions of strong shaking may show little evidence of these effects under less intense shaking, in which case the principles discussed above would still be applicable.

The Newmark (1965) type of analysis leading to evaluations of slope displacements by double integration of that portion of the induced acceleration response of a potential slide mass exceeding the computed yield acceleration for the mass (see Fig. 4), represented a major step forward in both the philosophy of evaluating embankment performance during earthquakes and in suggesting the means for its implementation. It is an interesting by-product that the use of this improved method based on the prediction of embankment displacements should ultimately lead to the conclusion that for some types of soils and conditions (i.e., those which do not build up large pore pressures or cause significant strength loss due to earthquake shaking and

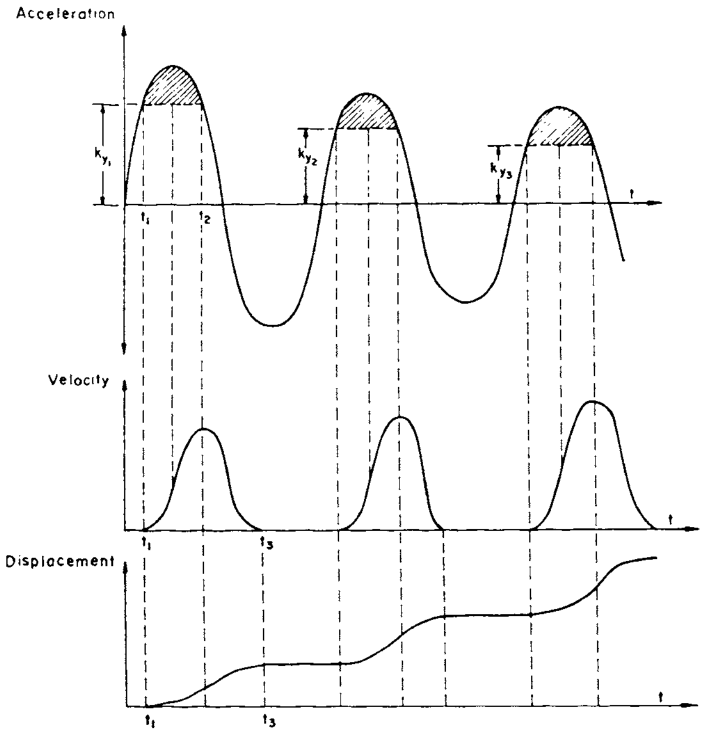


Fig. 4. Newmark Method of Evaluating Embankment Displacements

associated displacements), the old pseudo-static analysis often did an adequate job of limiting displacements in the first place and thereby provided, in many cases, an entirely adequate design procedure.

BEHAVIOR OF SATURATED COHESIONLESS SOILS

In contrast to the behavior of clayey soils and very dense cohesionless soils, loose to medium dense cohesionless soils do not exhibit any clearly defined yield strength and their behavior is complicated by the possibility of large pore pressure build-ups and redistribution during and following an earthquake. Similarly the stress-strain relationships for a sample of loose sand, before and after cyclic loading may be quite different.

In materials of this type, the pore pressure build-up during cyclic loading clearly affects the resistance to deformation, both under further cyclic stresses and under static stresses. It is difficult to establish a well-defined value of yield stress for such soils and both the generation and the redistribution of the large pore pressures developed during cyclic loading will affect the seismic stability, the induced deformations and possibly the post-earthquake behavior. This was made readily apparent by the failure of the Sheffield Dam in 1925 and the subsequent analysis of its stability (Seed et al., 1969). Predicted deformations, neglecting any pore pressure build-up would have been only a few inches, yet the dam failed completely. Similar results were subsequently observed in analyses of the Upper and Lower San Fernando Dams in the 1971 San Fernando earthquake. Accordingly, an analysis technique is required which takes into account the pore pressures generated by the earthquake shaking and their potential effects. A procedure for effecting this is described below.

SEED-LEE-IDRISS ANALYSIS PROCEDURE

In recognition of the limitations of the pseudo-static analysis approach and the difficulties of evaluating a yield stress criterion for many saturated cohesionless soils, the writer developed an alternative approach to the evaluation of deformations in earth dams (Seed, 1966). The details of the general procedure have undergone many improvements since that time (Seed et al., 1975a) primarily through the development and application of finite element procedures with the aid of Doctors I. M. Idriss, J. M. Duncan, F. I. Makdisi, N. Serff, J. R. Booker, M. S. Rahman, and Professor W. D. L. Finn, but also through the development of improved testing procedures developed mainly with the help of Professor K. L. Lee, and Doctors P. DeAlba, R. M. Pyke and N. Banerjee.

In spite of the improvements, however, the basic principles of the procedure have remained unchanged and involve a series of steps which might be summarized simply as follows:

(a) Determine the cross-section of the dam to be used for analysis.

(b) Determine, with the cooperation of geologists and seismologists, the maximum time history of base excitation to which the dam and its foundation might be subjected.

(c) Determine, as accurately as possible, the stresses existing in the embankment before the earthquake; this is probably done most effectively at the present time using finite element analysis procedures.

(d) Determine the dynamic properties of the soils comprising the dam, such as shear modulus, damping characteristics, bulk modulus or Poisson's ratio, which determine its response to dynamic excitation. Since the material characteristics are nonlinear, it is also necessary to determine how the properties vary with strain.

(e) Compute, using an appropriate dynamic finite element analysis procedure, the stresses induced in the embankment by the selected base excitation.

(f) Subject representative samples of the embankment materials to the combined effects of the initial static stresses and the superimposed dynamic stresses and determine their effects in terms of the generation of pore water pressures and the development of strains. Perform a sufficient number of these tests to permit similar evaluations to be made, by interpolation, for all elements comprising the embankment.

(g) From the knowledge of the pore pressures generated by the earthquake, the soil deformation characteristics and the strength characteristics, evaluate the factor of safety against failure of the embankment either during or following the earthquake.

(h) If the embankment is found to be safe against failure, use the strains induced by the combined effects of static and dynamic loads to assess the overall deformations of the embankment.

(i) Be sure to incorporate the requisite amount of judgment in each of steps (a) to (h) as well as in the final assessment of probable performance, being guided by a thorough knowledge of typical soil characteristics, the essential details of finite element analysis procedures, and a detailed knowledge of the past performance of embankments in other earthquakes.

This procedure may seem rather long and cumbersome but it also seems to incorporate the essential steps in evaluating such a complex problem as the response of earth dams to earthquake effects.

It lends itself naturally, however, to somewhat simplified versions of the method, which have often been used for reason of time and economy (e.g., Finn, 1967; Klohn et al., 1978; Lee and Walters, 1972; Lee, 1978; Leps et al., 1978a and b; Vrymoed and Galzacia, 1978; etc.). The ultimate simplification is, of course, the total elimination of all analysis procedures and a simple evaluation, based on a knowledge of the materials comprising the dam and the judgment resulting from conducting many previous analyses and observing the performance of existing dams. However, it should be noted that each of the steps is an essential element of the procedure and if one of them is performed incorrectly, the results of the analysis may be grossly misleading. In such cases, where the job cannot be done properly, it may be better not to do it at all rather than to be misled by the erroneous results which may ensue. It is for this reason that judgment is necessary at each step in the development.

In the most modern versions of the method, the assessment of pore water pressures during and following the earthquake shaking may involve studies of simultaneous pore pressure generation and dissipation using appropriate computer programs (Booker et al., 1976; Finn et al., 1978) and the evaluation of the final configuration of the structure using a strain-harmonizing technique, again involving finite element procedures (Lee et al., 1974; Serff et al., 1976).

The particular procedure used in any given case should depend on the complexity of the case being considered, the margin of safety provided for the level of earthquake shaking likely to develop, and the judgment and experience of the engineer responsible for the study.

An interesting example of the use of this method to analyze a slope failure is provided by the analysis of the Lower San Fernando Dam (Seed, 1979). The computed response of this dam to the earthquake ground motions is shown in Fig. 5 with the dark area indicating the zones where the residual pore water pressure at the conclusion of the earthquake was equal to 100%. A stability analysis of this section, shown in Fig. 6, clearly indicates that for these conditions a slope failure would develop.

An example of a seismic stability analysis where failure did not occur is illustrated by the computed response of the Upper San Fernando Dam in the same earthquake. Again extensive zones of high pore-water pressure were developed within the embankment as a result of the earthquake shaking but they were not sufficiently extensive to cause a failure, as illustrated in Figs. 7 and 8. In this case the embankment suffered significant deformations, the crest moving downstream about 5 ft. Deformation analyses based on the method described above predicted a movement of about 3.8 ft.

As with all analytical procedures used in geotechnical engineering, the method should only be used if it is found to work--that is, if it provides reasonable evaluations of behavior for cases where the behavior has been or can be observed. In all, the general procedure described above has been used to study the performance of eight dams whose performance during earthquakes is known. Two of these had major slides, one underwent large deformations, one underwent small deformations and four had no discernible damage. The behavior predicted by the analysis was similar to that observed in the field in each case, and while it is true that each of these cases was in fact studied after the event involved, it seems that the procedure has the capability of giving considerable insight into the possible behavior of embankments subjected to earthquake effects. For this reason, presumably, it has been adopted in studies of many dams throughout the world as a guide to final assessment of their probable performance during earthquakes (e.g., Seed et al., 1969, 1973, 1975b; Gordon et al., 1974; Kramer et al., 1975; Marcuson et al., 1977; Makdisi et al., 1978; Sadigh et al., 1978; State of California Department of Water Resources, 1979; etc.).

If the permeability of the shell material for a dam becomes sufficiently high, say of the order of the 1 cm/s, then it may be impossible for an earthquake to cause any build-up of pore pressures in the embankment since the pore pressures can dissipate by drainage as rapidly as the earthquake can generate them by shaking. A good example of such a situation is the upstream shell of Dartmouth Dam (Seed, 1974). This 650 ft high rock fill structure (see cross-section in Fig. 9) has highly pervious shells having a 10% size of about 2.5 cm and a permeability coefficient possibly of the order of 100 cm/s. In such a case analysis shows that the pore pressure build-up in 10 s of earthquake shaking would be a negligible proportion of the initial effective overburden pressure (Seed, 1979) even for an earthquake which might hypothetically be considered to produce shaking of sufficient intensity to generate a pore pressure ratio of 100% throughout the shell if it were undrained. Clearly in a case such as this, the upstream shell can for practical purposes be considered to be fully drained during any reasonable period of earthquake shaking and stability can be evaluated on this basis.

A similar conclusion has been drawn in the evaluation of the seismic stability of the Watauga

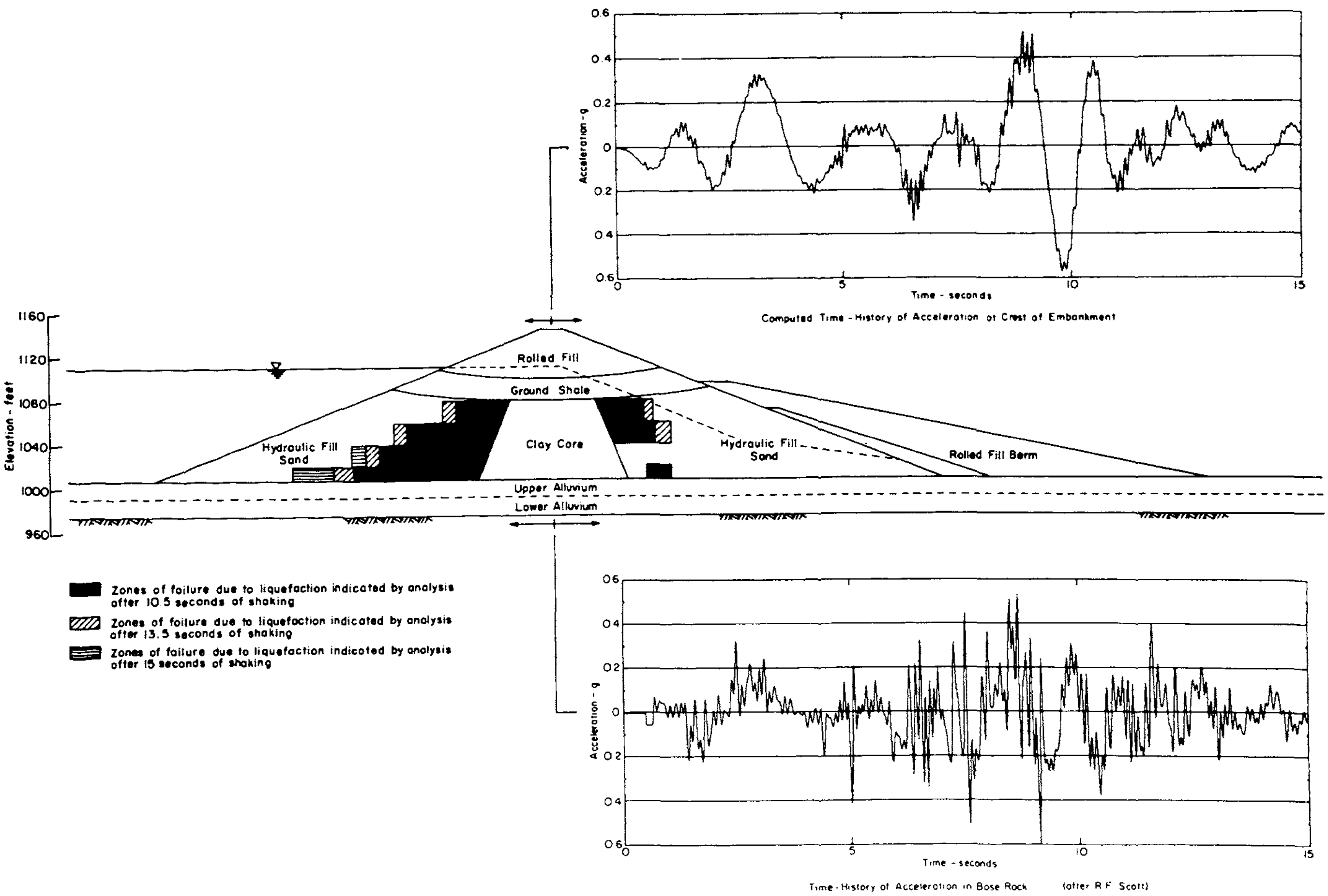


Fig. 5. Analysis of Response of Lower Dam During San Fernando Earthquake to Base Motions Determined from Seismoscope Record

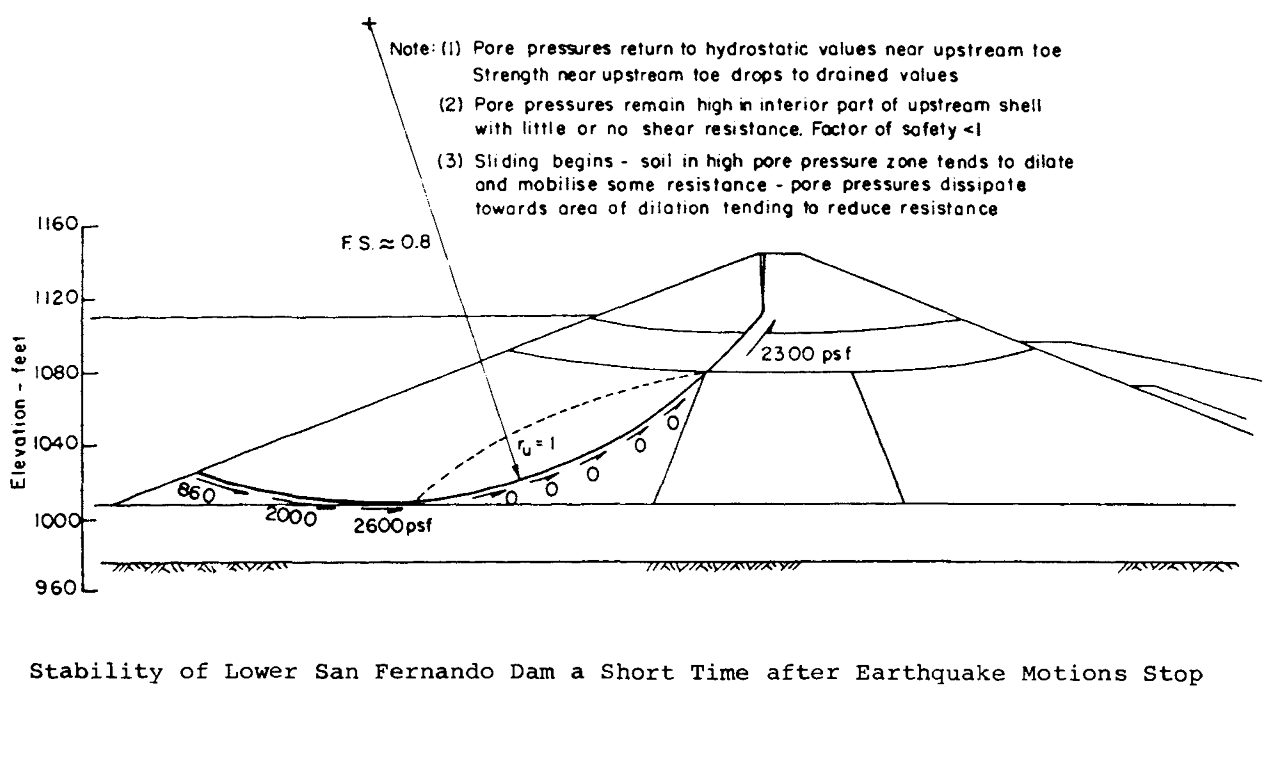


Fig. 6. Stability of Lower San Fernando Dam a Short Time after Earthquake Motions Stop

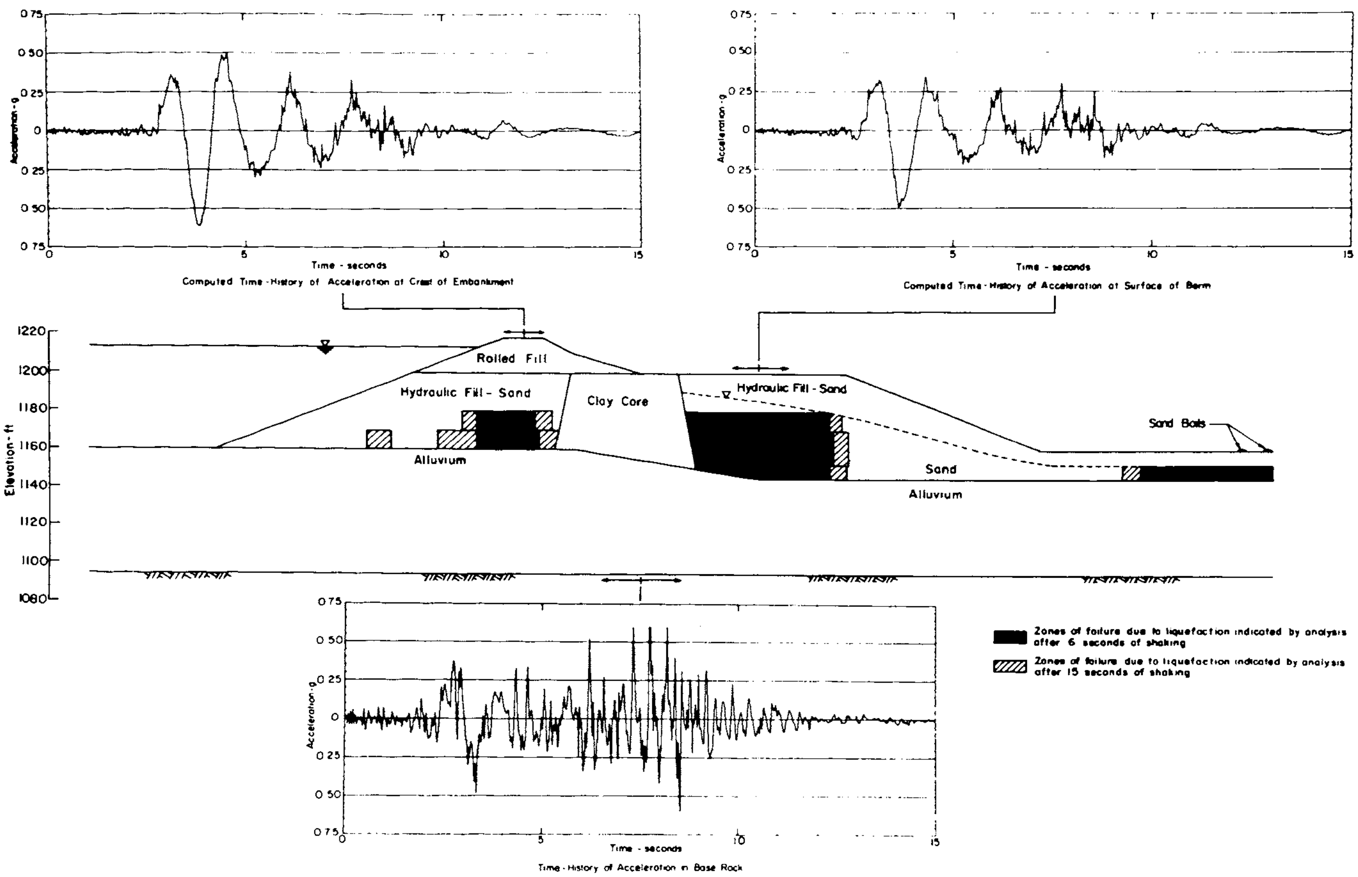


Fig. 7. Analysis of Response of Upper San Fernando Dam During San Fernando Earthquake

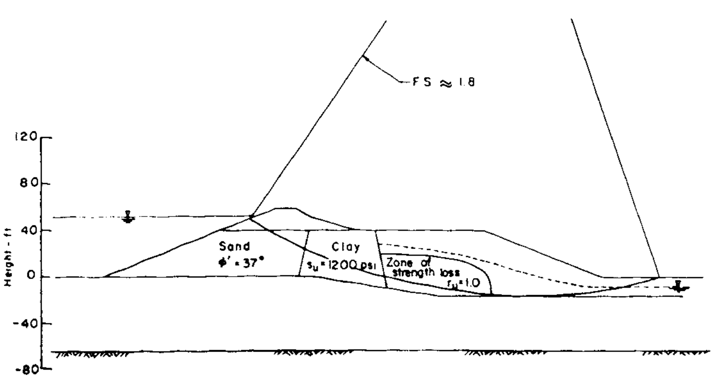


Fig. 8. Stability Analysis of Upper San Fernando Dam for Conditions Soon After Earthquake

Dam (Sadigh et al., 1978) and the high permeability of the shells can be considered a major factor in maintaining a high degree of seismic stability in all highly pervious rock fills.

NEW DEVELOPMENTS IN EVALUATIONS OF SEISMIC STABILITY OF EMBANKMENTS

During the past few years there have been several new developments in methods of evaluating the seismic stability of earth dams. These include the following:

1. Use of Penetration Test Data in Lieu of Cyclic Load Test Data

One of the most widely used methods of evaluating the liquefaction potential of a sand deposit under level ground conditions is to compare the cyclic shear stresses induced by an earthquake with those which, on the basis of field experience for soils of known penetration resistance, have or have not liquefied during earthquakes. The use of this procedure requires only the determination of the standard penetration resistance of a sand deposit, and avoids judgments on the part of the engineer concerning the representativeness of soil samples which might have been tested or the influence of sample disturbance on the results of such tests. The correlation charts most widely used in North America are shown in Figs. 10 and 11 (Seed, 1979) and

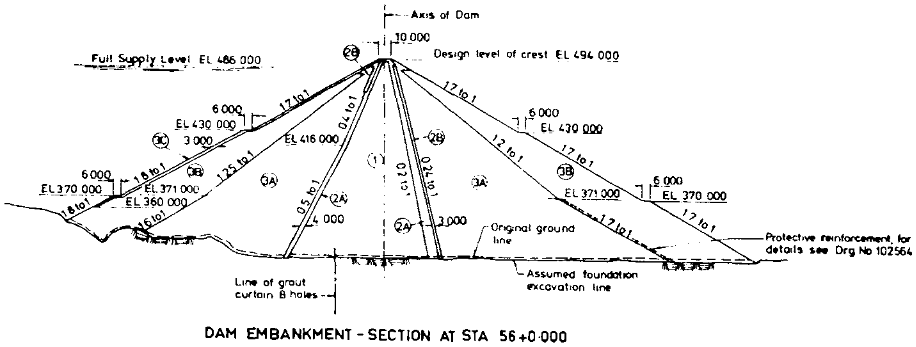


Fig. 9. Typical Cross-Section Through Dartmouth Dam

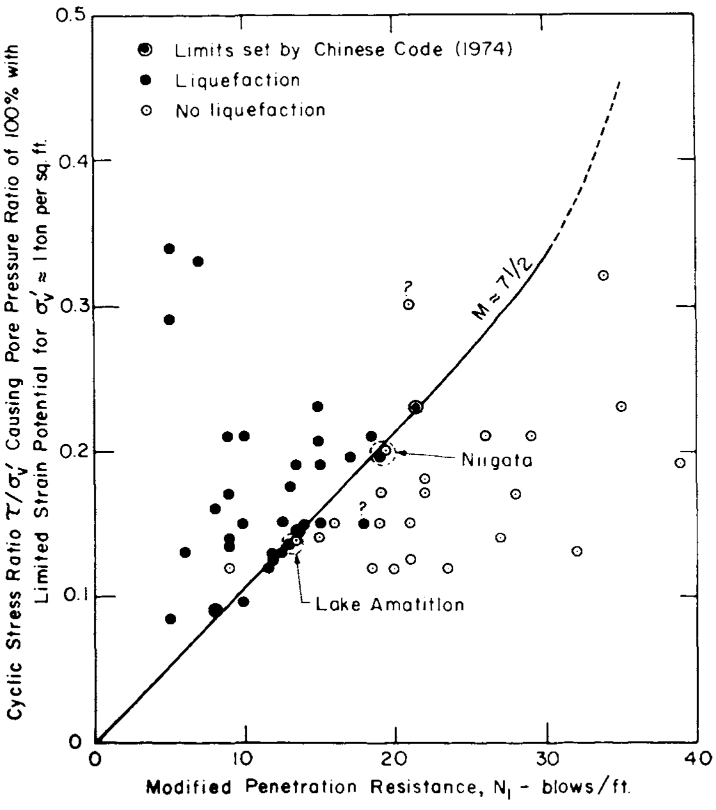


Fig. 10. Correlation Between Field Liquefaction Behavior of Sands for Level Ground Conditions and Penetration Resistance (Earthquakes with Magnitude ≈ 7-1/2)

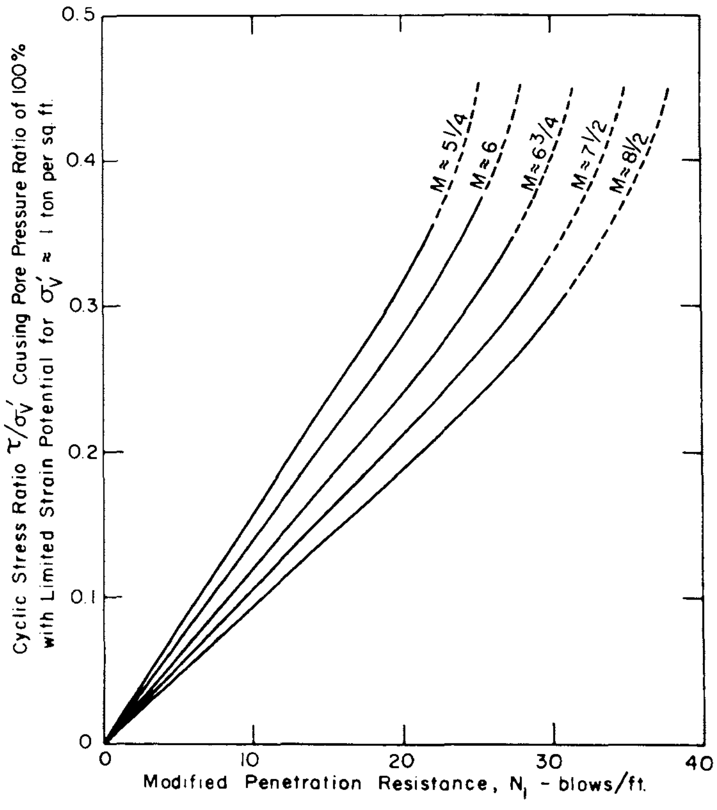


Fig. 11. Design Curves for Evaluating Field Liquefaction Resistance of Sands Under Level Ground from Standard Penetration Test Data

TABLE 3. Performance During Earthquakes of Five Dams with Sandy Shells and Central Clay Cores

(see material characteristics in Fig. 17)

| Dam | $(K_2)^*_{max}$ | Field Performance |
|---|---|---|
| Chabot | 65 | Survived magnitude 8-1/4 earthquake at distance of 20 miles with no apparent damage; would apparently survive magnitude 7 earthquake at distance of 2 miles. |
| Lower Franklin | 50 | Survived magnitude 6-1/2 earthquake at distance of about 20 miles ($a_{max} \simeq 0.2g$) with no apparent damage. |
| Fairmont | 50 | Survived magnitude 6-1/2 earthquake at distance of about 20 miles ($a_{max} \simeq 0.2g$) with no apparent damage. |
| Lower San Fernando | 45 | Major upstream slide including upper 30 ft of dam due to magnitude 6-1/2 earthquake at distance of about 5 miles ($a_{max} \simeq 0.55g$); performance and analysis indicate dam would have survived same earthquake at distance of 20 miles without significant damage but would have failed catastrophically for magnitude 8-1/4 earthquake at distance of 20 miles. |
| Upper San Fernando | 30 | Downstream slide movement of crest of dam due to magnitude 6-1/2 earthquake at distance of about 5 miles ($a_{max} \simeq 0.55g$); this embankment had an extremely wide downstream berm which together with its lower value of $(K_2)_{max}$ no doubt contributed to its better performance than the Lower San Fernando Dam. |

*Shear modulus $G = 1000\ K_2(\sigma_m')^{1/2}$

involves the use of a standardized penetration resistance $N_1 = C_N \cdot N$ where $C_N$ is a function of the overburden pressure at which the N-value is measured (see Fig. 12).

It has long been recognized that when sand is subjected to initial static shear stresses before earthquake loading, as naturally occurs under the sloping surfaces of an earth dam, the resistance of the sand to pore pressure generation is increased. Typical results showing the effect of an initial static stress on horizontal planes on the cyclic stress required to cause a pore pressure ratio of 100% in tests on simple shear samples are shown in Fig. 13. Similar results are obtained in cyclic loading triaxial compression tests conducted on samples consolidated initially under different principal stress ratios.

The form of the results shown in Fig. 13 is most useful for analysis purposes since they show the magnitude of the cyclic stress ratio $\tau_c/\sigma_o'$ which must be superimposed on the initial static stresses acting on a soil element in order to cause a pore pressure ratio of 100%, for different values of the initial static stress ratio $\alpha = \tau_{fc}/\sigma_{fc}$. Tests on a number of sands have shown that the value of $\tau_c/\sigma_o'$ under these conditions increases with the value of $\alpha$ approximately as shown in Fig. 14. Thus if the cyclic stress ratio required to cause a pore pressure ratio of 100% for the case where $\alpha = 0$ is known, the cyclic stress ratio causing the same pore pressure condition can be obtained from the relationship

$$\left(\frac{\tau_c}{\sigma_o'}\right)_{\ell-\alpha} = \left(\frac{\tau_c}{\sigma_o'}\right)_{\ell-\alpha=0} \times K_\alpha$$

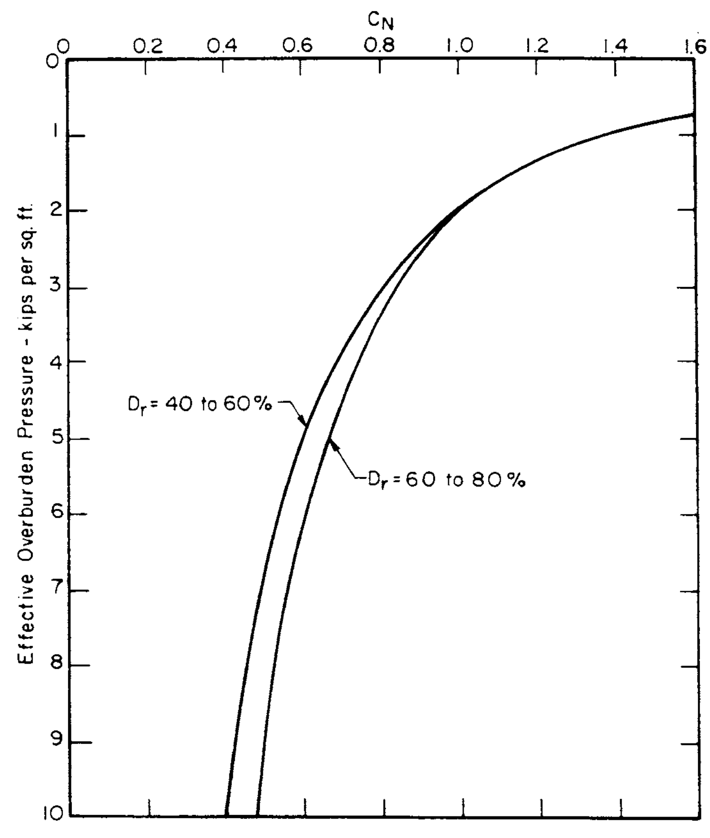


Fig. 12. Recommended Curves for Determination of $C_N$ Based on Averages for WES Tests

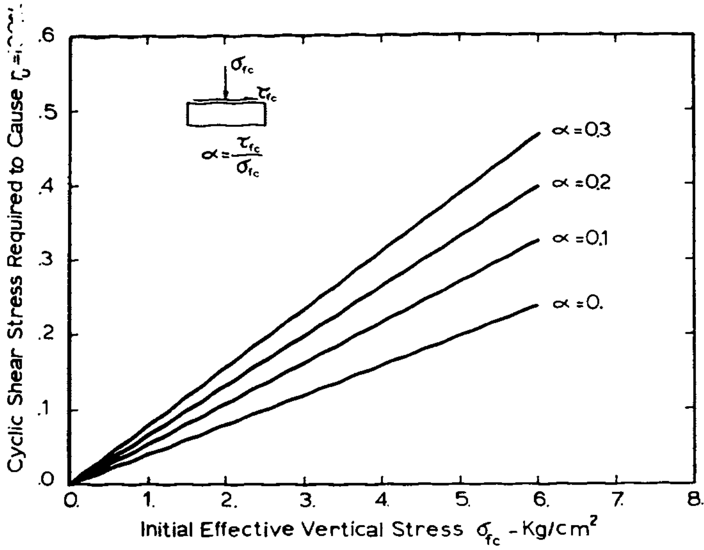


Fig. 13. Typical Effects of Initial Static Stresses on Cyclic Loading Resistance of Sands

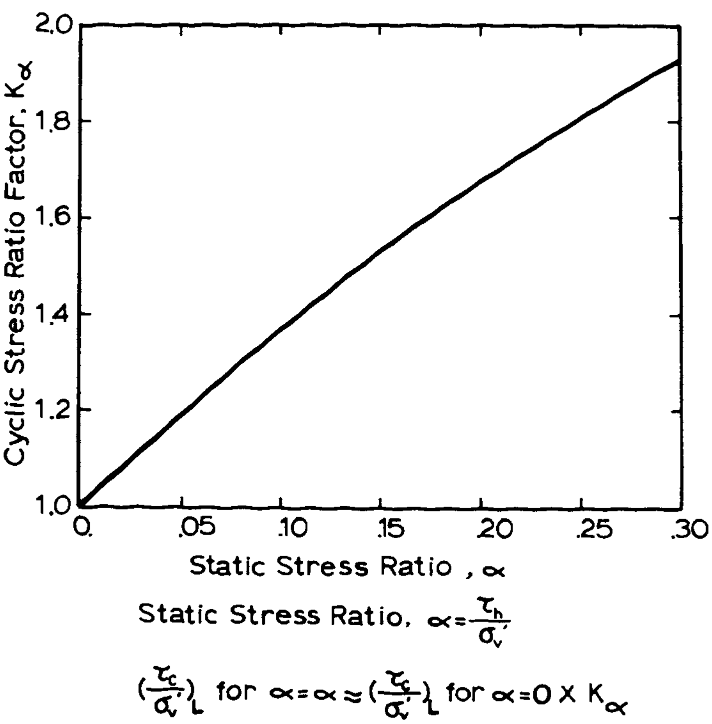


Fig. 14. Typical Chart for Evaluating Effect of Initial Cyclic Stresses on Cyclic Loading Resistance of Sands

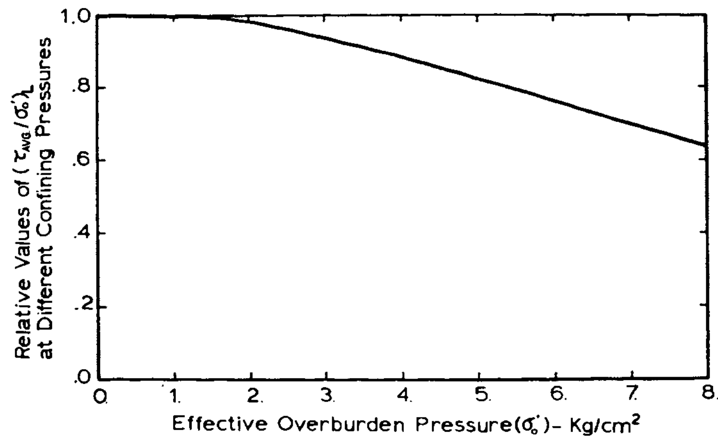


Fig. 15. Typical Reduction in Cyclic Stress Ratio Causing Liquefaction with Increase in Initial Confining Pressure

where $K_\alpha$ can be read off directly from Fig. 14. Since the values of $(\tau_c/\sigma_o')_{\ell - \alpha = 0}$ can be determined from charts such as Fig. 11, relating this parameter to the results of the standard penetration test, based on experiences at sites where liquefaction has or has not occurred during earthquakes, this method provides a simple but direct means of relating past field performance to probable future performance under given earthquake loading conditions.

It should be noted, however, that charts such as those shown in Figs. 10 and 11 are based on sites where liquefaction has occurred under fairly small overburden pressures, typically less than 1.5 tons per sq ft and the charts are therefore applicable only to these conditions. However, as the overburden pressure on a soil increases, the cyclic stress ratio required to cause a pore pressure ratio of 100% decreases and therefore a correction must normally be made for this effect. Typically the reduction in cyclic stress ratio with increase in confining pressure approximates the values shown in Fig. 15.

Thus a complete procedure for obtaining the cyclic shear stress required to cause a pore pressure increase of 100% could involve only the following sequence of steps:

1. Determine the standard penetration resistance, N, of the sand at a given point of interest in a dam or its foundation.

2. Convert the measured N value to the value it would have at a confining pressure of 1 ton/sq ft using the relationship

$$N_1 = C_N \cdot N$$

where $C_N$ is a function of the effective overburden pressure as shown in Fig. 12.

3. From the value of $N_1$ thus determined, read off the corresponding value of the cyclic stress ratio required to cause a pore

pressure ratio of 100% for cases where $\alpha = 0$ from charts such as that shown in Fig. 11.

4. Determine the pre-earthquake static stresses $\tau_{hv}$ and $\sigma_v'$ at the point under consideration. Hence determine the value of $\alpha = \tau_{hv}/\sigma_v'$ for the point.

5. From Fig. 14, determine the value of $K_\alpha$ corresponding to the known value of $\alpha$ and hence determine the value of

$$\left(\frac{\tau_c}{\sigma_v'}\right)_{\ell - \alpha} = \left(\frac{\tau_c}{\sigma_v'}\right)_{\ell - \alpha = 0} \times K_\alpha$$

6. If the effective vertical stress is greater than 1.5 tons/sq ft, reduce the value of $\left(\frac{\tau_c}{\sigma_v'}\right)_{\ell-\alpha}$ obtained in step 5 by the correction factors shown in Fig. 15.

7. For the final value of $\left(\frac{\tau_c}{\sigma_v'}\right)_{\ell - \alpha}$ obtained in step 6, determine the cyclic shear stress required to cause a pore pressure increase of 100% from the equation

$$\tau_c = \left(\frac{\tau_c}{\sigma_v'}\right)_{\ell - \alpha} \times \sigma_v'$$

The main advantages of this approach to cyclic resistance evaluation are that it is based on field performance data, it shows the variations in resistance from point to point, it avoids basing an analysis on a limited number of tests on hopefully representative samples and it avoids any issues associated with sample disturbance during the boring, transportation and handling of samples in the laboratory.

The main disadvantages of the approach are the well-known difficulties of evaluating the values of standard penetration test data in the light of the factors known to influence the test results: e.g., method of lifting the hammer, method of supporting the walls of the hole, diameter of the drill hole, length of the drill stem, type of anvil used in the striking mechanism, etc.

However, these limitations of the standard penetration test do not preclude a meaningful evaluation, and the directness of the method has a major appeal to many engineers. It has already been used on several dams for evaluating the characteristics of sand deposits in the foundation (Khilnani and Byrne, 1981; Wahler and Associates, 1981) and will no doubt be used more frequently in the future. A typical example is shown in Fig. 16, which shows the computed cyclic shear stresses induced by the design earthquake (Magnitude 6-1/2 at a distance of 24 miles) in the foundation deposits for the Camanche Dam in California and the resistance to cyclic loading determined from standard penetration test data following the procedure discussed above. It was concluded that the foundation sands had adequate capacity to support the dam, except near the toes of the embankment where soil liquefaction would cause minor sloughing of the outer parts of the shells but without any major damaging effects to the embankment.

2. Use of Comparative Procedures to Evaluate Seismic Stability

With the increasing number of dams for which (1) properties and performance under earthquakes of known intensities have been established and (2) properties and evaluations of seismic stability have been made, it is becoming possible to evaluate the probable performance of other dams by simple comparison with available performance or evaluation data for embankments constructed under similar conditions and with similar configurations. Thus no detailed analysis is required--simply a good file of past performance and analytical evaluation data.

Clearly this approach cannot be followed where variable and unusual foundation conditions dominate the response, but where embankments with well known characteristics are constructed on good foundations, this approach can provide a valid and inexpensive means of seismic stability evaluation.

Data for dams whose performance during earthquakes is known is shown in Table 3 and Fig. 17. Cyclic loading characteristics under standard conditions for a number of other dams whose stability has been found acceptable under conditions of strong earthquake shaking are shown in Fig. 18. These and other data can greatly simplify stability evaluations and will also find increasing use in the future.

3. Use of Three Dimensional Response Analyses

It has been conventional practice in the past to evaluate the dynamic response of an embankment by making plane-strain analyses of several representative sections through the dam. For cases where the valley width is several times the height this approach provides a good engineering approximation but when a high dam is built in a relatively narrow canyon, plane strain analyses can give grossly misleading results concerning embankment response. Thus while much effort is always directed towards simplification of the evaluation procedure, it is sometimes necessary to use more sophisticated analyses in the interests of improved understanding and economy.

An interesting case in point is the interpretation of the records of motion at the Oroville Dam in California which crosses a V-shaped valley and has a crest to height ratio of about 7. During the 1975 Oroville earthquake, records of earthquake-induced motions were made at the crest of the dam and at its base. These records can be used to determine the dynamic response characteristics of the cobble and gravel shells for this 750 ft high embankment. For such materials the shear moduli can be represented with a high degree of accuracy by an equation of the form

$$G = 1000\ K_2 \cdot (\sigma_m')^{1/2}$$

where $K_2$ is a material characteristic having a maximum value $(K_2)_{max}$ at very low strain levels of the order of $10^{-4}$ percent and $\sigma_m'$ is the effective mean principal stress. A plane strain analysis of the embankment response in this case would lead to about 100% error in evaluation of material characteristics compared

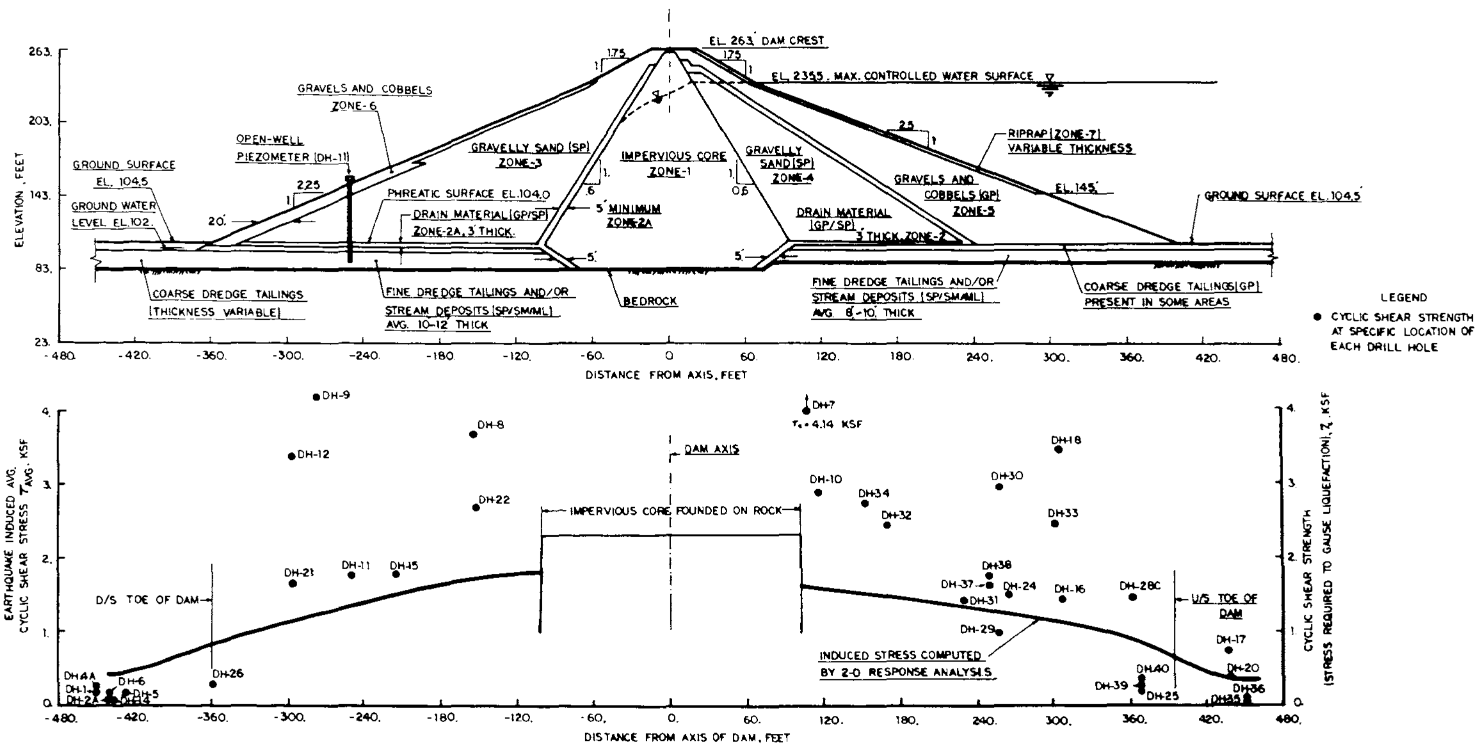


Fig. 16. Analysis of Stability of Foundation Sands at Camanche Dam Using Standard Penetration Test Data

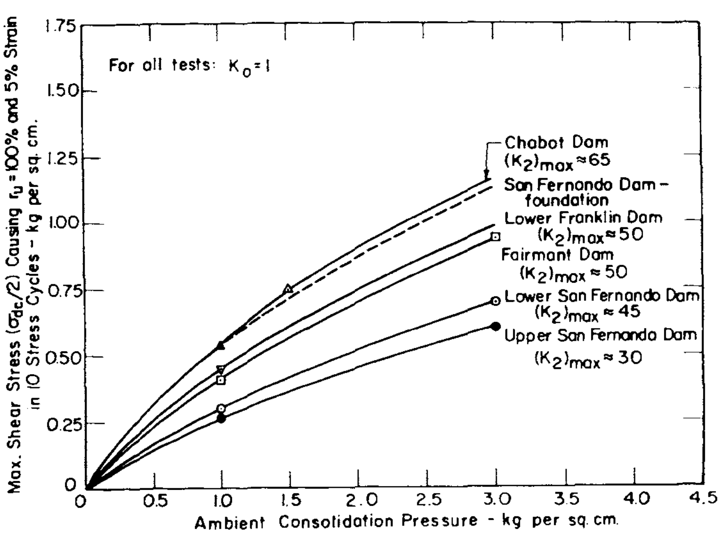


Fig. 17. Cyclic Loading Resistance of Shell Materials for Selected Dams (see Table 3)

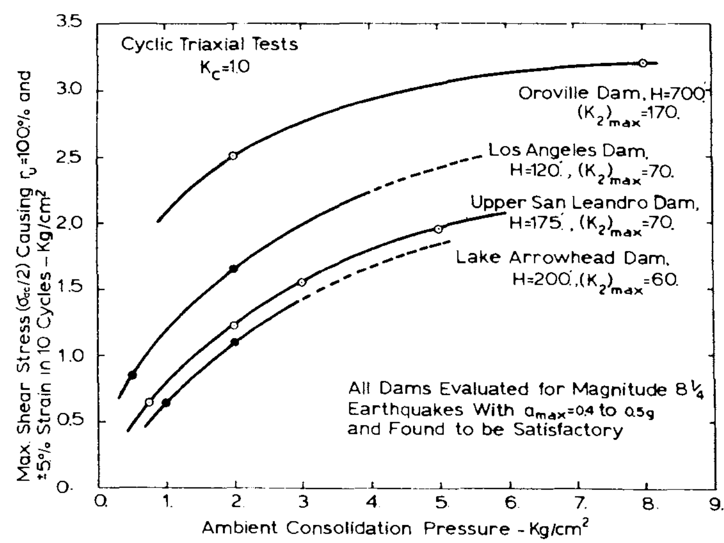


Fig. 18. Cyclic Loading Characteristics of Shell Materials for Several Dams Shown by Analyses to be Capable of Withstanding Strong Earthquake Shaking

with a three-dimensional evaluation of embankment response.

For high dams in narrow canyons three dimensional analyses of response are therefore a necessary tool in the development of improved understanding of embankment behavior and therefore of seismic stability evaluations.

4. Re-distribution of Pore Water Pressures and Post Earthquake Stability Evaluations

The delayed failure of one of the Mochi-Koshi tailings dams in the Near Izu-Oshima Japanese earthquake in 1978 (Marcuson et al., 1979) serves as a forceful reminder that the post-earthquake stability of an embankment dam may in some cases be as serious a problem as the during-earthquake stability. This condition results from a re-distribution of pore water pressures in the embankment following an earthquake, and while in many cases, this redistribution will have beneficial effects on stability it may in some cases have deleterious effects (Seed, 1979). Consideration of post-earthquake stability is thus now being considered an important aspect of seismic design and it imposes new and difficult demands on the skill of the design engineer.

## CONCLUSIONS

In the preceding pages I have tried to review the field experiences, design concepts, and analytical techniques currently available to assist the design engineer in making a reliable evaluation of the ability of a dam to resist the potentially damaging effects of earthquake shaking. Much progress in this field has been made during the past 10 years--both with regard to our understanding of seismicity and techniques for assessing the magnitude and locations of earthquakes--so that we can eliminate the surprises which so often have accompanied their occurrence in the past--and with regard to the development of design measures and embankment evaluation procedures which enable us to anticipate and mitigate the damaging effects of earthquakes.

It is important to keep the problem of earthquake-resistant design in perspective. Many parts of the world are not influenced significantly by these events and not all types of dams suffer major damage even if they are subjected to strong earthquake shaking. Thus the number of earthquake-induced failures of engineered dams is relatively small. However where the potential for strong earthquake shaking exists it is important that the possibility of failure be considered adequately and handled effectively even if it is only by a careful non-analytical assessment of hazards and the incorporation of design details to eliminate their effects. Analyses are a valuable supplementary tool to aid the engineer in achieving this goal. No matter how it is accomplished, however, the hazard potential of many dams makes it essential that adequate safety be achieved by one means or another.

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