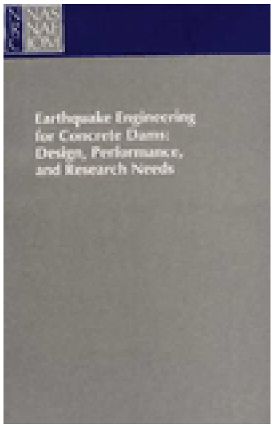

Earthquake Engineering for Concrete Dams: Design, Performance, and Research Needs

Panel on Earthquake Engineering for Concrete Dams, Committee on Earthquake Engineering, Division of Natural Hazard Mitigation, National Research Council

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Committee on Earthquake Engineering
Division of Natural Hazard Mitigation
Commission on Engineering and Technical Systems
National Research Council

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PANEL ON EARTHQUAKE ENGINEERING FOR CONCRETE DAMS

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Executive Summary

Concerns about the seismic safety of concrete dams have been growing during recent years, partly because the population at risk in locations downstream of major dams continues to expand and also because it is increasingly evident that the seismic design concepts in use at the time most existing dams were built were inadequate. To respond to these concerns, the Panel on Earthquake Engineering for Concrete Dams was appointed to serve under the NRC Committee on Earthquake Engineering. The mandate of the panel was to evaluate the present status of knowledge about the earthquake performance of concrete dams, including procedures for investigating the seismic safety of such structures, and to summarize its findings in a report. The report was intended specifically to inform research workers about the state of the art of earthquake analysis of concrete dams and to identify subject areas where additional knowledge is needed.

Questions about the safety of concrete dams were first brought into focus in this country by the failure of St. Francis Dam in California in 1928, which caused extensive property damage and the loss of over 400 lives. This disaster led to the formation of an agency of the State of California that would be responsible for dam safety. Now known as the Division of Safety of Dams of the California Department of Water Resources, this agency has jurisdiction over nearly 1,200 dams, and its operation during the past 60 years has served as a model for similar agencies in many other states and several other countries.

The hazard posed by large dams has been demonstrated since 1928 by the failure of many dams of all types and in many parts of the world. However, no *failure* of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failures in the foundation rock supporting the dams. On the other hand, two significant instances of earthquake *damage* to concrete dams occurred in the 1960s: Hsinfengkiang in China and Koyna in India. The damage was severe enough in both cases to require major repairs and strengthening, but the reservoirs were not released, so there was no flooding damage. This excellent safety record, however, is not sufficient cause for complacency about the seismic safety of concrete dams, because no such dam has yet been subjected to maximum conceivable earthquake shaking while retaining a maximum reservoir. For this reason it is essential that all existing concrete dams in seismic regions, as well as new dams planned for such regions, be checked to determine that they will perform satisfactorily during the greatest earthquake shaking to which they might be subjected.

Major factors that must be considered in verifying the seismic safety of existing or proposed concrete dams include definition of the expected seismic excitation and evaluation of the response of the structure to this input. Usually, the structural response is first calculated assuming that the dam is a linear system in which the displacements are directly proportional to the input excitation. To establish the ultimate resisting capacity, however, damage mechanisms and the resulting nonlinearities must also be considered. Observational evidence provides the best indication of the true performance of a dam in the nonlinear response range; hence, one important part of earthquake engineering for concrete dams is the evaluation of data from dynamic tests—real earthquake response information as well as laboratory and field test data. Finally, to ensure that proposed or existing concrete dams provide adequate safety, suitable performance criteria must be established against which the predicted earthquake performance of a dam can be judged. Discussions of all of these topics are presented in Chapters 2 through 6 of this report; a brief overview of each is contained in the following paragraphs.

EARTHQUAKE INPUT

To evaluate the expected earthquake performance of a concrete dam, it first is necessary to establish the intensity and frequency characteristics of the earthquake motions to which the dam might be subjected. Of course, these properties depend directly on the seismicity of the dam site—that is, upon the magnitude and epicenter location of the expected earthquake as well as its probable recurrence interval; however, these topics are not considered in this report. Instead, it is assumed here that the ground motion to be expected at the site is known, in the form of a strong-motion seismograph record, for example; thus, this report is concerned only with the expected performance of a specified dam when it is subjected to these given motions.

The first factor to be considered in a response analysis is how the prescribed earthquake motions are applied to the dam. In the early days of earthquake engineering it was always assumed that the structure was supported on a rigid base and that the earthquake input was applied as specified motions of this base. For lightweight relatively small structures supported on hard foundation rock, this input assumption was acceptable. However, for massive stiff concrete dam structures that are supported by very broad foundation rock surfaces, it is not reasonable to assume that the base support system is rigid. Clearly this assumption would not be consistent with the fact that an earthquake is actually a vibration wave being propagated through the earth's crustal structure. Thus, one of the principal questions to be answered with regard to the earthquake input is how to express the excitation in the context of seismic wave propagation.

The typical strong-motion earthquake record portrays three components of the acceleration history observed at a single point, usually at the ground surface in a free-field location (i.e., at a point where the motion is free of the vibratory influences of adjacent structures). Analytical procedures have been developed that account for the differences between the specified free-field earthquake motions and the motions of the points at which the structure is supported; however, a major difficulty in the analysis of dam response is the fact that there may be significant variations between the motions to be expected in the free-field at widely spaced points on the dam-foundation interface. Many theoretical studies have been made of the spatial variation of seismic waves propagated to highly idealized dam-canyon interfaces, but there is almost no information on the seismic motions that have actually occurred at the boundaries of a real dam. Hence, the planning and installation of strong-motion accelerograph networks at the canyon sites of actual concrete dams and also at free-field sites in canyons where concrete dams might be built are among the urgent research needs in the general area of defining earthquake input for concrete dams.

LINEAR RESPONSE ANALYSIS

In the earliest efforts to represent the effects of earthquakes on dams, the dams were considered to be rigid systems supported on a rigid foundation. Thus, the earthquake forces acting in the structure could be expressed as the product of the earthquake acceleration and the mass of the corresponding part of the structure. More recently the elastic properties as well as the mass of the dam materials have been incorporated into the mathematical models, and as a result the vibration properties of dams (the mode shapes and frequencies) have been recognized to have a controlling influence on earthquake response behavior.

Because of the ability of the finite element method to define mathematical models with arbitrary geometries and variations of material properties, this method is usually adopted in formulating a mathematical model of a concrete dam. In this sense the seismic response analysis of a concrete dam may be considered to be similar to any other structural dynamics analysis. However, the concrete dam analysis is greatly complicated by the fact that the structure interacts with its environment during its dynamic response—with the water retained in the reservoir and with the deformable foundation rock that supports it.

These interaction mechanisms may be included in the model in a crude way by combining finite element meshes representing a limited extent of the reservoir water and foundation rock together with the model of the concrete dam. However, it is recognized that dynamic pressure waves will be generated by the earthquake in the reservoir water, and similarly there will be stress waves in the foundation rock. The effect of these waves acting in the actual unbounded media will be to transmit energy away from the dam. Thus, a significant energy loss mechanism is not represented by the bounded finite element models of reservoir water and of foundation rock, and the development of mathematical models that properly account for these effects has become a major objective in research on the seismic behavior of concrete dams.

Gravity dams and their environment often can be idealized as simple two-dimensional systems, and for this reason interaction response analysis procedures were first developed for gravity dams; the most powerful analytical techniques employed a substructure formulation and a frequency-domain response-analysis procedure. More recently a similar approach has been applied to the analysis of three-dimensional arch dam-reservoir systems in which the arch dam is modeled as a finite element mesh and the reservoir is approximated as a limited form of continuum. In these analyses, however, the foundation rock is still modeled as a bounded finite element mesh, and the elimination of this rock boundary constraint is an important current research objective.

Another problem of present concern is the evaluation of the effective stiffness of typical reservoir bottom boundaries, which are generally covered by a layer of silt having indeterminate thickness and mechanical properties. It has been found by analytical experiment that a significant part of the vibratory earthquake response energy of a dam may be transmitted by reservoir pressure waves into the bottom silt layer, but the amount of such energy loss depends directly on the absorption coefficient of the reservoir bottom, and no measurements of this property have been made in actual reservoirs. A field measurement program to obtain this information is urgently needed, because the bottom absorption directly affects the earthquake response stresses that are calculated in the dam.

NONLINEAR RESPONSE ANALYSIS

Although linear response studies have provided great insight into the earthquake performance of concrete dams, it is evident that a rigorous esti mate of the seismic safety of a dam can be obtained only by a nonlinear analysis if a significant amount of earthquake damage is expected. In fact, minor local damage may have little effect on the global stiffness of a dam, and in such cases it is possible to make a reasonable estimate of the expected degree of damage by proper interpretation of the results of a linear response analysis. But for cases of severe damage, in which a collapse condition may be approached, the dynamic behavior is drastically changed from the linear response mechanism, and the true nonlinear performance must be incorporated into the analysis procedure if a valid estimate of the damage is to be made.

A nonlinear response analysis requires considerably greater computational effort than a linear analysis, because a much more detailed description of the performance is required to express the nonlinear response mechanisms. Special techniques are needed to carry out such calculations, and continued efforts must be directed toward improving the efficiency of nonlinear analysis procedures for concrete dams. However, the greatest impediment to effective nonlinear analysis at present is not the computational procedures; it is the lack of knowledge about the nonlinear properties of the mass concrete typically used in dams. It is well known that the mechanical properties of concrete vary with time, temperature, and moisture content; additionally, both the strength and the stiffness vary with the rate at which loads are applied. Thus, the properties associated with earthquake damage may be quite different from those measured by typical slow-speed laboratory tests, and it is essential that major programs of dynamic testing be carried out to determine material properties suitable for use in nonlinear seismic safety evaluations.

When efficient analytical procedures have been developed and when the nonlinear mass concrete material behavior can be modeled effectively, it will be necessary to perform extensive numerical parameter studies of concrete dams in order to understand the factors that control the seismic safety of different types of dam systems. In addition, properly modeled shaking-table tests will be needed to verify the effectiveness of the safety evaluation procedures that have been developed.

OBSERVATIONAL EVIDENCE

All analytical predictions of earthquake performance of concrete dams are based on numerous assumptions, each of which has a limited range of validity. Even the design of shaking-table experiments employing physical models of concrete dams requires the introduction of limiting assumptions, with regard to both the nature of the simulated earthquake motions to be applied and the properties of the model structure and of the foundation rock in which it is placed. For these reasons the best evidence by far about the earthquake behavior of concrete dams is that obtained from real dams that have been subjected to real earthquakes.

The number of instances when severe earthquake motions have acted on major concrete dams is quite small, and only in two cases was significant damage caused by the earthquakes: Koyna Dam in India and Hsinfengkiang Dam in China. In neither of these cases did the damage cause release of the reservoir, so no disaster resulted in the downstream region; however, in both cases major repairs and strengthening were done to increase the security against future earthquakes. Other concrete dams have been subjected to significant earthquake motions and have suffered only minor or no damage, so the seismic safety record of concrete dams is good; however, it must be recognized that no typical major concrete dam has been subjected to maximum credible earthquake excitation, so there is no cause for complacency about the seismic safety of concrete dams.

Because real earthquakes provide the best test of earthquake performance, it is essential that many more existing concrete dams be provided with adequate seismic instrumentation so that quantitative evidence can be obtained to be used in verifying safety evaluation procedures. In addition, free-field instrumentation should be installed at potential concrete dam sites in regions of high seismicity so that better information can be obtained about the seismic excitation to which typical dams may be subjected. The lack of adequate earthquake input data is probably the greatest present impediment to progress toward improving the seismic safety evaluation procedures for concrete dams.

However, even if the input is well established, the degree of damage to be expected from severe earthquake excitation of a concrete dam cannot be predicted with certainty. Part of the uncertainty lies in the nonlinear physical properties of the concrete, and this information can be obtained only from comprehensive, well-planned material test programs. In addition, many assumptions are made in formulating nonlinear analysis procedures, and these must be validated by experiments that simulate both the excitation and the structural response. Field vibration experiments, using either ambient or forced input excitation, are very useful in verifying the mathematical model of the dam system at low-response amplitudes, but shaking tables provide the only feasible means of testing the system well into the damage range. However, it must be recognized that the difficulty of achieving model similitude increases as the model dimensions are reduced; in particular, for the most commonly used scaling laws the model frequencies must be increased in proportion to the length scale; thus, a 1/100 scale model must be tested at frequencies 100 times greater than the actual earthquake motions. Very few shaking tables capable of testing moderate-size models can apply simulated earthquake motions that meet this requirement. The world's best facility for testing concrete dam models is operated by the

Institute of Water Conservancy and Hydroelectric Power Research in Beijing, and the initiation of a cooperative shaking-table research project with this institute would provide an excellent means for U.S. researchers to extend their experience with the nonlinear behavior of concrete dams.

SEISMIC PERFORMANCE CRITERIA

The design of earthquake-resistant concrete dams must be based on appropriate performance criteria—criteria that reflect both the desired level of safety and the nature of the design and analysis procedures. Before computers were used in design calculations, the usual criteria were merely checks on the static stability of the dam; thus, they did not represent the true dynamic earthquake response behavior and did not provide any guarantee against seismic damage or collapse. Now, with modern computer analysis procedures, it is possible to predict with some precision the linear elastic response that will result from a specified earthquake input; however, because of uncertainties in the characteristics of the expected earthquake, in the way the excitation is applied to the dam, in the dynamic strength properties of the concrete, and in the nature of the ultimate failure mechanism, it still is not possible to prescribe exactly the performance criteria to be used in the design of concrete dams. For this reason the criteria presently used in the seismic safety evaluation of existing concrete dams or in the design of new dams generally are simply stress checks in which the predicted elastic stress is compared with the expected concrete strength.

The seismic inputs specified in most criteria are the design basis earthquake (DBE) and the maximum credible earthquake (MCE). The DBE is defined as the greatest earthquake excitation expected to occur at least once during the life of the dam (possibly 100 years). It should cause no significant damage to the dam; thus, in the response to such an input it is appropriate to limit the maximum concrete stress (static plus dynamic) to the strength of the concrete applied with a factor of safety. The MCE is the greatest earthquake excitation that could ever occur at the dam site; it has a very low probability of occurrence, and therefore significant damage from this earthquake would be acceptable; however, the dam must not rupture and thus threaten life and property downstream. It is evident that the design criterion for response to the MCE must involve more than comparison of the predicted peak stress with the concrete strength. However, some judgment regarding the possibility of collapse can be obtained by studying the extent of the regions within the dam in which possible crack-inducing stresses are expected, especially how much of the structure may be subjected to such stresses concurrently. In addition, the possibility of collapse may be related to the number of times and the total duration that the cracking stress threshold is exceeded.

Eventually, it is hoped that nonlinear analysis capabilities will be improved to the point where the performance criteria can be stated in terms of acceptable amounts of nonlinear displacement; both the types of displacements and their location in the dam will influence the acceptable amplitude of displacement. However, significant advances will have to be made in the understanding of possible earthquake collapse mechanisms as well as in nonlinear analysis capabilities for concrete dams before such criteria may be proposed. Thus, it is evident that both analytical and experimental research on the nonlinear behavior of concrete dams continues to be of paramount importance.

1

Introduction

The seismic safety of dams is an issue that has been receiving increasing attention in many parts of the world during recent years. It has become a major factor in the planning for new dams proposed to be built in seismic regions and in the safety evaluations of existing dams in those areas. There are two main reasons for this increasing level of concern: (1) The risk of a major disaster due to dam collapse increases each year because the population is increasing downstream of nearly all large dams. Consequently, large population centers are now at risk in locations where only a few people may have lived at the time when the dams were built. (2) Knowledge of the complex nature of the earthquake motions that may attack a dam is increasing rapidly as more earthquake records are obtained all over the world, and it is becoming apparent that the seismic safety precautions taken in the past did not fully recognize the hazard.

The NRC Panel on Earthquake Engineering for Concrete Dams was appointed to evaluate the present status of seismic safety evaluation of concrete dams and to prepare a report summarizing its findings. Specific objectives of the report were stated to be as follows:

1. To indicate to research workers the present state of knowledge of the earthquake problem as it relates to concrete dams, and the areas where knowledge is lacking and where further research is needed.
2. To bring the earthquake problems of concrete dams to the attention of government agencies and other organizations that may initiate, direct, or fund research and to provide them with information helpful in planning such activities.

This evaluation has been limited to concrete dams because analysis procedures and research needs for embankment dams are quite different.

THE HAZARD OF DAMS

Dams probably were among the earliest major structures to be created by humans; the reservoirs retained by dams were key elements in water supply systems that dated back at least 5,000 years. The first dams undoubtedly were small earthen embankments that were designed by trial and error; failures of these dams must have occurred frequently but with very little consequence. However, with expanding needs for water and with confidence derived from previous successful dam construction ventures, the sizes of dams grew, leading to increasing potential for disaster. Understanding of the changes in behavior associated with bigger dams tended to lag behind the increasing boldness of the planners, with the result that failures continued to occur with increasingly tragic consequences. Ultimately, this history of dam tragedies led to the formation of agencies responsible for dam safety.

A notable example of a dam failure that was responsible for imposition of controls on dam safety was the collapse of the St. Francis Dam, located about 45 mi north of Los Angeles, California. This concrete gravity dam, 205 ft high by 700 ft long, and impounding a 35,000-acre-foot reservoir, is shown before and after the collapse in Figure 1-1. The collapse occurred almost instantaneously just before midnight on 12 March 1928, creating a surge of water 125 ft high and an estimated discharge rate of 500,000 cfs. The resulting extensive property damage, and especially the loss of over 400 lives, clearly demonstrated the hazard presented by major dams and led directly to the organization of an agency of the State of California responsible for the safety of dams. This agency, now known as the Division of Safety of Dams of the California Department of Water Resources, has jurisdictional authority over nearly 1,200 dams. Its operation during the past 60 years has served as a model for the organization of similar agencies in many other states and in other countries.

The hazard posed by large dams has been demonstrated by failures of many dams since 1928—dams of many types located all over the world. Two recent examples of dam disasters occurred with concrete thin arch structures—Malpasset in France and Vaiont in Italy. The Malpasset Dam failed in a sudden burst on the evening of 2 December 1959, creating a huge wave that destroyed towns and villages downstream with tremendous loss of life. As was the case with the St. Francis Dam, the failure occurred not in the concrete structure but rather in a weak seam in the foundation rock supporting the dam.

The situation at the Vaiont disaster was entirely different in that the dam did not fail; it remained intact despite being severely overloaded when it was overtopped on the night of 9 October 1963 by a great wave reaching more than 300 ft over the crest. This wave was generated by a tremendous landslide that dropped 350 million cubic yards of rock and earth into the reservoir from a cliff high on the side of the valley. However, the overtopping

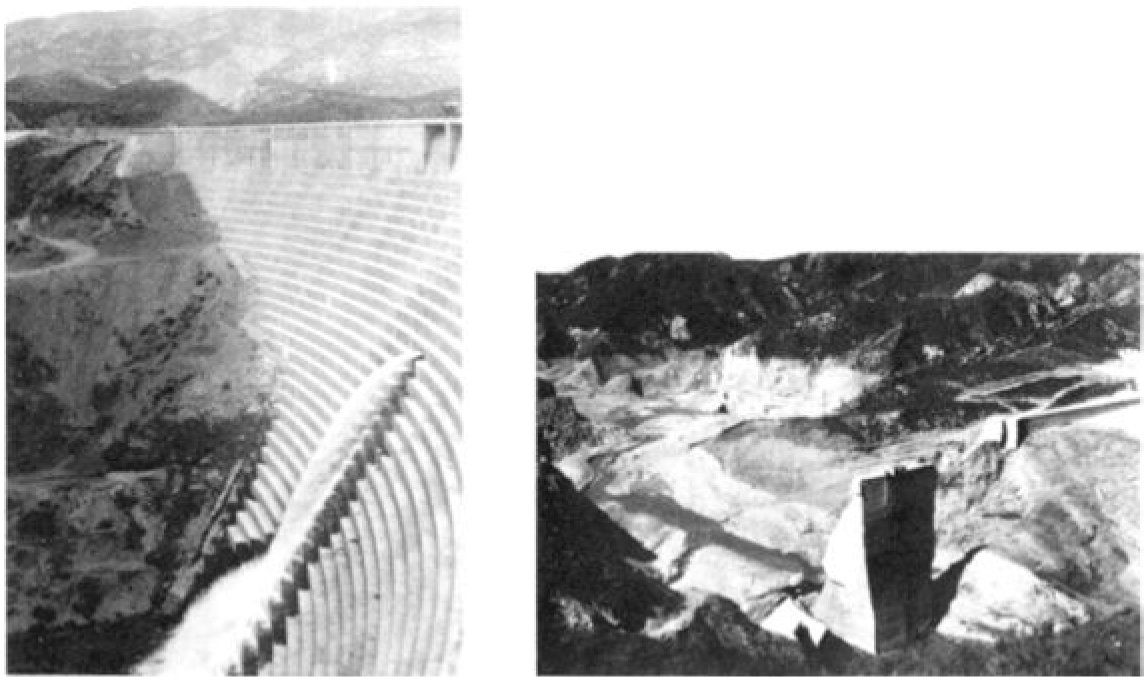


Figure 1-1 St. Francis Dam. The 1928 failure of this dam, caused by shear failure in the foundation rock, led to promulgation of dam safety legislation in California. (a) Before failure, the dam was 205 ft high and 700 ft long and retained 35,000 acre-feet of water. (b) The foundation failure caused dam rupture, and the reservoir release completely destroyed the dam.

had the same effect on property and people downstream as if the dam had been breached; over 2,000 fatalities resulted.

The failure of Teton Dam, built in Idaho by the Bureau of Reclamation, demonstrated that embankment dams also present a threat to populations downstream, even when they are built by modern construction techniques. The failure occurred on 5 June 1976, when the water level was 3.3 ft below the spillway during the first filling of the reservoir. It was initiated by leakage in the contact zone between the impervious core material and the foundation rock; the leak quickly led to erosion of the embankment and caused total release of the reservoir within a few hours. The resulting flood produced property damage estimated at over one-half billion dollars, but fortunately flood warnings were issued early enough that only 14 lives were lost.

None of the aforementioned catastrophes was related to earthquake activity, but several other events during the same time span have demonstrated the additional risk that applies to dams located in seismic regions. Two significant examples of earthquake damage to concrete dams were incidents observed at Hsinfengkiang Dam in China and at Koyna Dam in India. These are discussed more fully in subsequent chapters of this report, but are mentioned here to emphasize that earthquakes truly are a major hazard to concrete dams. Hsinfengkiang Dam is located in Kwangdong Province; it is 1,444 ft long and 344 ft high and was completed in 1959. As originally designed, it had 19 buttress blocks in the central segment, with gravity sections located on either side. The original design was intended to resist ground shaking of only intensity 6 on the Modified Mercalli Scale, but when numerous earthquakes were experienced in the vicinity during and shortly after the filling of the reservoir, the dam was strengthened to withstand earthquakes up to intensity 8. On 19 March 1962 the dam was severely shaken by ground motions considerably above intensity 8, caused by an earthquake of magnitude 6.1 located very close to the dam. No instrumentation had been installed in time to record this event, but strong-motion seismographs were installed in the dam to record the effects of aftershocks, and a magnitude 4.5 aftershock produced a peak horizontal acceleration of 0.54 g recorded at the dam crest. During the even more intense motion that occurred in the main shock, the dam developed horizontal cracks at a change of section in the nonoverflow buttress blocks on each side of the spillway, but no leakage occurred. Subsequent analysis demonstrated that cracking was to be expected at this location on the structure during an earthquake of this intensity, and the dam was then strengthened so that it could withstand even more intense ground motions.

Koyna Dam is a straight gravity structure built of rubble concrete in southwestern India. It is 2,800 ft long and 338 ft high at the tallest block; it has an unconventional change of slope on the downstream face resulting from changes introduced while construction was in progress. Originally the dam was intended to be built in two stages, but while the first stage of construction was in progress it was decided to build the dam to its final height in one stage. This change of plans necessitated modification of the dam cross section. On 11 December 1967 a magnitude 6.5 earthquake occurred, with its epicenter at a distance of 8 mi but producing a fault trace that passed as close as 2 mi from the dam. Instrumentation within the dam recorded peak horizontal accelerations of 0.63 and 0.49 g in the cross-channel and the stream directions, respectively, and horizontal cracks caused by these motions appeared on both the upstream and downstream faces of the tallest nonoverflow blocks. They were located at the elevation where the slope of the downstream face changed, and subsequent dynamic finite element analysis demonstrated that such cracking was to be expected from an earthquake of the observed intensity. However, it is noteworthy that the reservoir was not released by this damage, even though minor leakage was observed. Thus, the cracking did not cause dam failure, and there was no flooding disaster downstream, although 180 people in the vicinity were killed by the earthquake ground vibration effect. Subsequently, the dam was strengthened by the addition of buttresses on the downstream face of the nonoverflow blocks.

A notable example of earthquake damage to an embankment dam occurred at Van Norman Dam near Los Angeles, California, during the February 1971 San Fernando earthquake. This hydraulic fill structure was built in a series of stages starting in 1912, with the last stage completed in 1940. It retained the lower San Fernando reservoir, and by 1971 tens of thousands of people were living immediately downstream from it. During the earthquake a large slice of the upstream embankment—extending up to include the dam crest—slid into the reservoir. Only the fact that the reservoir had been lowered significantly due to concerns about seismic safety (it was 35 ft below the dam crest at the time of the earthquake) prevented the dam from being overtopped, with disastrous consequences to the population downstream.

This near disaster was a major factor leading Congress to promulgate the National Program of Inspection of Dams in 1972. The subsequent nonseismic failures of Teton Dam in 1976 and Kelley Barnes Dam in Georgia in 1977 caused additional concern for dam safety on the part of Congress and resulted in the appropriation of funds to implement the inspection program.

SEISMIC SAFETY OF CONCRETE DAMS

The dam inspection program was the first formal step toward improvement of dam safety in the United States. It provided for taking a census of all dams in the country and for identifying those for which detailed studies would be needed to determine whether they met the required level of safety.

Possible failures due to all types of causes were considered for both concrete and embankment dams. For dams located in seismic regions it generally was concluded that further investigations of seismic safety were needed, due to advances in knowledge of the earthquake input as well as to better understanding of the dynamic response mechanisms. Accordingly, major reevaluations have been made of the seismic safety of many existing dams during the past decade. In addition, performance predictions have been made for the relatively small number of new dams built or proposed to be built in seismic regions in this country during this time.

In order to put the question of the seismic safety of concrete dams in proper perspective, it must be noted that there is no record of any concrete dam ever having been damaged due to earthquake excitation to the extent that the reservoir was released. This perfect record stands even though more than 100 concrete dams of all types have been subjected to measurable shaking due to earthquakes in many parts of the world. But this record does not justify complacency, because in very few of these numerous examples were measurements actually made of the intensity of shaking, and the severity of the tests generally is not known. In one of the few cases where severe shaking was recorded—at Pacoima Dam, a concrete arch in the epicentral region of the 1971 San Fernando, California, earthquake—the earthquake forces were greatly reduced because the reservoir surface at the time was at less than half the design water level. Hence, it is not possible to draw definitive conclusions concerning seismic safety from this history of performance. Consequently, even though the earthquake safety record of concrete dams is excellent, much more research must be done if the details of their seismic behavior are to be understood and predicted reliably.

Until two or three decades ago, the only consideration given to earthquakes in the design of concrete dams was to apply a static horizontal force specified as a fraction of the weight of the structure to represent the seismic design loading. In later stages of its use this equivalent static load design procedure often included an additional weight to represent the inertial resistance of the water behind the dam. In these analyses a gravity dam was modeled as a simple cantilever column of varying cross section, but with the development of digital computers and the finite element method of analysis, it became possible to model the geometric configuration of the dam in a realistic way. By using three-dimensional finite elements, even arch dams could be modeled reliably. Subsequently, the analysis procedure was extended to treat the true dynamic character of the earthquake motions and the dynamic interaction of the dam with the reservoir water and foundation rock; then the aforementioned use of equivalent static loads was applied only in preliminary analyses. Present studies that take advantage of these advances in analytical procedures give greatly improved estimates of the seismic safety of existing or proposed concrete dams. However, uncertainties still remain in nearly every aspect of the analyses, and an intensive continuing research effort is needed to reach the point where full confidence in the predictions of seismic safety will be attained.

SCOPE OF THE REPORT

This report describes the present state of the art of earthquake response analysis for concrete dams and identifies major areas where additional research is needed. The subject matter that follows is divided into five chapters, each dealing with a different aspect of the analysis procedure. It is important to note that the critical issue of defining the design earthquake (i.e., the ground motions expected to act at the dam site) is not considered in this report. It is assumed that geologists and seismologists will have established the magnitude and epicentral position of the greatest earthquake that must be considered in the seismic performance evaluation and that this information will have been applied, together with knowledge of the foundation rock properties, to obtain an estimate of the most intense free-field ground motions that may be expected in the vicinity of the dam. Thus, this report deals only with procedures for evaluating the response of the concrete dam to these specified earthquake motions. Urgent research needs that have been identified in the various subject areas are listed and described at the end of each chapter.

2

Earthquake Input

EARTHQUAKE EXCITATION CONCEPTS

In evaluating the earthquake performance of concrete dams, it is evident that descriptions of the earthquake ground motions and the manner in which these motions excite dynamic response are of paramount importance. The procedure that leads to the selection of the seismic input for concrete dams is similar, in general, to that for other large important structures such as nuclear power plants and long-span bridges. It involves the study of the regional geologic setting, the history of seismic activity in the area, the geologic structure along the path from source to site, and the local geotechnical conditions. Ground motion parameters that may be utilized in characterizing earthquake motions include peak acceleration (or effective peak acceleration), duration, and frequency content of the accelerogram. However, the methods of selecting such seismic parameters for the purpose of generating input ground motions or response spectra are well documented (2-1) and are not repeated here.

The focus of this chapter is on the specification of earthquake input motions to be used in the analysis of concrete dam-reservoir-foundation systems. Obviously, the level of sophistication to be used in defining the seismic input is closely related to the degree of understanding of the dynamic behavior of the dam system and to capabilities for modeling such behavior. Thus, progress in defining seismic input has followed a long evolutionary process parallel to that of the dynamic analysis capability. It is not surprising, therefore, that the earliest method of defining earthquake input to concrete dams was merely to apply a distributed horizontal force amounting to a uniform specified fraction (typically 10 percent) of the weight of the dam body. This force was intended to represent the inertial resistance of a rigid dam subjected to the horizontal motion of a rigid foundation. The procedure was easily extended in an approximate sense to include the hydrodynamic pressure effects of the reservoir water by invoking the added mass concept (i.e., assuming that a portion of the reservoir water would move together with the dam body).

Major improvements over the rigid dam approach resulted when the dynamic effects of dam deformability, i.e., the free vibration behavior, were recognized. The first improvement was to convert the equivalent static force from a uniform distribution to a form related to the dam fundamental vibration mode shape. The second improvement was to account for the amplification of the base input motions in the response of the dam. Representing these frequency-dependent amplification effects by means of the earthquake response spectrum provided an appropriate amplitude of equivalent static load distribution to be used in the response analysis. The basic assumption of all these methods of analysis is that the foundation rock supporting the dam is rigid, so the specified earthquake motions are applied uniformly over the entire dam-foundation interface.

However, as the methods of response analysis improved, it became apparent that the rigid base earthquake input no longer was appropriate. Because of the great extent of the dam, and recognizing the wave propagation mechanisms by which earthquake motions are propagated through the foundation rock, it is important to account for spatial variation of the earthquake motions at the dam-foundation interface; these spatial variations also may result from "scattering" of the propagating earthquake waves by the topography near the dam site. A brief discussion of basic procedures for defining seismic input is presented in a recent report (2-2); the essential concepts contained in that report are summarized in the following paragraphs.

Standard Base Input Model

The dam is assumed to be supported by a large region of deformable rock, which in turn is supported by a rigid base boundary, as shown in Figure 2-1(a). The seismic input is defined as a history of motion of this rigid base, but it is important to note that the motions at this depth in the foundation rock are not the same as the free-field motions recorded at ground surface.

Massless Foundation Rock Model

An improved version of the preceding model is obtained by neglecting the mass of the rock in the deformable foundation region. This has the effect of eliminating wave propagation mechanisms in the deformable rock, so that motions prescribed at the rigid base are transmitted directly to the

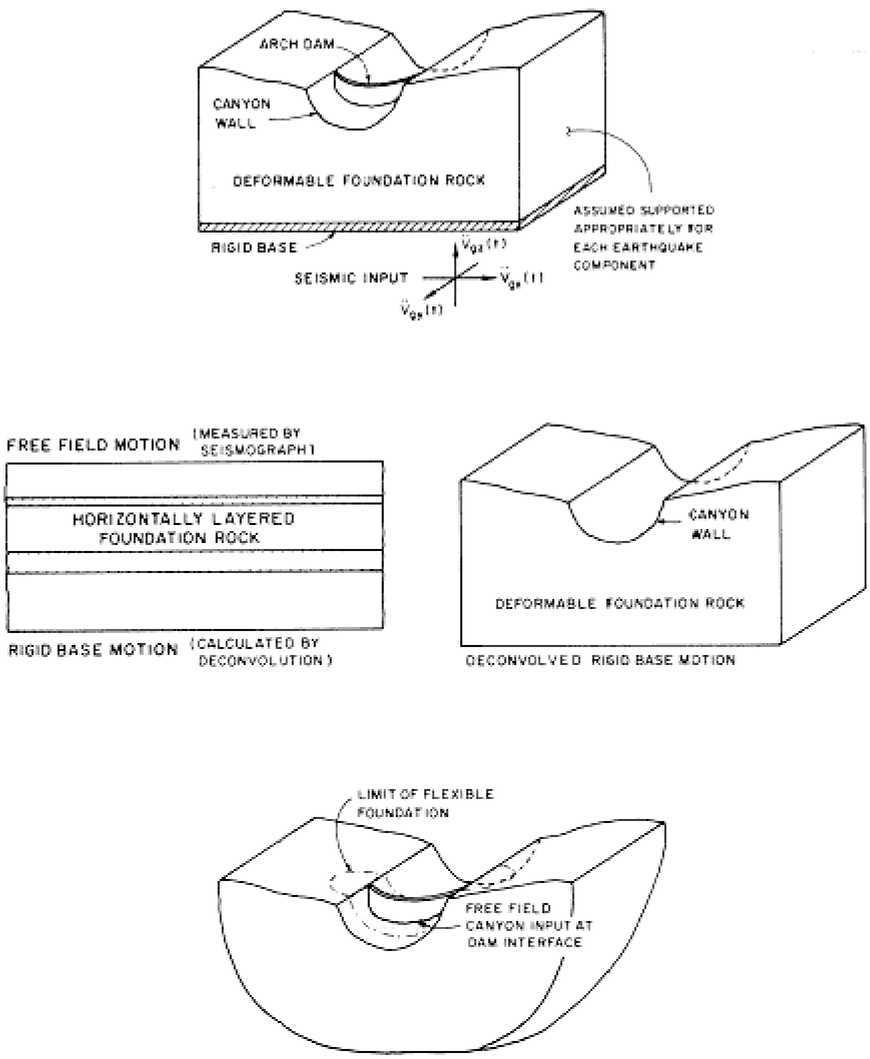


Figure 2-1 Proposed seismic input models for concrete dams (2-2). (a) Standard rigid base input model; mass of foundation rock either included or neglected. (b) Deconvolution of free-field surface motions to determine rigid base motions. (c) Analysis of two-dimensional free-field canyon motions using deconvolved rigid base motions as input. (d) Analysis of three-dimensional dam-foundation system response using two-dimensional free-field canyon motions as input.

dam interface. With this assumption it is reasonable to prescribe recorded free-field surface motions as the rigid base input.

Deconvolution Base Rock Input Model

In this approach, as illustrated in Figure 2-1(b), a deconvolution analysis is performed on a horizontally uniform layer of deformable rock to determine motions at the rigid base boundary that are consistent with the recorded free-field surface motions. The resulting rigid base motion is then used in the standard base input model. This procedure tends to be computationally expensive because the mathematical model includes a large volume of foundation rock in addition to the concrete dam.

Free-Field Input Model

A variation of the preceding procedure is to apply the deconvolved rigid base motion to a model of the deformable foundation rock without the dam in place, in order to determine the free-field motions at the interface positions where the dam is to be located. These calculated interface free-field motions account for the scattering effects of the canyon topography on the earthquake waves and are used as input to the combined dam-foundation rock system. In many cases of the form shown in Figure 2-1(c), it may be reasonable to assume two-dimensional behavior in modeling the scattering effects and then to apply these two-dimensional free-field motions as input to a three-dimensional dam-canyon system, as indicated by Figure 2-1(d).

In the above-mentioned report (2-2) these seismic input models are discussed in the context of arch dam analysis, but they are equally applicable for gravity dams when the foundation topography warrants a three-dimensional analysis. If the dam is relatively long and uniform, so that its response may be considered to be two-dimensional, the canyon scattering effect need not be considered, but the seismic input still may vary spatially due to traveling-wave effects. It is important to note that all four of the input models listed above include a rigid boundary at the base of the deformable foundation rock; thus, vibration energy is not permitted to radiate from the model. Elimination of this constraint is one of the key issues in present research on seismic input procedures.

Of these four input models the free-field model usually is the most reasonable, so a key element in the input definition is the determination of appropriate free-field motions. Research progress in this area is discussed in the next section.

PRESENT STATUS OF KNOWLEDGE

Prediction of Free-Field Motion

Definition of seismic input is very closely related to the way the dam-reservoir-foundation system is modeled. Although existing finite element programs for the dynamic analysis of concrete dams (2-3, 2-4, 2-5) use uniform base motion as input, these programs can be modified to accept nonuniform earthquake excitation at the interface between dam and canyon wall. In this case the free-field motion is defined as the motion of the damfoundation contact surface due to seismic excitation without the presence of the dam.

Two-Dimensional Case—Method

If the canyon where the dam is to be located has an essentially constant cross section for some distance upstream and downstream, it may be treated as a linearly elastic half-plane, and the problem of evaluating the earthquake free-field motion can be formulated as a wave scattering problem with the canyon as the scatterer. Various approaches have been used to obtain solutions to the problem.

The case involving earthquake SH waves (i.e., horizontal shear waves) is somewhat simpler in the sense that only out-of-plane displacements occur. Closed-form solutions have been obtained for semicircular canyons (2-6) and for semi-elliptical canyons (2-7). For cases with more general geometry, further assumptions must be made in the formulation or the numerical solution techniques. By using the method of images and integral equation formulation, results have been obtained for SH wave scattering due to arbitrarily shaped canyons by solving the integral equation numerically (2-8). The same problem also has been solved using a different integral equation formulation (2-9); in this approach the free boundary condition at the canyon wall is satisfied in the least squares sense.

An integral equation approach that imposes an approximately satisfied boundary condition also has been used in the solution of problems involving P and SV waves (2-10) (i.e., compression and vertical shear waves). Similar procedures have been used by others in solving P, SV, and Rayleigh wave problems (2-11, 2-12). A solution for incident SH, P, and SV waves also has been obtained by assuming periodicity in the surface topography and a downward-only scattered wave (2-13).

Direct numerical solution using a finite difference formulation has been employed to evaluate the scattering effects of various surface irregularities—for example, a ridge with incident SH wave (2-14), vertically incident SV and P waves on a step change in surface (2-15), and the use of nonreflecting boundary conditions with other surface irregularities (2-16).

Direct finite element solutions also have been used to solve scattering problems of P and SV waves incident on a mountain and on an alluvium-filled canyon (2-17), using more than one solution to obtain the canceling effect on nonreflecting boundaries (2-18). Standard plane-strain soil dynamics finite element programs with special treatment at nonreflecting boundaries (2-19, 2-20, 2-21) reportedly have been used for P, SV, and SH waves with a simple modification (2-22). A particle model combined with finite element modeling to account for an irregular surface has been used for SV waves incident to cliff topography (2-23). The free-field motions at V-shaped and close-to-V-shaped canyons also have been studied using a combination of finite and infinite elements in a model with finite depth that extends to infinity horizontally (2-24). In this case earthquake motions prescribed at the rigid base of the foundation are taken as input to the system.

Two-Dimensional Case—Results

Results for a semicircular canyon are used as the basis of discussion here because the most information is available for this simple geometry; some reference also is made to cases involving other geometries, as depicted in Figure 2-2. Results expressed in the frequency domain are described in many cases to indicate the effects of wave frequency. Motions at the canyon walls generally are found to be dependent on the ratio of canyon width to wave length (wave frequency), on the angle of wave incidence, and on the wave type. The effect of scattering is more significant when the wave length is of the same order as or smaller than the canyon width. In comparison with the free-field motion without any canyon, the free-field motion at the canyon surface can be either amplified or reduced depending on the location of the observation point, as shown in Figure 2-3. In general, motions near the upper corner of the canyon facing the incident wave are amplified; the amplification increases as the wave length decreases and as the direction of incidence tends toward the horizontal. For incident SH, P, SV, and Rayleigh waves, the maximum amplification is found to be 2 for semicircular and semi-elliptical canyons (2-6, 2-7, 2-12). However, this amplification factor can be higher if the canyon surface has local convex regions, which tend to trap energy (2-8). Motion from SH and Rayleigh waves generally is reduced near the bottom of the canyon. For Rayleigh waves and close-to-horizontally incident SH waves, the motion at the back side of the canyon also is often reduced, but this shielding effect disappears for SV and P waves. For vertically incident SH waves, the wall slope of a triangular canyon has significant effects on the motion at the wall surface (2-9); steeper slopes lead to greater reductions in motion near the bottom of the canyon.

The amplification of motion at some locations and the attenuation of motion at others results in a large frequency-dependent spatial variation of motion along the canyon walls. This spatial variation is more abrupt when the canyon-width-to-wave-length ratio is larger than 1 (higher frequency) for all types of waves. Calculated relative motion ratios of 2 to 3 are common in many cases of differing incident angles and wave types. For the more irregular geometry of a real canyon, a calculated relative motion ratio as high as 6 was reported for Pacoima Dam, California (2-8).

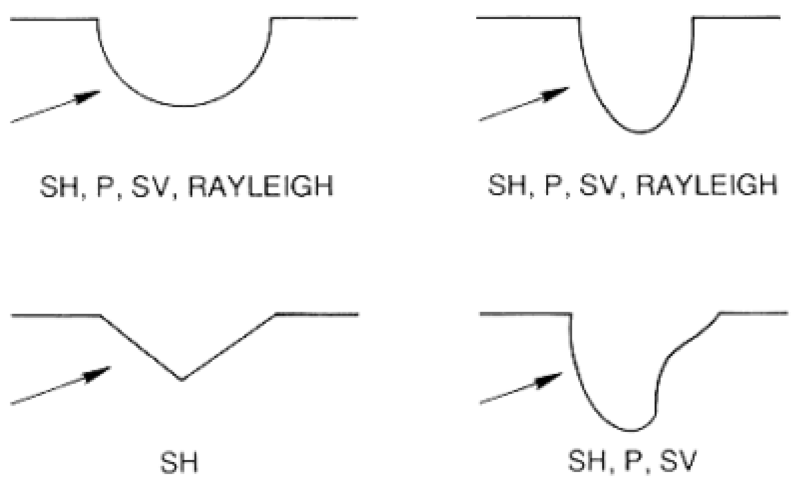


Figure 2-2 Valley shapes and input wave types for which two-dimensional analyses of wave scattering effects have been reported.

The relative phase of motions along the canyon walls has been reported for the case of SH waves incident to a semicircular canyon (2-6). It seems that the phase variation is close to what can be predicted from simple traveling-wave considerations for most of the canyon wall. Near the upper corners of the canyon, however, more abrupt variations of phase angle appear.

From the above brief description of theoretical results, it is clear that the spatial variation of free-field motion is very complex and frequency dependent. In an effort to obtain an averaged index of motion intensity, a Topographical Effects Index was defined using the Arias Intensity Concept (2-25); however, this index is still dependent on location and angle of incidence.

Three-Dimensional Case

Analytical solutions for three-dimensional canyon topography are much more difficult to obtain. For the simple case of a hemispherical cavity at the surface of an elastic half-space, series solutions have been obtained for

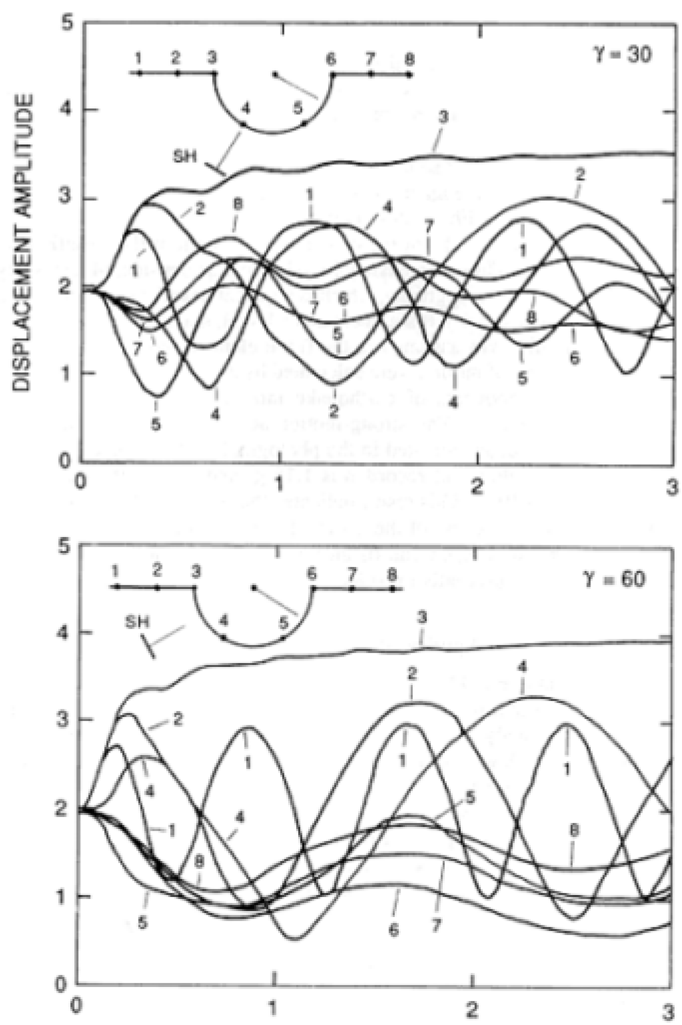


Figure 2-3 Calculated amplification of incident plane SH waves by a semi-cylindrical canyon surface (2-6). A flat free-field surface gives a displacement amplitude of 2; β represents the SH wave velocity.

incident P and S waves (2-26). A boundary element method that satisfies the free surface condition at and near the canyon walls in a least squares sense has also been applied to axisymmetric cavity problems (2-27), although results are given only for a hemispherical cavity with a vertically incident P wave.

Perhaps the most relevant solution for three-dimensional canyon topography is that obtained by finite element analysis of Pacoima Dam and its adjacent canyon (2-28), shown in Figure 2-4. Part of the foundation rock was included with the finite element model of the dam, taking account of the variations in the rock properties. Three-dimensional modeling was considered necessary because of the complex topography, which is apparent near the dam, consisting of a thin, spiny ridge at the left abutment and a broad, massive right abutment. A rigid base motion was assumed at the finite element base boundary, and its three components of motion were calculated by a process of deconvolution from the three components of earthquake motion recorded on the ridge above the left abutment. The strong-motion accelerograph was located at the crest of the ridge, as indicated in the photograph. The peak acceleration of the filtered left abutment record was 1.15 g, and that of the calculated base motion was 0.40 g. This result indicates that the amplification may be larger than expected because of the assumed rigid energy-trapping boundary at the base; also, the assumption of uniform motion at the base may have contributed to the conservative results.

Applicability of the Results

The theoretical free-field motion results have limitations in their application due to the various simplifying assumptions made in their derivation. In the two-dimensional analyses it was assumed that the change of topography along the upstream-downstream direction was negligible; therefore, the results are valid only for prismatically shaped canyons. Moreover, the results apply only to specific wave types, and the amplification effect of wave scattering is very much dependent on the type of incident wave. Unless the composition of an actual incident earthquake wave is known in terms of its wave types, such results are not directly applicable.

Often a complicated canyon geometry requires that free-field motion varying in three dimensions be considered, and it is doubtful that any method other than a numerical one can be expected to produce realistic results for such cases. Even with a numerical approach the various assumptions made in treating the finite boundary and in modeling the inhomogeneous media may introduce errors; thus, both two-and three-dimensional results need to be compared with actual free-field earthquake records to assess their applicability (2-29).

Because of the many uncertainties involved in modeling the geometry, the foundation material properties, and the incident earthquake motion, it is probable that a stochastic approach to defining the free-field motions will be needed in addition to the deterministic procedures reviewed in this report. Random field theory (2-30, 2-31) is quite relevant to the problem of spatial variation of earthquake input motion. Using the stochastic approach, some work already has been done for free-field motions over a flat, open surface (2-32, 2-33, 2-34, 2-35). To date, no results have been reported on the probabilistic nature of seismic motions along a canyon wall.

Figure 2-4 Pacoima Dam, California, was subjected to the 1971 San Femando earthquake; the seismic motions were recorded by a seismograph on the narrow rock ridge above the left abutment, at the point indicated (2-28).) (Courtesy of George W. Housner)

Measured Motions of Foundation Rock

Reports of actual earthquake motion recorded at the walls of a canyon are very scarce; however, the importance of differential input motion at a dam site is well recognized, and a few such reports do exist, mostly for abutment motion of existing dams. As early as 1964, differential motion at the two abutments of the Tonoyama (arch) Dam in Japan was reported (2-36, 2-37). The dam is 65 m high, with a cross-canyon width at the crest level of about 150 m. The maximum recorded acceleration at the center of the dam crest during an earthquake in 1960 was 0.018 g, and those at the two abutments were less than 0.010 g. In general, the records at the two abutments appeared to be quite similar in magnitude and phase. However, Fourier analysis revealed that the amplitude of motion at the right abutment was two to three times that at the left abutment for frequencies greater than 4 Hz. Other observations made in Japan at the Tagokura (gravity) Dam (2-38) and at the Kurobe (arch) Dam (2-39) also indicate differing motions at the opposite abutments; however, in both of these cases there was amplification of motion over the height of the abutments.

Eight aftershock measurements were made in the vicinity of Pacoima Dam after the San Fernando earthquake (2-40). Comparison of the records obtained at the south abutment near the original strong-motion station with those obtained at a downstream canyon floor location some distance from the dam revealed an average amplification of about 1.5 for horizontal motions at the top of the ridge. The amplification was about 4.2 near a frequency of 5 Hz but decreased to a ratio near 1 for lower frequencies. In a separate study four aftershocks of the San Fernando earthquake were recorded at two stations, one near the dam base and the other near the top of Kagel Mountain (2-41). The two stations were approximately 3,000 ft apart and were selected to represent the free-field motions at the base and top of the mountain. The highest time-domain horizontal acceleration ratio (top-to-base) was about 1.75, but the frequency-domain ratio, as measured by the pseudorelative velocity spectra, was as high as 30 at a frequency of about 2 Hz.

A correlation study on ground motion intensity and site elevation was carried out for the general area of Kagel Mountain and Pacoima Dam using the San Fernando earthquake data (2-42). On a larger scale it was found that there was an almost linear relationship between the peak recorded motion of a site and the recorder elevation, as the profile rises from the lower San Fernando dam site (approximately 1,200 ft) to the Pacoima dam site (approximately 2,000 ft) to the peak of Kagel Mountain (approximately 3,500 ft). Based on this linear relationship, it was calculated that the base rock acceleration at the Pacoima dam site was about 0.99 g, but it is evident that this calculation ignores local topographic features such as the abutment ridge.

In a somewhat similar case, aftershocks of the 1976 Friuli earthquake in

Italy were measured at the Ambiesta (arch) Dam (2-43). The dam is 60 m high, and the canyon width at the crest level is about 140 m. Records were obtained at three locations along the dam-foundation interface, two at crest level at the abutment and one at the base of the dam. The average ratio of horizontal peak velocity at the crest level to that at the base of the dam ranged from 3.11 to 1.88. The predominant frequency of motion at the base of the dam was about 4 Hz, based on more than 35 records having peak velocities greater than 0.002 cm/sec.

Observed motion at the Chirkey (arch) Dam in the Soviet Union was reported in a translated paper (2-44). The motion at the left abutment was recorded at three heights during a magnitude 3.5 earthquake that occurred on 4 February 1971 at an epicentral distance of 46 km. The peak velocities at heights of 160, 220, and 265 m were 0.4, 0.63, and 0.62 cm/sec, respectively. In the frequency domain it was found that the maximum spectral value increased by a factor of 2.5 when the height of the observation point increased by 100 m.

The Whittier, California, earthquake of 1 October 1987 triggered all 16 of the accelerometers that had been installed on Pacoima Dam. Preliminary reports indicated that the accelerations at the dam base were on the order of 0.001 g, while those at about 80 percent height of the dam on the damabutment interface were on the order of 0.002 g (2-45).

All of the above-mentioned records were of small amplitude due to low-intensity shaking. A larger-amplitude record was obtained at the Techi dam site in Taiwan during an earthquake on 15 November 1986 (2-46). This arch dam is 180 m high, and the canyon width is about 250 m at crest level. The peak acceleration recorded at the center of the crest in the upstream-downstream direction was 0.170 g. Three strong-motion accelerographs had been installed along the dam-foundation interface, one at the base of the dam and the others at about midheight on the opposite abutments. Unfortunately, one of the midheight instruments malfunctioned, leaving only one operational. The peak acceleration obtained at the dam base in the upstream-downstream direction was 0.014 g, while that at midheight of the dam-abutment interface was 0.022 g. In the cross-canyon direction the peak acceleration at the dam base was 0.012 g, versus 0.017 g at the midheight abutment location. These records clearly demonstrate a large spatial variation of motion along the foundation interface, but it is probable that dam interaction contributed significantly to the recorded motion. Consequently, these data are not representative of free-field canyon wall motions.

A 1984 earthquake of amplitude comparable to the Techi event was reported recently for the Nagawado (arch) Dam in Japan (2-47, 2-48). The dam is 155 m high with a crest length of 355.5 m. The peak recorded radial accelerations of the dam crest were 0.197 g at midspan and 0.245 g at the quarter point from the left abutment. The recorded peak accelerations in the foundation rock 17 m below the base of the dam were 0.016 g in the N-S direction and 0.029 g in the E-W direction; the dam axis lies approximately in the N-S direction. At a level about 25 m above the base of the dam, an accelerograph installed deep in the right abutment rock away from the dam recorded accelerations of 0.018 g (N-S) and 0.021 g (E-W). Almost directly above this accelerograph, also deep in the right abutment, an instrument at crest level recorded peak accelerations of 0.031 g (N-S) and 0.026 g (E-W). Across the canyon at crest level deep in the left abutment, another recorder indicated peak accelerations of 0.026 g (N-S) and 0.021 g (E-W). The spatial variation revealed by these data is quite indicative of the lack of uniformity in the earthquake motions of the rock supporting the dam.

In a translated paper (2-49) the findings of a model test of Toktogul Dam are reported. The model had a length scale of 1:4,000 and it simulated the topography of the Toktogul dam site, which covers an area of 6 × 6 km and was 4 km deep. The model was subjected to excitation initiated at different points on the model, and the motion along the canyon wall was recorded up to a height from the bottom of the canyon equal to twice that of the dam. Three configurations were tested: without the dam, with the dam but without water, and with both dam and water; generally the greatest motion occurred for the empty canyon case.

More recently model test results were reported for an existing arch dam and for a proposed arch dam, both in China (2-50). The model scales were 1:600 and 1:2,000, respectively, and input to the models was both random excitation and impact. The model test results were found to be consistent with those from finite element analyses and from ambient vibration surveys. An amplification factor of between 2 and 3 was observed for abutment motion at the crest level relative to motion at the bottom of the canyon.

Predicted Response to Spatially Varying Input

Dynamic Excitation

Direct application of measured earthquake motions to predict dam response has been reported for the Ambiesta (arch) Dam (2-51). Three records are available, one at the base of the dam and two at the crest level near opposite dam-abutment interfaces. In the analysis the interface was divided into three zones by drawing a horizontal line near midheight on an elevation view of the downstream face. Within each zone a uniform boundary motion identical to what was recorded in that zone was used as interface input. It was reported that agreement between the calculated accelerations along the crest of the dam and the corresponding measured quantities was startlingly good, while poor agreement resulted if uniform input motion was used along the entire interface.

Using prescribed input motions at the foundation rock boundary, the effects of differential input motion on the responses of a gravity dam and an arch dam have been studied by finite element analysis (2-52). A two-dimensional plane-strain analysis was performed of the gravity dam and its supporting block of foundation rock, assuming a traveling-wave input along the horizontal foundation base boundary. To reduce the amount of computation, the boundary was divided into four regions, with uniform motion assumed in each. The dam was 46 m high, and the length of the horizontal base boundary was approximately twice the height of the dam. The input wave form was that of the S16E component of the 1971 San Fernando earthquake recorded at Pacoima Dam, and three wave speeds were used: 2,000 m/sec, 4,000 m/sec, and infinite. Stress analysis results indicated that as the wave speed was reduced the stresses in the dam increased.

In the case of the 110-m-high arch dam, which had a crest length of 528 m, a three-dimensional analysis was performed. The left half of the foundation boundary was assumed to move uniformly according to the prescribed San Fernando earthquake record, while the right half was held fixed. It was reported that different stress patterns in the dam were obtained for the variable base input as compared with the uniform base input.

In a recent study on traveling-wave effects, a small portion of the foundation rock was treated as an extension of the dam body, and shell equations were used to model the extended arch dam (2-53). Cross-canyon traveling waves in the form of harmonic motion or earthquake motion were then assigned to the periphery of the shell; reservoir effects were neglected. Results indicated that stresses in the dam increased when the period of the input harmonic wave approached the fundamental period of the shell.

In a separate study the effects of traveling waves on arch dams were examined using a finite element approach (2-54). The model was similar to the free-field input model described above, but the free-field motion was taken as a prescribed traveling earthquake wave. It was found that the effects of a wave traveling in the upstream direction were not significant when compared with the rigid base input. However, a traveling wave in the cross-canyon direction caused an average stress increase of 40 to 50 percent and a doubling of the maximum computed stress.

Traveling-wave effects also have been studied, with emphasis on the energy input to the reservoir water. A two-dimensional solution was reported for the problem of a rigid gravity dam with infinite reservoir excited by a vertical traveling ground motion (2-55). Three cases were studied: infinite wave speed, wave leaving the dam moving upstream, and wave approaching the dam from upstream. The vertical component of the El Centro 1940 earthquake was used, and in the latter two cases the wave propagation speed was taken to be three times the speed of sound in water. It was found that maximum pressure on the dam occurred when the wave approached from upstream, and it was lowest when the wave propagated away from the dam. In terms of maximum total force or overturning moment, the larger traveling-wave response was almost twice that resulting from the infinite wave speed. A similar conclusion, but with much less difference between the cases for infinite and finite wave speeds, was obtained by a finite difference study of a flexible-dam finite-reservoir model (2-56). Later a finite difference solution scheme was applied to an improved model that included energy-transmitting boundaries and elastic foundation (2-57). It was reported, however, that traveling waves did not produce more critical stress conditions in the gravity dam than did the wave with infinite propagation speed.

Applications of an energy-transmitting boundary approach to a free-field input model were reported recently (2-58). A numerical example was given to illustrate the computation procedure for a uniform free-field input motion assumed along the dam-foundation interface. In another study a new input procedure that included the influence of the infinite foundation domain was developed (2-59). The analysis procedure was divided into two stages: first, the stresses were computed on a fictitious fixed boundary facing the incident wave; these stresses were then released in the second stage, when the complete domain was modeled by finite elements near the canyon and by infinite elements away from the canyon. Numerical results were obtained for a three-dimensional dam topography, considering an SH wave propagating across the canyon. The displacement amplitudes at the crest and on the crown cantilever were found to be much reduced from those obtained using a uniform base input. The presence of the dam body was found to have the general effect of reducing the motion at the canyon wall, as compared with the free-field values at the same locations.

A recent study (2-60) of an arch dam used free-field motions for the seismic input that were computed for a canyon embedded in a two-dimensional half space and subjected to incident SH, SV, and P waves. These free-field motions were applied to a three-dimensional finite element model containing the dam, a massless foundation region, and an infinite reservoir of compressible water. Frequency-domain responses were converted into the time domain in the form of standard deviations of the response to a random input with an earthquakelike frequency content. As shown by an analysis of Pacoima Dam, inclusion of nonuniformity in the stream component of the excitation reduces the dam response, while the effect of nonuniformity in the cross-stream and vertical components varies, with the potential for some increase. For various cases of nonuniform input, the average arch stress along the crest ranged from 62 to 122 percent of that for uniform input.

Fault Displacement

All the above-mentioned analyses were for vibratory seismic input motions. The case of a fault-displacement offset occurring directly beneath the base of a concrete gravity dam also has been studied. In a two-dimensional nonlinear analysis of a dam-foundation system (2-61), a reverse fault was simulated by applying concentrated forces along an assumed fault zone that extended from the finite element boundary of the modeled foundation rock to the base of the dam body. Results for the particular case studied indicated that the dam did not crack as a result of fault displacement but partially separated from the foundation. It was recommended that the combined effect of fault displacement and vibratory seismic input could be accounted for in preliminary studies by performing a dynamic-response analysis with a linear finite element model, using a softened foundation.

More recently a model test study of a proposed 185-m-high arch dam in Greece was carried out to determine the effects of fault displacements occurring directly beneath the base of the dam (2-62). Selection of the dam type and location was based on economic considerations. A thorough seismotectonic investigation concluded that fault displacement at the dam base of as much as several decimeters could not be ruled out; consequently, design measures were taken to accommodate such possible movement. Among these was a sophisticated joint system to alleviate the adverse effects of the fault displacement. A 1:250 scale model was built and tested at the Laboratorio Nacional de Engenharia Civil (LNEC) in Lisbon. The horizontal movement of the fault was simulated by imparting to the left abutment a gradual displacement upstream to compress the arch. Results of the test indicated that the joint system worked very well in protecting the dam from collapse for a displacement of up to 1 m in prototype scale. It was also concluded that the joint system would enable the dam body to withstand a fault displacement of the order of 5 to 10 cm without damage.

There have been two cases where concrete gravity dams have actually been constructed in which the plane of an underlying fault has been extended through the entire dam section in the form of a sliding joint to accommodate possible fault movement (2-63). These are Morris Dam (2-64) in California, which was completed in 1934, and the recently finished Clyde Dam (2-65) in New Zealand. In both cases the fault ran along the river channel perpendicular to the axis of the dam and dipped about 60 degrees off horizontal. Figure 2-5 is a photograph of Morris Dam. The joint is located near the gallery entrance, which can be seen on the downstream face. The sliding joints in both dams were oriented vertically and were designed for displacements on the order of 2 m. This required an interesting geometric solution, details of which were quite different in the two cases. To date, no movement has occurred on either fault.

Figure 2-5 Morris Dam, California, was constructed over a known fault in the foundation rock with a vertical sliding joint formed in the concrete directly above the fault (2-64). The joint passes next to the galley entrance, visible on the downstream face.

Other methods of defensive design of concrete gravity dams for fault displacement include the placement of a zoned self-healing berm of embankment material at the heel of the dam and a buttressing berm of free-draining granular fill against the downstream face (2-63). Reference 2-63 also states that an acceptable defense may not exist for thin arch dams against fault displacements.

STATUS OF STRONG-MOTION INSTRUMENT NETWORKS

The topic of strong-motion instrumentation placed at concrete dam sites for the purpose of studying the spatial variation of ground motion has not received sufficient attention. Traditionally, for recording the input motion it was considered adequate to have one strong-motion recorder at either the toe of the dam or one of the abutments. As early as 1975, however, it was recommended that there be a minimum of two accelerographs located at the dam site to "record earthquake motions in the foundation" (2-66, p. 1,099). The purpose of requiring two instruments was "to give some indication of the uniformity of conditions, and to ensure some useful information in the event of an instrument malfunction." In the 1978 International Workshop on Strong-Motion Instrument Arrays, various aspects of instrumentation were discussed, and useful suggestions were made specifically for study of the spatial variation of seismic ground motions (2-67). One of the array types suggested was the "local effect array" that could be used to study the "variation of ground motions across valleys." But in that suggestion the emphasis was clearly on the motion of the overburden soil in a valley rather than that along a canyon wall. In a follow-up meeting of U.S. researchers in 1981 (2-68, p. 8), the following recommendation was made: "Lifeline and other systems should be instrumented along with building structures. These should include highway bridges and overpasses, dams, and other utility system facilities. The degree of instrumentation should be sufficient to obtain information equivalent to that for building structures." However, few if any concrete dams in the United States are currently instrumented to the extent needed to study the seismic input problem. Of the 45 concrete dams listed in a survey report (2-69), only Pacoima Dam has strong-motion instruments installed at both the toe and two abutments (as well as at other locations on the dam). Although the survey list is not complete, very few other concrete dams have seismographs installed near the toe. Even though the measuring of free-field and interface input motion has been recognized to be as important as that of the dam response (2-70), current strong-motion instrumentation for concrete dams in the United States is inadequate for the purpose of defining seismic input.

RESEARCH NEEDS

Various theoretical models have been developed for prediction of free-field motion at the surface of a valley or canyon to be used as input to a dam system; however, no verification of such input predictions has yet been achieved by comparison with actually recorded earthquake motions. The existing strong-motion instrumentation at concrete dams is not designed to provide such essential data. It is clear, therefore, that an improved instrumentation program for observation of earthquake motions at sites of existing or proposed dams is needed. Similarly, further theoretical work is needed on the deterministic and stochastic modeling of input motion to provide the basis for realistically modeling seismic input to concrete dams. Specific recommendations for research on earthquake input to concrete dams follow:

1. Deployment of Strong-Motion Instrumentation

(a) Arrays of strong-motion instruments should be deployed at selected dam sites. The locations of the instruments at each site should include at least three elevations along the abutment interfaces and selected locations within the abutment rock, the face of the canyon downstream of the dam, and several positions along the reservoir bottom. Triggering of these instruments should be synchronized so that traveling-wave effects can be detected. The seismographs should be made part of an overall instrumentation system that includes pressure transducers at the dam surface in the reservoir and accelerographs at selected locations within and on the dam body.

(b) An array of strong-motion instruments should be deployed at sites being considered for construction of concrete dams to obtain the free-field motions at a canyon location without the interference of an existing dam.

2. Strong-Motion Instrumentation Program

(a) The necessity of obtaining actual records and the high cost of instrumentation point to the need for a concerted joint effort between the dam owner/operator and the research community. A permanent or semipermanent instrumentation program should be established after a careful study of potential sites for instrumentation.

(b) A joint program of strong-motion instrumentation for concrete dams in areas of high seismicity should be developed in cooperation with other countries having similar hazards.

3. Use of Recorded Seismic Motion Records

Seismic motions actually recorded at a dam-foundation interface should be utilized in analyses intended to verify the various input methods. Because the recorded motions would be affected by dam-foundation interaction, a system identification approach may be needed to determine the free-field input motion.

4. Enhancement of Two-Dimensional Analyses

   Currently available two-dimensional theoretical free-field canyon or valley wall motions are presented in terms of the incident wave angle and wave type and are in a frequency-dependent form. Even though they are of limited applicability because of assumptions regarding two-dimensional geometry and homogeneity of the medium, these results should be synthesized to provide guidelines for defining realistic input for concrete dams.

5. Enhancement of Three-Dimensional Models

   The deterministic prediction of free-field motion at a dam site with three-dimensional topography can be performed by numerical methods such as finite elements, boundary elements, finite differences, or some combination of the three. The development of nonreflective boundaries for such three-dimensional models remains a high-priority requirement.

6. Stochastic Approach

   In view of the many uncertainties involved, the simulation of spatially varying free-field motion may require the application of stochastic theory; therefore, methods should be developed for simulating stochastic inputs for valley and canyon topographies.

7. Effects of Fault Displacement

   Further studies should be carried out focusing on the effects of fault displacements on the safety of concrete dams, using both numerical simulation procedures and physical model testing. The effectiveness of joint systems in a dam at the location of the fault break should be included in these studies.

3

Analysis of Linear Response

PRELIMINARY COMMENTS

The ability to evaluate the effects of earthquake ground motion on concrete dams is essential to assessing the safety of existing dams, to determining the adequacy of modifications planned to improve old dams, and to evaluating designs for proposed new dams. However, the prediction of the performance of concrete dams during earthquakes is one of the most challenging and complex problems found in the field of structural dynamics because of the following factors:

1. Dams and the retained reservoirs are of complicated shapes, as dictated by the topography of the sites.
2. The response of a dam is influenced to a significant degree by the interaction of the motions of the dam with the impounded water and the foundation rock; thus, the deformability of the foundation and the earthquake-induced response of the reservoir water must be considered.
3. A dam's response may be affected by variations in the intensity and frequency characteristics of the earthquake motions over the width and height of the canyon; however, this factor cannot be fully considered at present because of the lack of instrumental data to define the spatial variation of ground motion, as discussed in Chapter 2.

When evaluating the earthquake behavior of concrete dams, it is reasonable in most cases to assume that the response to low-or moderate-intensity earthquake motions is linear. That is, it is expected that the resulting deformations of the dam will be directly proportional to the amplitude of the applied ground shaking. Such an assumption of response linearity greatly simplifies both the formulation of the mathematical model used to represent the dam, reservoir water, foundation rock system and also the procedures used to calculate the response. The results of linear analysis serve to demonstrate the general character of the dynamic response, and the amplitudes of the calculated strains and displacements indicate whether the assumption of linearity is valid. In the case of a major earthquake it is probable that the calculated strains would exceed the elastic capacity of the dam's concrete, indicating that damage would occur; in this case a much more complicated nonlinear analysis would be required to determine the expected extent of damage. However, a linear analysis still can be very valuable in helping to understand the nature of the dynamic performance and in deciding whether a nonlinear analysis will be necessary. In many cases a reasonable estimate of the expected degree of damage can be made from the linear analysis, even though the results suggest that a slight to moderate degree of cracking or other form of nonlinearity is to be expected.

In this chapter both the earliest, rather primitive, techniques of estimating earthquake performance are described and also the refined, modern, linear computer analysis procedures that are presently recommended for seismic safety evaluations of concrete dams.

STATIC ANALYSIS

Traditional Analysis and Design

Because most dams in the United States were built prior to the development of modern computer analysis procedures, earthquake effects were accounted for in the designs by using methods that are now considered oversimplified. In particular, the dynamic behavior of the dam, reservoir water, foundation rock system was not recognized in defining the earthquake forces used in traditional design methods (3-1, 3-2). Thus, the forces associated with the inertia of the dam were expressed simply as the product of a seismic coefficient—taken to be constant over the surface of the dam, with typical values of 0.05 to 0.10—and the weight of the dam per unit surface area expressed as a function of location. Seismic water pressure in addition to hydrostatic pressure was specified in terms of the seismic coefficient and an additional pressure coefficient; the latter was evaluated based on the assumptions that the dam was rigid and had a plane vertical upstream face and that water compressibility effects were negligible (3-3, 3-4). Generally, interaction between the dam and the foundation rock was not considered in evaluating the aforementioned earthquake forces, but in the seismic stress analysis of arch dams the flexibility of the foundation rock sometimes was recognized through the use of Vogt coefficients (3-1). Stresses in gravity dams with ungrouted construction joints were usually determined by treating the concrete blocks as vertical cantilever beams; for the analysis of arch dams, and sometimes for gravity dams in narrow canyons or with keyed joints, the trial load analysis procedure was usually used.

Traditional design criteria (3-1, 3-2) require that the compressive stresses not exceed either one-fourth of the specified compressive strength or 1,000 psi. Tensile stresses were usually not permitted in gravity dams or, if they were, were limited to such a small value that cracking was not considered possible; in arch dams the tensile stresses were required to remain below 150 psi. In the design of gravity dams it was generally believed that stress levels were not a controlling factor, so the designer was concerned mostly with satisfying criteria for overturning and sliding stability.

Earthquake Performance of Koyna Dam

As mentioned in Chapter 1, Koyna Dam in India is one of two concrete dams that have suffered significant earthquake damage (3-5, 3-6). A photograph of this dam is shown in Figure 3-1, and it is useful to discuss its earthquake performance in some detail in this report because it was designed by the traditional static analysis procedure using a seismic coefficient of 0.05. Even though a "no-tension" criterion was satisfied in the design procedures considering seismic as well as all other forces, the earthquake of 11 December 1967 caused important horizontal cracks on the upstream and/or downstream faces of a number of nonoverflow monoliths near the elevation at which there is an abrupt change in slope of the downstream face. Although the dam survived the earthquake without any sudden release of water, the cracking appeared serious, and it was decided to strengthen the dam by constructing buttresses on the downstream face of the nonoverflow monoliths; the overflow monoliths were not damaged.

To understand why the damage occurred, the dynamic response of the tallest nonoverflow monolith was calculated, assuming linear behavior. The results indicated large tensile stresses on both faces, with the greatest values near the elevation of the downstream-face change of slope. These calculated stresses (shown in Figure 3-2), which exceeded 600 psi on the upstream face and 900 psi on the downstream face, were about two to three times the estimated 350-psi tensile strength of the concrete at that elevation. Hence, significant cracking consistent with what was observed could have been expected during an earthquake of this intensity. The maximum compressive stress in the monolith (not shown in Figure 3-2) was about 1,100 psi, well within the compressive capacity of the concrete. A similar analysis of the nonoverflow monoliths indicated that little or no cracking should have occurred there, which also is consistent with the observed behavior.

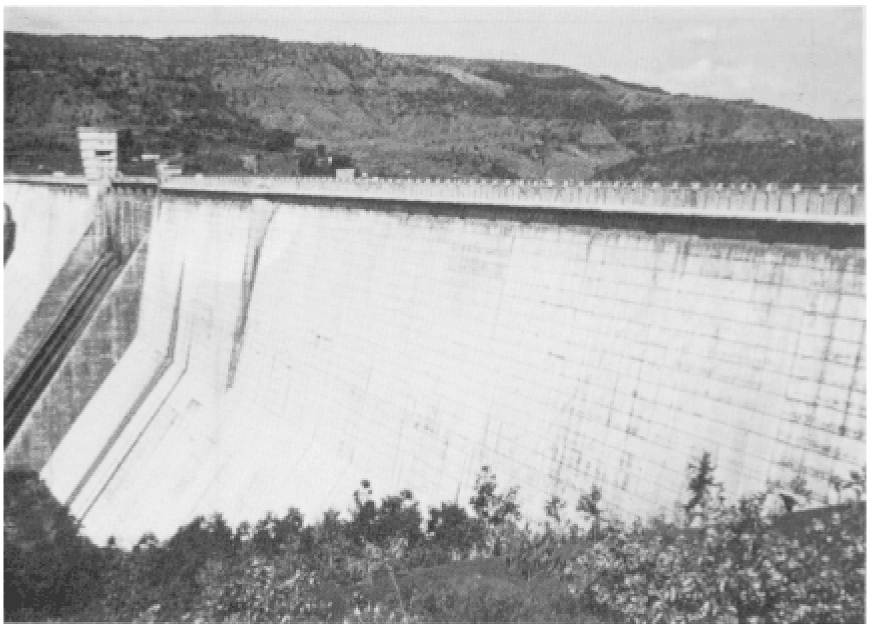


Figure 3-1 Koyna Dam, India, was damaged by a magnitude 6.5 earthquake in December 1967 (3-5, 3-6).

Limitations of Traditional Procedures

It is apparent from the preceding discussion that the dynamic stresses that develop in gravity dams due to earthquake ground motion bear little resemblance to the results given by standard static design procedures. In the case of Koyna Dam the earthquake forces based on a seismic coefficient of 0.05, uniform over the height, were expected to cause no tensile stresses; however, the earthquake caused significant cracking in the dam. The discrepancy is the result of not recognizing the dynamic amplification effects that occur in the dam's response to earthquake motions.

The typical design seismic coefficients, 0.05 to 0.10, used in designing concrete dams are much smaller in the typical period range for such dams than are the ordinates of the pseudoacceleration response spectra for intense earthquake motions, as shown in Figure 3-3. It is of interest to note that the seismic coefficients used for dams are similar to the base shear coefficients specified for buildings. However, the Uniform Building Code provisions (3-7) are based on the premise that the structures should be able to:

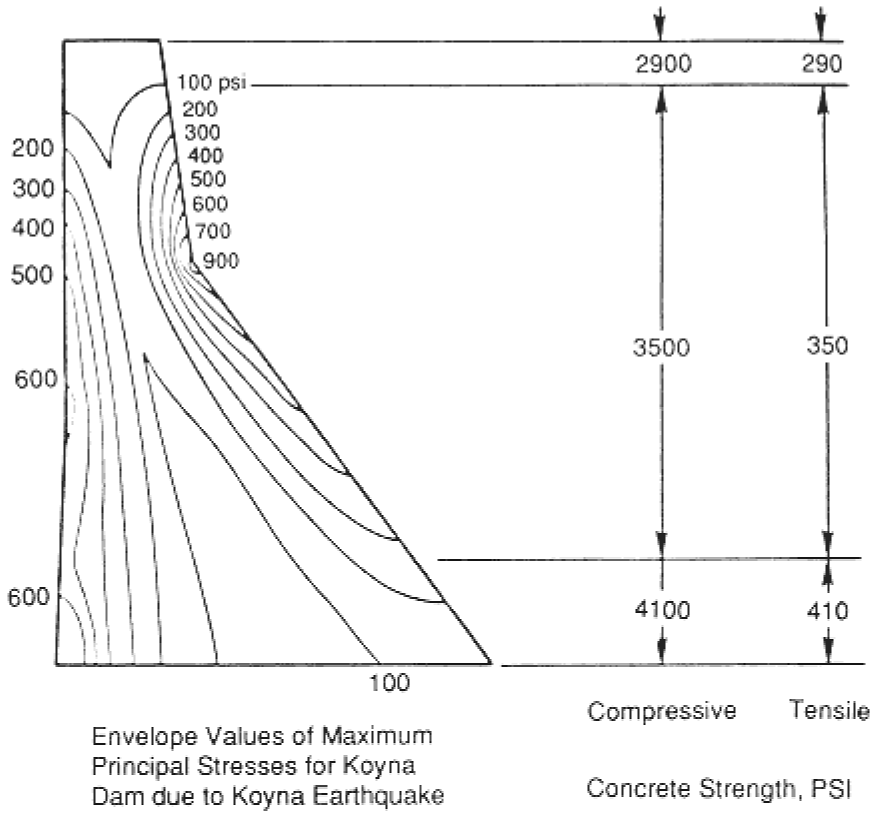


Figure 3-2 Maximum stresses in the tallest Koyna Dam monolith calculated by input of the Koyna earthquake record, compared with the estimated strength of the concrete (3–6).

1. resist minor levels of earthquake ground motion without damage;
2. resist moderate levels of earthquake ground motion without structural damage, but possibly some nonstructural damage; and
3. resist major levels of earthquake ground motion . . . without collapse, but possibly with some structural as well as nonstructural damage.

While these may be appropriate design objectives for buildings, major dams should be designed more conservatively, and this intended conservatism is reflected in the no-tension or at most small-tension limitation used in traditional methods for designing dams. What the traditional methods fail to recognize, however, is that in order for dams to satisfy these criteria during earthquakes, consideration must be given to their dynamic behavior. For linearly elastic structures the dynamic aspect of the response is indicated by the response spectra and by the dynamic displacement patterns, which are conveniently expressed in terms of free-vibration-mode shapes.

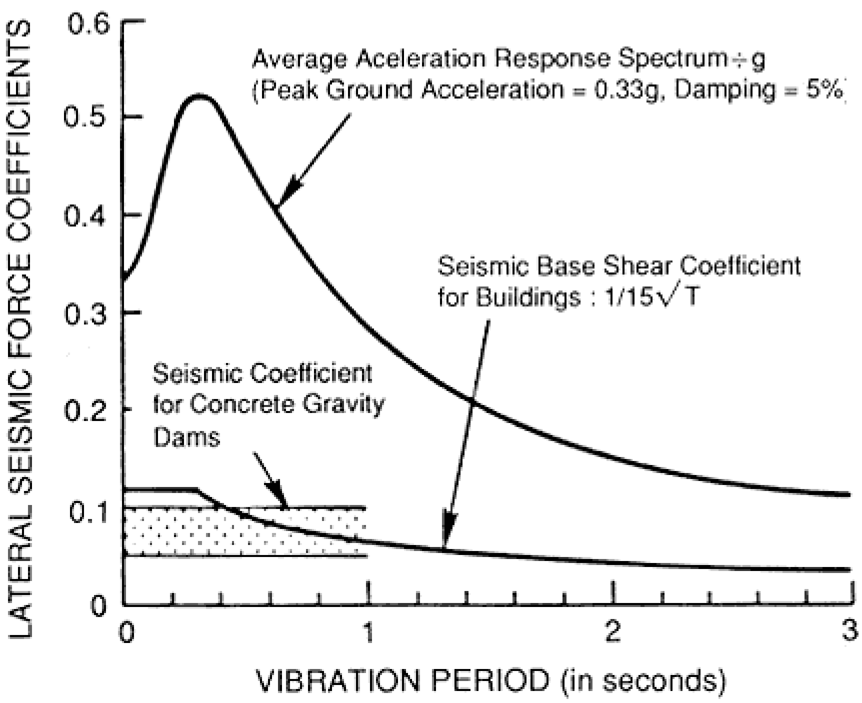


Figure 3-3 Comparison of seismic coefficients used in traditional design with the response spectrum of a strong earthquake (3-29).

In linear analyses the effective modal earthquake forces may be expressed as the product of a seismic coefficient (which depends on the earthquake pseudoacceleration response spectrum and the vibration period of the mode, and varies according to the shape of the mode) and the unit weight of concrete. The seismic coefficient associated with forces in the fundamental mode of a gravity dam varies with height, somewhat as shown in Figure 3-4. For the first two modes of a symmetric arch dam (fundamental symmetric and antisymmetric modes), the coefficient may vary over the dam face, as shown in Figure 3-5. These figures are in sharp contrast with the uniform distribution of seismic coefficient that has been assumed traditionally and that has led to erroneous distribution of lateral forces and hence of stresses in the dam.

One of the results of assuming a heightwise-uniform seismic coefficient is that calculated stresses in gravity dams are found to be greatest at the base of the dam. This has led to the concept of decreasing the concrete strength with increases of elevation, as has been done for some dams (e.g., Koyna Dam in India and Dworshak Dam in the United States). However, the results of dynamic analysis of Koyna Dam (Figure 3-2), as well as the location of the earthquake-induced cracks, demonstrate that the largest stresses actually occur at the two faces in the upper part of gravity dams. Therefore, those are the regions of gravity dams where the highest-strength concrete should be provided if the designer chooses to vary the concrete strength within the dam.

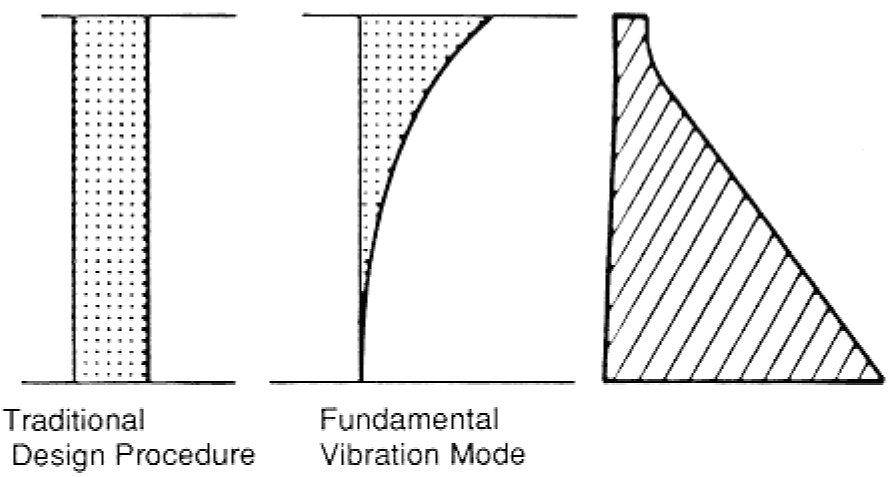
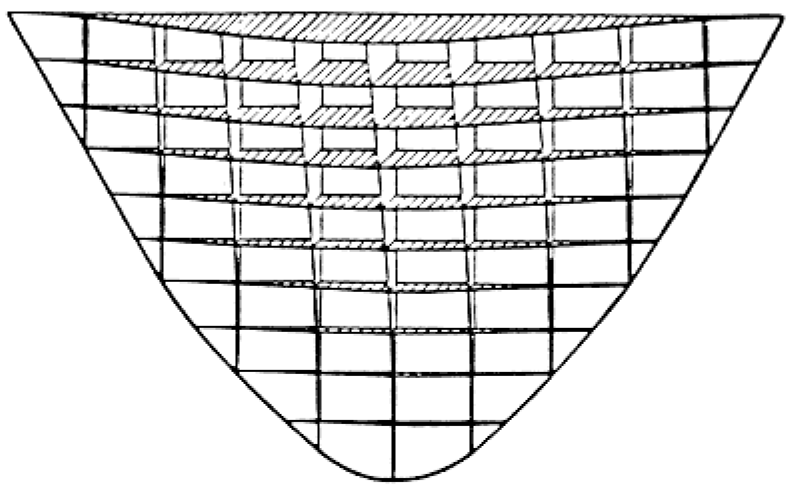
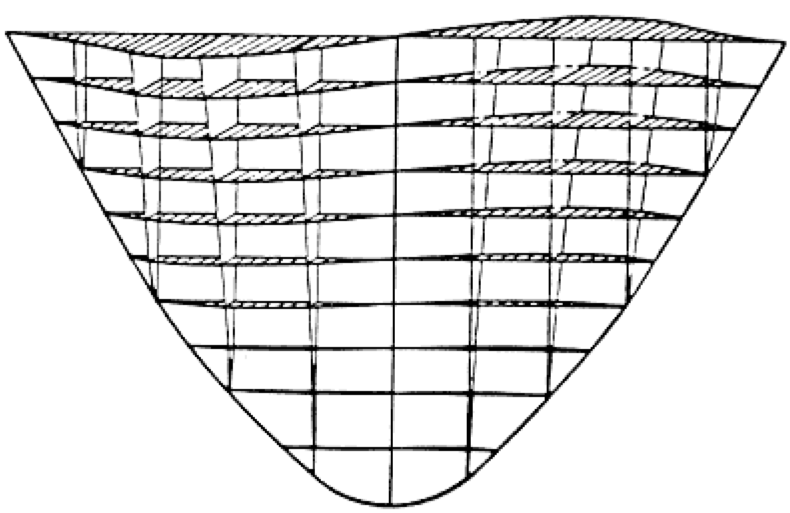


Figure 3-4 Variation of seismic coefficient along height of gravity dams: traditional constant value versus dynamic variation (3-29).

Another undesirable consequence of specifying a heightwise constant seismic coefficient is that the detrimental effects of concrete added near the dam crest are not apparent, as has been shown by analytical study of Pine Flat Dam (3-31). This typical gravity dam, shown in Figure 3-6, was built by the U.S. Army Corps of Engineers in California. As can be seen in the photograph, it was widened at the crest to provide for a roadway. The resulting added crest concrete may appear to have a beneficial effect in reducing the stresses predicted by traditional static analysis and also may serve useful functions in providing freeboard above the maximum water level, in resisting the impact of floating objects, and in affording the roadway. However, because of the sharp increase of seismic coefficient in the crest region (Figure 3-4), the crest mass may cause a dramatic increase in the dynamic stresses—approximately doubling them in the earthquake response of Pine Flat Dam, as shown in Figure 3-7. An interesting consequence of this type of unfavorable response mechanism was seen in monolith 18 of Koyna Dam, which suffered the worst damage during the earthquake; it is believed that this exaggerated damage resulted from an elevator tower that extended 50 ft above the top of the block and therefore was subjected to greatly increased inertial forces.



FUNDAMENTAL SYMMETRIC VIBRATION MODE



FUNDAMENTAL ANTISYMMETRIC VIBRATION MODE

Figure 3-5 Variation of seismic coefficient over face of arch dams (3-12).

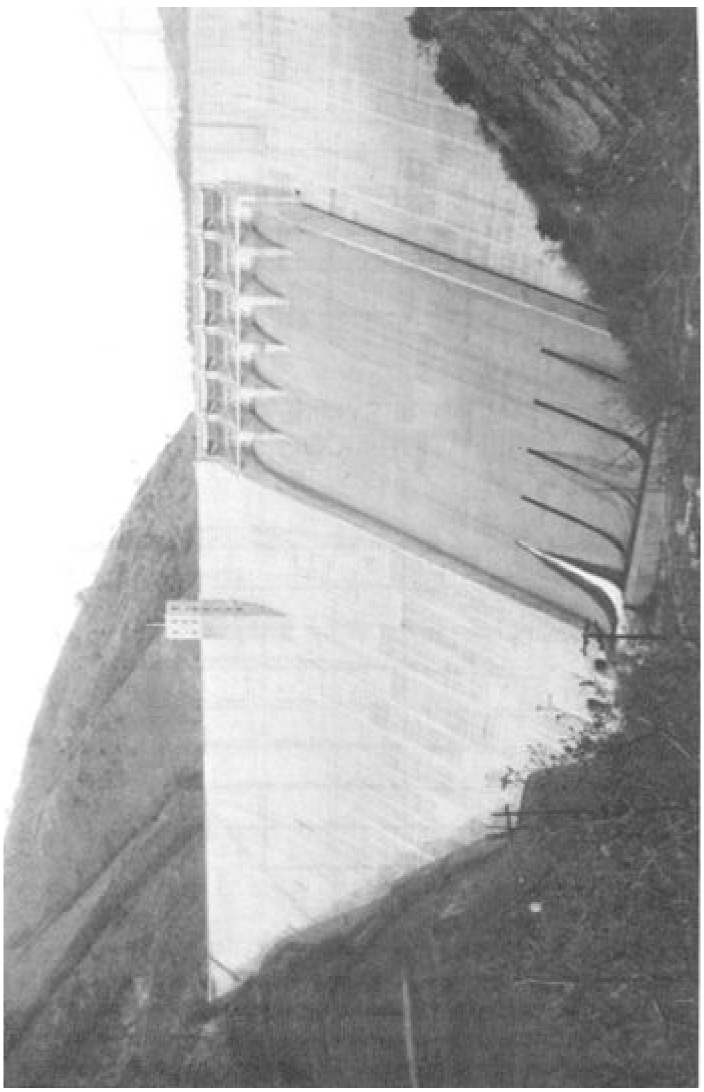


Figure 3-6 Pine Flat Dam, California, was completed by the U.S. Army Corps of Engineers in 1954; it is 429 ft high and 1,820 ft long.

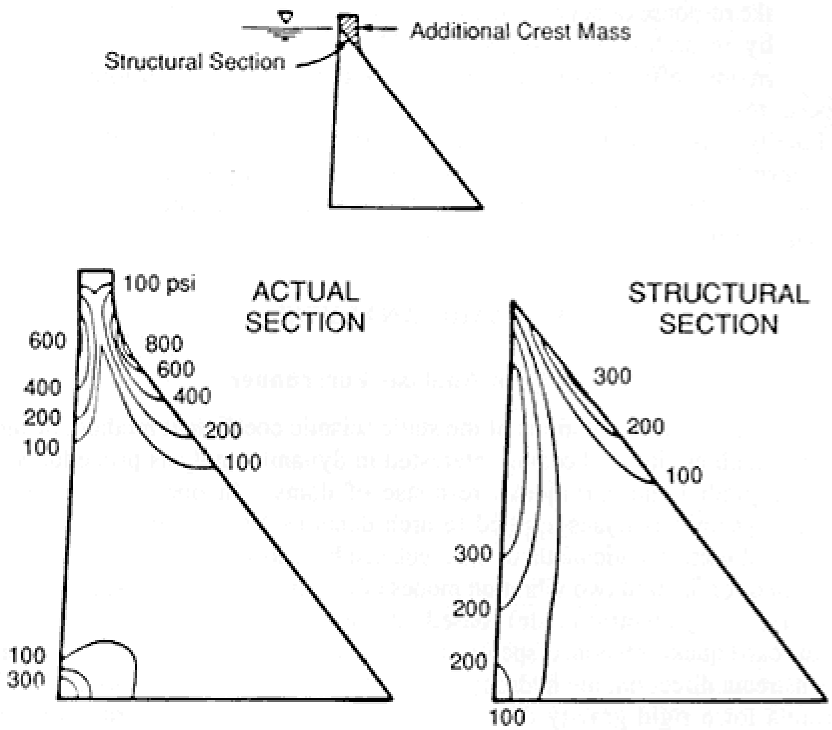


Figure 3-7 Effects of nonstructural crest mass on maximum principal stresses calculated in the tallest Pine Flat Dam monolith due to Koyna earthquake input (3-31).

Traditional design earthquake loading for concrete dams includes seismic water pressures in addition to hydrostatic pressure, as specified by various formulas (3-2, 3-9). These formulas differ somewhat in detail and in numerical values but not in the underlying assumptions, because they are all based on the classical results of Westergaard (3-3) and Zangar (3-4). In a typical formula the seismic water pressure is given as Pe = CSwH, where C is a coefficient that varies nonlinearly from zero at the water surface to about 0.7 at the reservoir bottom, S is the specified seismic coefficient, w is the unit weight of water, and H is the total depth of water. For a seismic coefficient of 0.10 the additional pressure at the base of the dam is 7 percent of the hydrostatic pressure; values at higher elevations also are small. These additional pressures have little influence on the computed stresses and hence on the geometry of the gravity section that satisfies the standard design criteria. On the other hand, when the true dynamic behavior of the dam is considered, including dam-water interaction and water compressibility, numerous analyses have demonstrated that hydrodynamic effects are significant in the earthquake response of concrete dams and can lead to important stress increases, especially in arch dams (3-8, 3-10, 3-11). It is apparent, therefore, that hydrodynamic effects are not properly modeled in the traditional design procedures.

Finally, it is evident that the static overturning and sliding stability criteria that have been used in traditional gravity dam design procedures have little meaning in the context of the oscillatory responses produced in dams by earthquake motions.

DYNAMIC ANALYSIS

Arch Dam Analysis Forerunner

Recognizing the limitations of the static seismic coefficient method, design and research engineers became interested in dynamic analysis procedures to reliably predict the earthquake response of dams. In one of the earliest (1963) dynamic analyses applied to arch dams (3-12), the uniform seismic coefficient used in static methods was replaced by a spatially varying coefficient computed for the first two vibration modes of a symmetric dam (first symmetric and first antisymmetric mode), based on the modal periods and shapes and on an earthquake response spectrum. For ground motion in the upstream-downstream direction, the hydrodynamic effects were based on Westergaard's formula for a rigid gravity dam; for cross-stream motions a formula based on the work of Zienkiewicz and Nath (3-13) was applied. The stress analysis for each mode was done by the trial load method, and the modal stresses were combined appropriately.

Finite Element Modeling

The procedures for earthquake analysis of dams began to change rapidly in the late 1960s with the adoption of finite element modeling procedures, with advances in methods of dynamic analysis, and with the increasing availability of large-capacity, high-speed computers. One of the earliest applications of this new technology to analysis of the earthquake response of an arch dam was reported in 1969 (3-14). In this investigation the dam was modeled as an assemblage of three-dimensional finite elements on a rigid base, and the impounded water was modeled as a mesh of incompressible liquid elements. The dynamic response was calculated by the mode-superposition method.

Subsequently, the Bureau of Reclamation funded the development of a computer program based on similar concepts intended specifically for the static and dynamic analysis of the Bureau's arch dams (2-5). This program, called ADAP (Arch Dam Analysis Program), modeled the dam body by solid elements designed to represent the arch dam geometry efficiently. In addition, the program provided for modeling the flexibility (but not the mass) of the foundation rock, either by means of Vogt coefficients, which treat the foundation as an elastic half space, or by a finite element mesh representing a specified block of foundation rock. When the project was planned, it also was intended to model the interaction with water in the reservoir by means of a mesh of incompressible liquid elements, as mentioned above. However, this final phase of the research was not funded, and in practice the impounded water was modeled by Westergaard-type added masses modified to take account of the doubly curved upstream face (3-15).

In the mid-1970s a dynamic finite element analysis procedure, including a different Westergaard-type added mass, was described in Section 4-32 of a 1976 Bureau of Reclamation publication on gravity dams (3-16) and in Section 4-56 of a 1977 Bureau of Reclamation publication on arch dams (3-12). In the latter publication the same added hydrodynamic mass was used in response analysis for both the upstream-downstream and the cross-stream components of ground motion; the obvious inconsistency of this assumption was recognized in Section 4-55 of the publication, but no better alternative was proposed.

Foundation Model Deficiency

The two principal limitations of the finite element modeling of a dam-foundation system, mentioned in Chapter 2 of this report, were revealed by extensive work done during the past 20 years in the seismic analysis of nuclear power plants. First, the boundary hypothesized at some depth to define the foundation rock region included in the analysis is typically assumed to be rigid. For concrete dam sites, where similar competent rock usually extends to great depths and there is no obvious "rigid" boundary such as may be assumed at a soil-rock interface, the assumption of a rigid boundary may result in serious distortion of the foundation interaction effects; this distortion results mainly because energy loss (damping) associated with radiation of vibration waves beyond the assumed foundation block is not properly represented. These effects may significantly reduce the seismic stresses in the dam (3-8).

Second, the earthquake input is usually applied to the dam-foundation model as prescribed motions of the rigid foundation block boundary, as discussed in Chapter 2. Clearly, the earthquake motions at depth in the foundation rock will not be the same as the free-field motions recorded at the ground surface; hence, the dynamic analysis procedure should be formulated to use some different specification of the seismic input, as described in Chapter 2. Ideally, the earthquake input should be specified as spatially varying motion at the dam-foundation rock interface, but this has usually not been possible for lack of appropriate instrumental records from past earthquakes.

Limitations of Reservoir Water Interaction Models

As mentioned above, the traditional static analysis procedures represent the effect of the impounded water during earthquake excitation by means of added masses calculated from Westergaard's classical formula. However, Westergaard's added mass result applies rigorously only to the case of a rigid dam with planar vertical face, and the analysis underlying the result implicitly neglects the effects of water compressibility. Although the concept has long been used in practical design, these limitations often have not been understood; extensive studies during the past two decades have been devoted to obtaining better understanding of the dam-water interaction problem.

From the first it was evident that the Westergaard rigid dam assumption is not consistent with the concept of dynamic earthquake response analysis, so efforts were made to formulate procedures for dynamic analysis considering dam-water interaction arising from dam flexibility. Results of these studies have contributed greatly to the understanding of dam-water interaction (3-10). The first attempt to deal with water interaction for dams having curved faces made use of a liquid finite element model of the reservoir, as mentioned above (3-14). Because this numerical approach eliminated the geometric limitation inherent in the Westergaard formulation, it was adopted subsequently in the ADAP computer program (3-15). Using this extended program, it was shown that the geometry of the dam face and of the reservoir boundaries can have a significant effect on the hydrodynamic forces applied to the dam (3-15). In this work the water was assumed to be incompressible, and because this assumption greatly simplifies the analysis of dam-water interaction, it continues to be used in some research (3-26) and in many practical earthquake response analyses of concrete dams (3-18).

On the other hand, earlier work on gravity dams indicated that the compressibility of impounded water might play an important part in the interaction mechanism (3-10, 3-19); further research along these lines has confirmed the significant contribution of water compressibility in the earthquake response of most concrete dams. The key parameter that determines the significance of water compressibility in the earthquake response of dams is the ratio of the fundamental natural frequency of the impounded water to the fundamental natural frequency of the dam alone. If this ratio is large enough (greater than 2 for gravity dams), the impounded water affects the dam response essentially as an incompressible fluid.

However, this frequency ratio usually is less than 2 for dams with realistic values for the elastic modulus of concrete; hence, water compressibility is expected to be important in the earthquake response of both arch dams (as shown in Figure 3-8) and gravity dams (figure not included). Water compressibility has less influence on the earthquake response of dams with lower concrete moduli, to the point of becoming negligible where the modulus is taken to be unrealistically low, as shown in Figure 3-9. However, even for such unrealistically flexible concrete, the compressibility can have a significant effect on the response to vertical ground motion (Figure 3-9). These data and additional results (3-20) indicate that water compressibility is expected to be significant in the response of most concrete dams. Thus, the typical added mass representation of hydrodynamic effects that is based on assumed incompressible water (as, for example, in the ADAP computer program) may lead to inaccurate results whether the added mass is calculated from some modification of Westergaard's classical result for a straight dam or is based on a three-dimensional finite element analysis of the liquid domain.

On the other hand, the influence of water compressibility will vary from one dam to another, and in some cases the effect may not be large. Comparison of the top two stress contour plots of Figure 3-10 shows that water compressibility had little effect on the calculated seismic response of Monticello Dam (3-26) when the reservoir bottom was assumed to be rigid. In that case the compressibility was the only important difference between the analyses done by ADAP and those done by a similar program named EACD-3D (2-3).

This Monticello Dam example and also the results shown in Figure 3-11 for Morrow Point Dam demonstrate another limitation of incompressible water formulations. That is, they cannot recognize the potential absorption of hydrodynamic pressure waves into the sediments and rock at the reservoir boundaries. These results show that when reservoir boundary wave absorption effects are considered, significant reductions of the dam stresses may be indicated. It is apparent from these figures that these wave absorption effects can be significant in the response of arch dams and are particularly important in the response to the vertical component of the earthquake motions.

PRESENT KNOWLEDGE AND CAPABILITIES

Conclusions About Interaction Effects

The extensive research that has been devoted during the past 20 years to evaluating hydrodynamic and foundation interaction effects has led to greatly increased understanding of these phenomena. It is now generally recognized that:

1. The earthquake response of concrete dams is increased significantly by interaction with the impounded water, with the hydrodynamic contribution being especially large in response to the vertical earthquake component.
2. Hydrodynamic effects usually are more significant in the earthquake

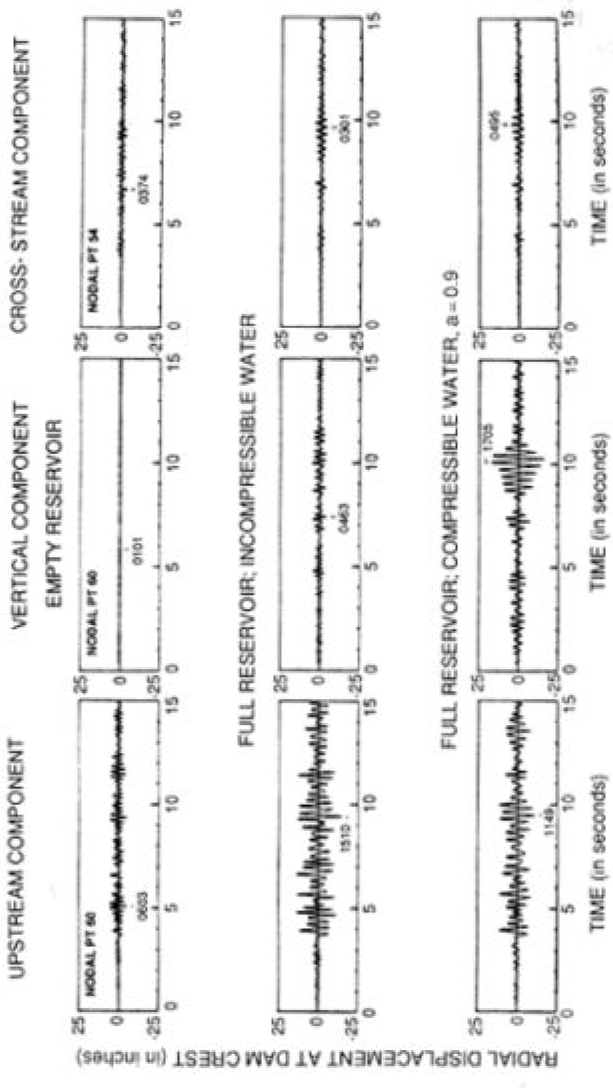


Figure 3-8 Calculated effect of water compressibility on displacement response of Morrow Point Dam due to Taft earthquake record: concrete modulus = 4 × 106 psi (3-20).

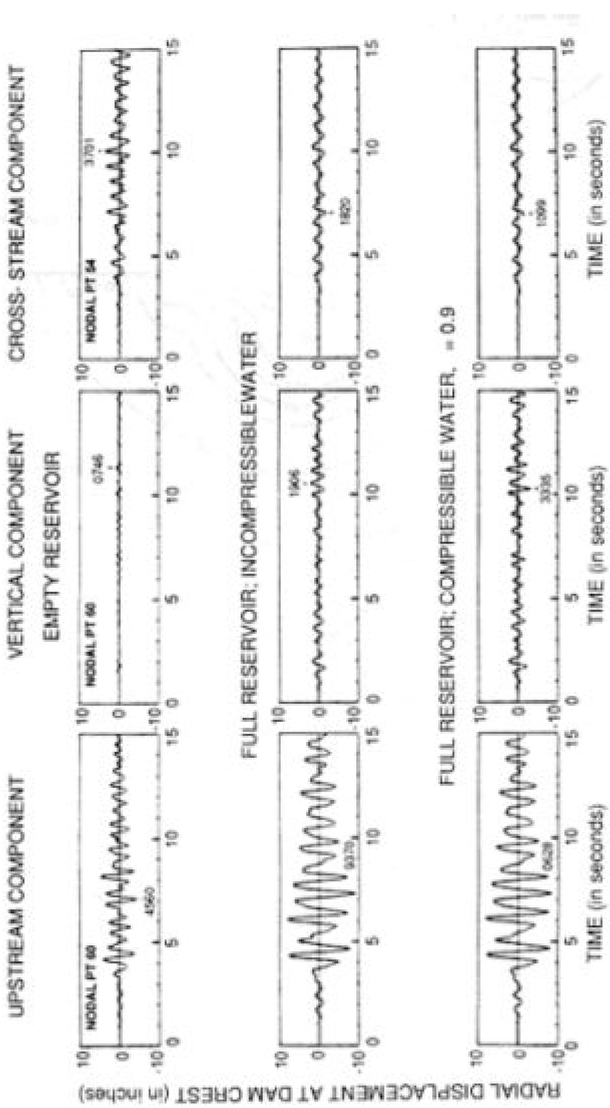


Figure 3-9 Calculated effect of water compressibility on displacement response of Morrow Point Dam due to Taft earthquake record: concrete modulus = 0.5 × 106 psi (3-20).

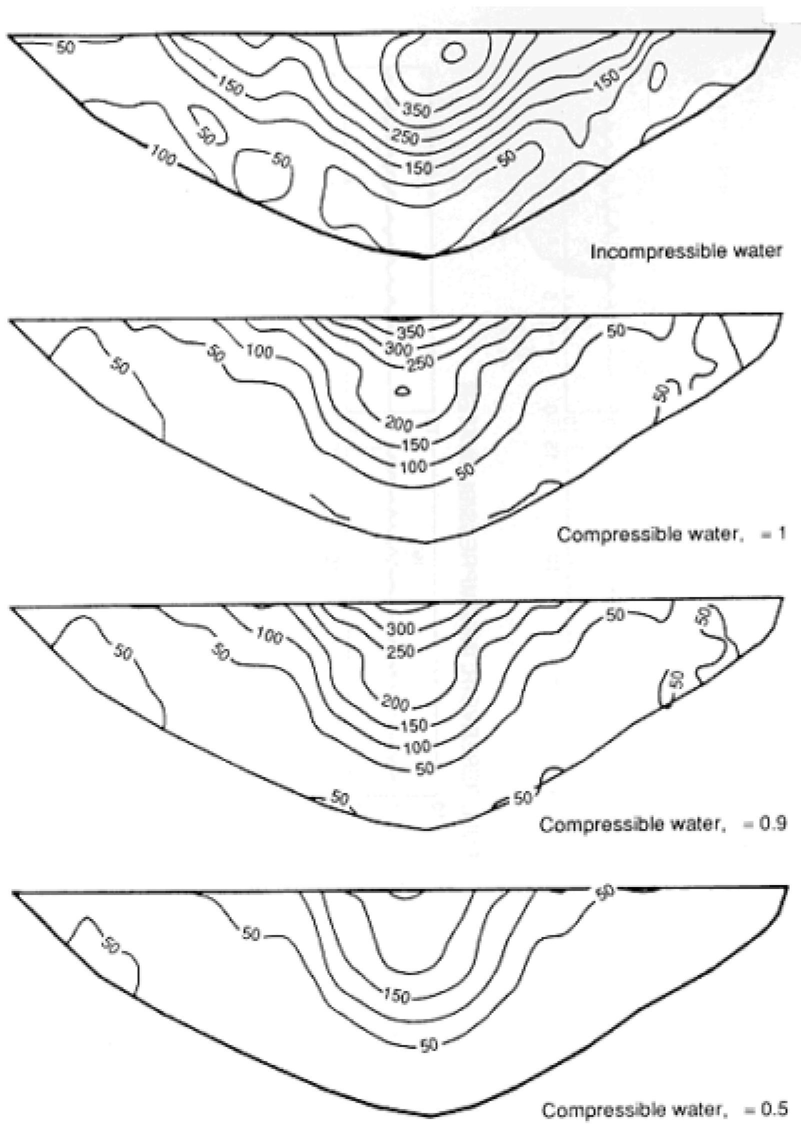


Figure 3-10 Calculated effects of water compressibility and reservoir boundary absorption on upstream face maximum envelope stress contours for Monticello Dam subjected to Morgan Hill earthquake record (3-26).

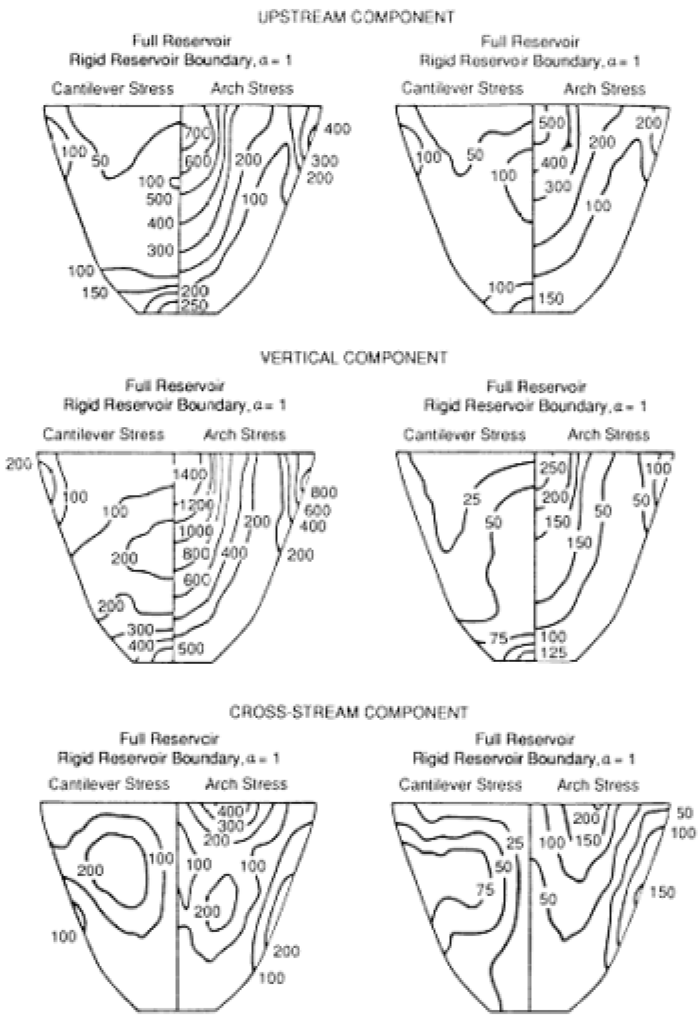


Figure 3-11 Calculated effects of reservoir boundary absorption on upstream face maximum envelope stress contours for Morrow Point Dam due to Taft earthquake record (3-11).

response of a slender arch dam than for a massive gravity dam.

3. The assumption of water incompressibility that is commonly used in practical analysis may lead to errors on either the conservative side or the unconservative side for upstream-downstream earthquake motion, but it is more likely to be unconservative in predicting response to the vertical and cross-stream components of motion (3-10, 3-20, 3-21).
4. An important deficiency of the incompressible water approximation is that the hydrodynamic-wave absorption effects of the underlying rock or reservoir boundary sediments cannot be taken into account, and it has been shown that neglecting boundary wave absorption may lead to unrealistically large estimates of seismic response.
5. Neglecting the dynamic interaction of gravity dams with deformations of the foundation rock also will generally lead to overestimation of the seismic response. Because arch dams resist the reservoir water pressures and the thermal and earthquake forces, at least in part, by transmitting them by arch action to the canyon walls, dam-foundation rock interaction also is likely to be significant in the earthquake response of arch dams, possibly more so than in the case of gravity dams.

The preceding observations lead to the conclusion that dam-reservoir water interaction, including water compressibility and pressure wave absorption at the reservoir boundaries, and dam-foundation rock interaction all should be considered in the earthquake response analysis of concrete dams.

Dynamic Analysis Procedures and Computer Programs

Two-Dimensional Analysis

Analysis procedures and computer programs that take account of all of these factors have been developed for evaluating the earthquake performance of concrete dams idealized as two-dimensional systems (2-4, 3-22), and these procedures and programs should be used in those cases where two-dimensional analyses are appropriate because of their computational efficiency. In gravity dams with plane vertical joints the monoliths tend to vibrate independently, as evidenced by the spalled concrete and water leakage at the joints of Koyna Dam during the 1967 earthquake (3-5, 3-6); hence, a two-dimensional plane-stress model of the individual monoliths is usually appropriate for predicting the response of such dams to moderate or intense earthquake motions. In some cases of gravity dams built in a broad valley, especially roller-compacted concrete dams built without vertical joints, a plane-strain idealization may be adopted in place of the plane-stress model. On the other hand, if the gravity dam has effectively keyed contraction joints or is located in a narrow canyon, the assumption of independent response of the blocks may not be appropriate.

One computer program that has been developed to perform the earthquake analysis of two-dimensional dam-water-foundation rock systems, called EAGD-84, is based on a substructure formulation of the problem (2-4, 3-22). In this program a finite element idealization is used to model arbitrary geometry and variations of material properties of the dam; consequently, both overflow and nonoverflow sections and also appurtenant structures can be modeled satisfactorily. The impounded water is treated as a continuum in order to efficiently represent its large extent as well as the radiation of hydrodynamic pressure waves upstream. The effects of alluvium and silts that accumulate at the bottom of the reservoir and of the underlying rock are modeled approximately by a boundary that may partially absorb the incident hydrodynamic pressure waves; more rigorous methods for treating this effect without such a simplifying approximation have recently become available (3-23). The foundation rock supporting the dam is idealized as a viscoelastic half-plane continuum that, as mentioned earlier, accounts for the energy radiation effects of the dam-foundation rock interaction. With this computer program a complete interaction analysis can be performed of the dynamic response of a gravity dam to the upstream and vertical components of free-field earthquake motion, with both components acting simultaneously at the dam-foundation rock interface.

Three-Dimensional Analysis

As mentioned earlier, computer programs employing finite element idealizations for the earthquake analysis of arch dam-water-foundation systems have been in use for as long as two decades. ADAP (2-5), the first widely available program developed specifically for dynamic analysis of arch dams, also uses a mesh of finite elements to model the foundation rock. However, as mentioned earlier, these elements are assumed to be massless, so they model the foundation flexibility but do not account for wave propagation in the rock and the consequent radiation damping effect. Also mentioned earlier is the fact that the liquid finite elements used in recent versions (3-15) of this program are assumed to be incompressible. Thus, the water compressibility effects and hydrodynamic wave absorption effects of the reservoir boundary, which as stated earlier can be significant in the seismic response, are not considered in ADAP.

These limitations are overcome in a computer program that is based on the substructure method and was developed recently for the three-dimensional analysis of concrete dams. The program, named EACD-3D (2-3, 3-24), accounts for dam-water interaction, including the effects of compressibility and reservoir boundary pressure wave absorption, using procedures analogous to those employed in the two-dimensional program EAGD-84. So far, however, it has not been possible to take full account of the dam-foundation rock interaction. All three substructures—dam, reservoir water, and supporting rock—are idealized as finite element systems to represent the complicated dam geometry and site topography, but special techniques were introduced to efficiently recognize the great upstream extent of the reservoir. The massless finite element model of the foundation rock is similar to that used in ADAP, and thus it also is deficient in representing radiation energy loss. On the other hand, recent research in Japan (3-27) has been directed toward modeling of arch dam-foundation rock interaction, but it has not yet advanced sufficiently to be of use in practical arch dam earthquake response analyses.

Both EACD-3D and ADAP can be used to perform a complete dynamic analysis of a concrete arch dam subjected to the simultaneous action of upstream, vertical, and cross-stream components of the free-field motion specified at the interface between dam and foundation rock. In the model with massless foundation rock the free-field surface motion at the dam-rock interface is the same as the motion at the rigid boundary of the foundation block. In principle, spatial variation of the input earthquake motions could be specified either for the free-field input used in EACD-3D or for the foundation block boundary input used in ADAP, and it is evident that such spatial variation does occur across dam sites, as discussed in Chapter 2. However, reliable descriptions of the earthquake motions to be expected at such locations are not available at present, and "multiple-support" excitation is seldom used for arch dam analysis, even though it is technically feasible (3-25).

Selection of numerical values for the parameters necessary to describe a dam-water-foundation rock system for analysis by the aforementioned computer programs should be based on appropriate experimental tests. Clearly, the properties of the reservoir water present no problem, and the properties of the concrete comprising the dam can be defined adequately by standard procedures. Evaluation of the elastic modulus and damping of the foundation rock is not so simple, but, as discussed in Chapter 5, numerous field measurement studies have demonstrated that vibration properties calculated using typical finite element system models agree well with measured values. However, techniques are not presently available to determine the reservoir wave reflection coefficient, which is seen in Figures 3-10 and 3-11 to have a major influence on the seismic stresses. Until such measured data are available, it is suggested that values for this coefficient be calculated based on the properties of the water and the underlying rock, as described in references 3-22 and 3-24. This approach neglects the unknown cushioning effect of reservoir boundary sediments and thereby will probably overestimate the earthquake response in most cases.

The procedures applied in the analysis of arch dams can be used to evaluate the earthquake response of other types of dams that also must be modeled as three-dimensional systems; buttress dams (including both flat slab and multiple arch dams) are a typical example. In the early decades of this century, buttress dams were often used in preference to gravity dams because they require about 60 percent less concrete. But as stated in a Bureau of Reclamation publication (3-28, p. 10), "the increased formwork and reinforcing steel required usually offset the savings in concrete. . . . [Hence, this] type of construction usually is not competitive with other types of dams when labor costs are high." However, even though new dams of this type are not being built in the United States, it still is necessary to carry out seismic safety evaluations of the existing structures. Littlerock Dam in California, shown in Figure 3-12, is a typical multiple arch water supply and flood control dam for which a seismic safety evaluation has been performed.

Simplified Dynamic Analysis Procedures

While the response history analysis procedures described above are appropriate for final-stage analyses of the seismic safety of existing dams and proposed new dams, simplified analysis procedures would be preferable for the preliminary evaluation or design stages.

In response to this need a simplified procedure was developed in 1978 for the analysis of gravity dams in which the maximum response due to the fundamental mode of vibration was represented by equivalent lateral forces computed directly from the earthquake design spectrum (3-29). Recently, this simplified two-dimensional analysis of the fundamental mode response has been extended to include the effects of dam-foundation rock interaction and wave absorption at the reservoir bottom (3-30), in addition to the effects of the compressible water-dam interaction considered in the earlier procedure. Also now included in the simplified procedure is a "static correction" method to approximate the response contributions of the higher-vibration modes. The simplified procedure is sufficiently accurate for preliminary design and safety evaluation of gravity dams.

While many of the basic concepts underlying the procedure may be applicable to arch dams, the extension of such a method to treat three-dimensional systems is likely to be very difficult for several reasons: (1) the geometry of arch dams varies considerably from one site to another, thereby reducing the possibility of defining "standard" values for vibration properties and parameters, and (2) their response is generally not dominated by a single mode of vibration.

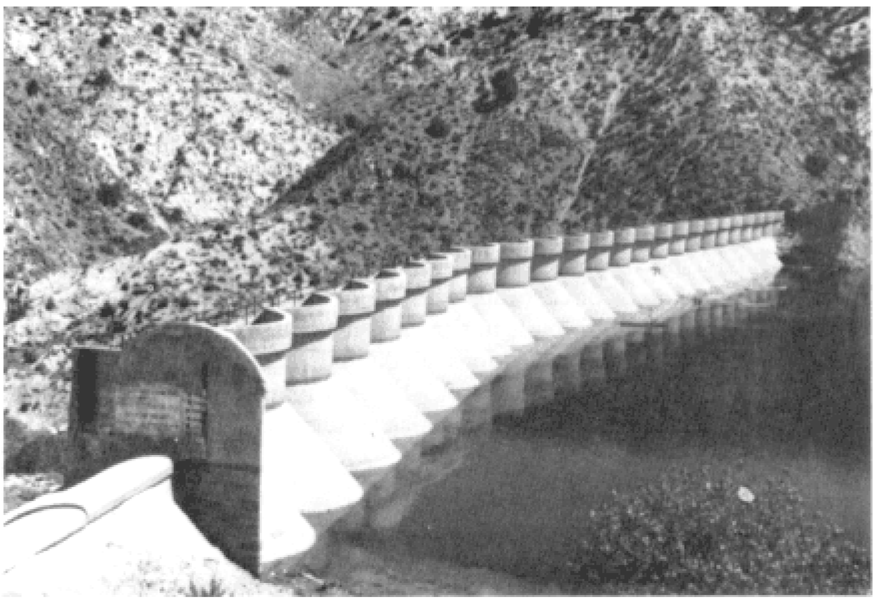


Figure 3-12 Littlerock Multiple Arch Dam, California, completed in 1924 for use in irrigation, has a maximum height of 168 ft and length of 800 ft.

RESEARCH NEEDS

Although considerable progress has been made in the past 20 years, much additional research needs to be done to improve the reliability of methods for the seismic analysis, design, and safety evaluation of concrete dams. To meet this objective the following tasks should be pursued:

1. Instrumentation

Major dams in seismic areas of the United States should be instrumented to measure their responses during future earthquakes. The instrumentation should be designed to provide adequate information on the characteristics and spatial variation of the ground motion at the site, on the response of the dam, and on the hydrodynamic pressures exerted on the dam. Because of the urgent need for such data, dams in highly seismic regions of other countries also should be considered for instrumentation. This effort should be coordinated with the plans for the seismic arrays recently installed in Taiwan, India, and the People's Republic of China under cooperative agreements with the United States.

2. Field Forced-Vibration Tests

Forced-vibration tests should be conducted on selected dams—using more than one water level where feasible—and the resulting hydrodynamic pressures and motions of the structures and their foundations should be recorded and analyzed.

3. Evaluation of Analytical Methods for Response Analysis

Existing analytical methods for computing the response of all types of concrete dams to earthquakes should be evaluated by comparing calculated results with the responses recorded during forced-vibration tests and, more importantly, during actual earthquakes when significant ground motions are recorded at appropriate dam sites. If necessary, the methods should be refined and the computer programs needed for their implementation prepared in a form convenient for application in engineering practice.

4. Improvement of Arch Dam Analysis

The methods used for input of the earthquake motions in present methods of earthquake analysis of arch dams urgently need improvement. Similarly, improvements are needed in procedures used to account for the interaction between arch dams and their supporting foundation rock.

5. Simplified Analysis Procedures

Simplified analysis procedures should be developed that are suitable for the preliminary phase of design and safety evaluation of arch dams.

6. Evaluation of Dynamic Sliding and Rocking Response of Gravity Dams

Analysis procedures should be developed to determine the dynamic sliding and rocking response of gravity dam monoliths. Utilizing these procedures, rational stability criteria should be derived, replacing the traditional sliding and overturning criteria that do not recognize the oscillatory response of dams during earthquakes.

4

Nonlinear Analysis and Response Behavior

INTRODUCTION

As stated in Chapter 3, useful information about the expected earthquake response of concrete dams can be obtained from linear dynamic analysis based on assumed elastic behavior. However, a severe earthquake may cause significant damage to concrete dams, as was observed at Koyna Dam, and such damage usually is associated with important changes in structural stiffness. Consequently, if significant damage occurs, the actual performance of the dam can be predicted only by a nonlinear analysis that takes account of these stiffness changes. For this reason a rigorous evaluation of concrete dam seismic performance should consider the behavior of the materials and components in the system, which is characterized by nonlinear force-displacement relationships, in conjunction with nonlinear response analysis procedures. Additionally, realistic consideration of the nonlinear behavior of dams may indicate mechanisms that limit the earthquake response and provide an added margin of safety against failure.

After definition of the seismic input, a comprehensive safety evaluation of a concrete dam requires (1) identification of the behavior of materials and components under dynamic loads, (2) mathematical models that can represent the nonlinear behavior, (3) efficient numerical procedures for computing the nonlinear earthquake response of the dam system, and (4) criteria to assess acceptable performance and evaluate failure modes. In this chapter the nonlinear behavior important in the earthquake response of concrete dams and the requirements for nonlinear analysis are identified. In addition, recent developments in modeling and analysis of such dams are summarized, and further research needed to improve the seismic safety evaluation is outlined.

CHARACTERISTICS OF NONLINEAR BEHAVIOR

Concrete exhibits a complicated nonlinear relationship between stress and strain that is dependent on loading rate and history. The nonlinear behavior of mass concrete becomes significant as stresses approach the compressive strength or the comparatively small tensile strength. Because concrete dams are designed to resist their primary loads—gravity and hydrostatic—through compressive stress fields, the tensile stresses induced by such static loads are minimal (or nonexistent). Moreover, when current design criteria for static loads are followed, the computed compressive stresses are found to be much less than the compressive strength of the concrete.

However, strong earthquake ground motion can produce large dynamic stresses in dams, both compressive and tensile, and the combination of static and dynamic stresses may exceed the linear response range of the concrete, particularly with regard to the tensile stresses. Experiments demonstrate that concrete behavior is essentially linear under cyclic compressive loads up to approximately 50 to 60 percent of the compressive strength (4-1). For example, assuming a compressive strength of 4,000 psi, nonlinear behavior is important for compressive stresses greater than about 2,000 to 2,400 psi. But linear elastic dynamic analyses of gravity and arch dams show that compressive stresses rarely exceed this range during typical earthquakes; consequently, the nonlinear behavior of concrete in compression, including hysteretic energy dissipation, can generally be neglected in the earthquake response analysis of dams.

Tensile stresses in a dam produced by earthquake ground motion can be resisted only by the tensile strength of concrete, because reinforcement generally is not provided in the body of a dam. The tensile strength of concrete is an order of magnitude less that its compressive strength, and linear analyses demonstrate that the tensile limit may be exceeded at widespread different locations in a dam during an earthquake (3-8, 3-11). For example, in a recent study (4-2) a gravity dam monolith subjected to a wide range of earthquake ground motions was analyzed assuming linear behavior. A moderate earthquake with a peak ground acceleration of 0.25 g induced tensile stresses greater than the tensile strength of typical concrete in both the top part of the dam and the heel, indicating the potential for cracking.

As the stress in a dam approaches the tensile strength of the concrete, microcracks (which are always present in concrete) coalesce to form a crack surface. However, it is important to note that the tensile cracking caused by moderate earthquakes may not be deleterious to the performance of a dam, because the dynamic forces open and then reclose the cracks during a cycle of vibration. After the earthquake, static loads will generally return the stresses to compression, leaving the cracks in a closed condition and maintaining the dam's stability. Because tensile cracking in concrete is fundamentally a fracture process that dissipates energy as the cracks propagate, the amplitude of dynamic response may be decreased by the cracking if the dam remains stable during the earthquake. During severe ground motion, however, it is possible for tensile cracks to propagate completely through a dam, potentially leading to dynamic instability and to uncontrolled release of water. Both linear analyses and model studies of arch dams have indicated related failure modes: extensive tensile cracking that results in the formation of a semicircular or rectangular notch near the crest, leading to dynamic instability of the notched portion. Although it is not certain that such a failure can actually develop in an arch dam due to an earthquake, or that crack propagation completely through a dam section necessarily results in instability, the major goal of a safety evaluation is to determine whether unstable response could develop due to credible levels of earthquake ground motion.

Realistic analytical modeling of tensile cracking in mass concrete should recognize the variation in properties of the materials in a dam, the incremental construction procedure, and the effect of pore water on crack development. The practice adopted in some designs, as mentioned in Chapter 3, of varying the concrete strength over the dam height may affect the locations where tensile cracks develop. Also, contraction joints between the monoliths, as well as horizontal planes of weakness that may exist at lift joints, can be a major influence on crack location. The presence of pore water in saturated mass concrete affects the stress state and the initiation and propagation of cracks, particularly in the lower portions of a dam and at the interface with the foundation rock. Generally only the static pore water pressure is considered in an earthquake analysis, as a constituent of the combined (static plus dynamic) state of stress. The earthquake response is associated with dynamic total stresses that include both intergranular and pore pressure components, but typically it is assumed that the pore water does not migrate during the cyclic pressure changes because of their short duration. Consequently, it is usually assumed that the earthquake does not significantly influence the static pore pressure effects. It is recognized that some sort of hydraulic fracture mechanism might result at the wetted face of the dam due to dynamic pore pressures during the earthquake, but there is no evidence that such hydraulic fracturing has actually occurred. Thus, present practice assumes that pore pressures have no direct effect on dynamic cracking and that the cracking behavior may be characterized simply in terms of the dynamic total stresses.

The preceding comments on concrete cracking pertain to both gravity and arch dams; however, the stress state in many gravity dams tends to be essentially uniaxial (cantilever stresses), whereas the stresses in arch dams are considered to be biaxial, involving components in both the cantilever and arch directions. On the other hand, the development of arch-direction tensile stresses is inhibited by vertical contraction joints that are provided between monoliths, because the joints are unable to resist any significant net tension. The joints tend to open and close during a severe earthquake (3-17), and this nonlinear response mechanism has two consequences in the earthquake response of arch dams. First, the intermittent opening temporarily reduces the arch-direction resistance, causing transfer of load to both cantilever action and inertial resistance. The load picked up by cantilever bending may then lead to flexural overstress and failure of the monoliths. The second potential consequence of the joint opening-closing is compressive failure of the joint itself. Model tests have shown that loss of joint integrity is possible in arch dams (4-3), because as a joint opens the compressive stresses in the portion of the joint remaining in contact may increase dramatically, possibly crushing the concrete. On the other hand, the nonlinear behavior of the joints may limit the earthquake response of large (long-period) dams by further lengthening the vibration period, thereby reducing the dynamic amplification that is a function of the vibration properties of the dam and characteristics of the ground motion (3-17).

The behavior of the foundation rock supporting a dam is typically nonlinear, because the rock is often fractured and discontinuous. The nonlinear behavior of foundation rock will affect the static and dynamic response of a dam. For example, an elastic modulus based on small strain overestimates the stiffness of fractured foundation rock because of the rock's inability to resist large tensile stresses. During a severe earthquake the forces acting at the abutments of an arch dam also can increase significantly. In some cases it may be possible for the abutments to fail because of shear failure along planes of weakness in the foundation rock. Questions about foundation stability are a safety concern for gravity dams as well. Thorough consideration of the potential for foundation rock failure is often difficult, because information on subsurface rock conditions is limited, particularly in the case of older dams. However, foundation stability must always be studied intensively, because experience shows that actual failures of concrete dams generally are initiated in the foundation rock.

The response predicted by a linear model of a concrete dam depends on both the spatial variation of earthquake ground motion (as described in Chapter 2) and the temporal variation. Analytically, the response of a linear dam system can be considered as the summation of responses to harmonic components of the ground motion specified for one or more support degrees of freedom. The nonlinear response of concrete dams may depend on other characteristics of the earthquake ground motion. For example, it may be affected by the duration of the ground motion and the amplitude of incremental ground velocity, particularly in the propagation of tensile cracks and the integrity of joints. Crack propagation under dynamic loads is affected by the number of loading cycles, which is related to the duration of ground motion. Similarly, the integrity of a joint may depend on the number of times it opens and closes. A large increment of ground velocity, typical of ground motion near an earthquake source, will impart an impulse to a dam that can initiate and propagate tensile cracks or affect the dynamic stability after extensive cracking. For these reasons a realistic safety evaluation based on nonlinear analysis would require a detailed seismotectonic study of a dam site and the development of appropriate site-specific ground motion records.

The nonlinear behavior mechanisms mentioned above (tensile cracking of concrete, loss of joint integrity, and foundation failure) result from dam vibration in response to earthquake ground motion. Overall sliding and overturning of a dam due to ground motion, the traditional criteria for static stability, are generally unrealistic failure modes during earthquakes. Indirect earthquake failures of dams due to fault displacement or overtopping also are important considerations, but they are not directly related to dynamic response.

MATERIAL MODELS AND RESPONSE ANALYSIS

As discussed in Chapter 3, linear dynamic analysis of a concrete dam system is valuable in understanding the general characteristics of earthquake response. However, if the linear analysis shows repeated tensile stresses significantly greater than the tensile strength, the concrete can be expected to crack, and the linear response results would no longer be valid. The ultimate earthquake behavior cannot be predicted from such linear models, because the strength of the materials is not represented, nor is the redistribution of forces due to tensile cracking, joint opening-closing, or foundation instability.

A realistic nonlinear earthquake response evaluation of a concrete dam requires analysis of mathematical models that include tensile crack propagation in concrete, cyclic displacement behavior of joints and abutments, and deformability of the foundation rock. Because these phenomena induce nonlinear relationships between resisting forces and displacements, nonlinear methods of response analysis are required. These are considerably more complicated than linear analysis. A rigorous nonlinear response analysis should include the important dam-water and dam-foundation rock interaction effects mentioned in Chapter 3, together with nonlinear models of the dam concrete, joints, and foundation rock. The finite element method is presently recognized as the best approach for spatial discretization of the equations of motion for concrete dams because of its ability to represent arbitrary geometry and to incorporate arbitrary variations of material behavior. Finite elements can also be used to discretize the reservoir water and foundation rock, although alternatives such as boundary integral elements may be more efficient in modeling large reservoir and foundation regions.

When concrete dam systems exhibit nonlinear response to severe earthquake ground motion, the principle of superposition is no longer valid, and frequency-domain analysis may not be applied directly. Usually, the equations of motion are solved in the time domain using a time-stepping procedure. Because of the large amount of computation required to obtain the nonlinear dynamic response of a concrete dam system, it is mandatory to truncate the size of the water and foundation rock domains; however, the boundaries should be modeled in a manner that allows radiation of energy from the system. Methods for reducing the number of generalized coordinates while still representing the nonlinear behavior of a dam system may be explored (4-4). The use of a substructure formulation also may provide an efficient approach for nonlinear earthquake analysis of dams. Reference 4-5 presents a numerical method in which a structure is divided into linear and nonlinear substructures. The method was used to analyze a concrete arch dam, including opening of vertical joints, which were modeled as local nonlinearities using gap elements. Other dynamic analysis procedures for concrete structures are described in reference 4-6. Finally, evaluation of the dynamic stability of portions of a dam after extensive cracking requires numerical procedures that properly represent the impact and sliding of the sections and that conserve momentum.

An important decision in the analysis of the earthquake response of concrete dams is whether to use a two-dimensional or a three-dimensional model of the dam system. The complicated geometry of arch dams and associated valleys or canyons necessitates use of a three-dimensional model to represent their complex resistance mechanisms. Two-dimensional models are often employed for analysis of gravity dam monoliths with the restrictions discussed in Chapter 3, and nonlinear analysis of two-dimensional models is considerably less complicated than that for three-dimensional models.

The complete nonlinear response analysis of concrete dams introduces considerations not normally required in a linear analysis. As mathematical models become more representative of true nonlinear material behavior, the numerical results may be sensitive to additional parameters of the model. Further research should be able to evolve mathematical models that can be expressed in terms of measured material properties with confidence, but the cracking of concrete is likely to be very sensitive to its failure-strain limit as well as to the static state of stress and strain that exists at the time the earthquake occurs. This initial state is primarily due to the gravity load and hydrostatic pressure, but it is also affected by shrinkage and temperature strains that accumulate during the incremental construction process and after completion up to the time of the earthquake. Diurnal and seasonal temperature variations control the temperature gradient through the dam, and this may have a major effect on the initial stress and strain distribution, particularly for arch dams, as does creep of the concrete over the service life of the dam. The importance of thermal and shrinkage strains (and possibly creep) is evidenced by the cracks that developed in several monoliths in the Dworshak and Richard B. Russell dams under static loading conditions (4-7). An aerial photograph of the Richard B. Russell Dam is shown in Figure 4-1. The realistic analytical evaluation of earthquake response based on nonlinear behavior must recognize the distribution of stress and strain prior to an earthquake. This implies that for a given earthquake several analyses of a dam may be necessary, corresponding to the initial conditions at different times during the expected service life.

The challenge of nonlinear dynamic analysis of concrete dams lies in developing mathematical models that represent the true behavior of concrete and joints and in incorporating these models in efficient numerical procedures. Although researchers and practitioners have investigated limited aspects of the nonlinear behavior of concrete dams, a great deal of innovative analytical research must yet be done to develop practical nonlinear response analysis procedures; in addition, the results of such analyses must be verified by careful experimentation before the procedures can be fully accepted for earthquake safety evaluation of concrete dams.

CRITERIA FOR SAFETY EVALUATION

Although the criteria for evaluating the seismic performance of concrete dams are discussed in detail in Chapter 6, it is pertinent to note here that performance criteria based on nonlinear response evaluations are especially important. Traditionally, a no-tension stress criterion has been used in the design of concrete dams. However, microcracking is always present in concrete, and the acceptance of moderate tensile cracking that does not impair the function of a dam is a realistic point of view for earthquake loads. The complete nonlinear earthquake analysis of new and existing dams is not likely to be undertaken in the near future, and linear analysis will remain the normal practice for some time to come. The U.S. Army Corps of Engineers (4-8) has proposed an evaluation procedure that uses several linear dynamic analyses to estimate the extent of cracking by varying the level of acceptable tensile stresses and modifying the viscous damping ratio to represent energy losses associated with cracking. If cracking is clearly indicated by large tensile stresses, a separate stability analysis must be performed based on the estimated elevation of the cracks. An evaluation (4-9) of the proposed criteria shows that, although they are conservative in predicting the presence of tensile cracking, incorrect elevations of the potential cracks are indicated. Further work is definitely required to improve the criteria for acceptable tensile cracking.

Figure 4-1 Gravity section of Richard B. Russell Dam, Georgia, built by the U.S. Army Corps of Engineers (4-7).

SUMMARY OF RECENT RESEARCH

To understand the observed damage to Hsinfengkiang, Koyna, and Pacoima dams, experimental and analytical studies have been undertaken to determine the importance of nonlinear behavior in the earthquake response of concrete dams. Much of the work in concrete dams is related to or based on research in other applications of plain and reinforced concrete, for which excellent descriptions are available in references 4-10 and 4-11.

Experimental Research

Experimental investigations of the nonlinear behavior and failure mechanisms for concrete dams are discussed in Chapter 5 of this report, but the testing of the materials used in such dams is discussed briefly in the following paragraphs.

Most mathematical models for tensile cracking are based on data from direct uniaxial, split-cylinder, and flexural tension tests. The measurement of tensile behavior is affected by the type of test and by the size, configuration, and curing conditions of the specimen; this fact clouds the selection of parameters for material models (4-12). Most tests use plain concrete specimens of small aggregate that may not properly represent the effect of the large aggregate used in typical mass concrete. Tests of core samples properly taken from actual dams are more representative of the mix and in situ properties of the concrete. Because of the sensitivity of tensile tests to the loading and measurement system, optical interferometry techniques have been used to provide a continuous measurement of deformation in test specimens (4-13). The results show the nonuniform distribution of strain longitudinally and transversely in the specimens and strain discontinuities in the microcracked and fractured regions. Analyses of the test results indicate that the energy dissipated in the fracture process is due to separation of the two sides of the crack, while the extension of the crack front produces relatively little energy dissipation.

There have been few experimental investigations of the dynamic tensile behavior of concrete. Many mathematical models are based on early uniaxial tests (4-14) that showed increased tensile strength and stiffness with increasing monotonic strain rate. Recent biaxial tests of hollow concrete cylinders have been performed (4-15) using an impulsive loading system. The biaxial tensile strength in tension-compression loading also increased with decreasing rise time of the impulsive load, and the strain at failure was essentially independent of the rise time, which confirms the conclusions from earlier studies. These results suggest that a cracking criterion for concrete should be based on a maximum tensile strain instead of tensile strength. The dependence of the behavior on the loading rate, which is characteristic of a viscoelastic or viscoplastic material, should be accounted for in the mathematical model. These trends are also indicated in a review of uniaxial tension tests of concrete specimens (4-14), where it is recommended that the tensile strength be increased by 50 percent to include the effect of strain rates typical in the earthquake response of dams.

Models for Concrete Cracking

Early work in evaluating the static behavior of reinforced concrete members, including tensile cracking, used the finite element method with predefined cracks modeled by separations between elements (4-16). The modeling of tensile cracking by discrete gaps in the mesh was extended to allow automatic locations of cracks (4-17). When the average stress between two elements exceeded the tensile strength of the concrete, the common nodal points were separated, altering the finite element mesh. This discrete-crack approach involves a number of computational difficulties because the finite element mesh is redefined at each loading stage, and it is difficult to determine in which direction a crack will propagate during the next load increment. Moreover, in this approach the crack propagation is dependent on the size, shape, orientation, and order of the elements used in the mesh.

To overcome problems with the discrete crack approach, the tensile cracks can be considered "smeared" over an element (4-18). In the smeared-crack approach the discontinuous displacement field caused by cracking is averaged over the element and represented by the continuous displacement functions used to derive the element. A crack is assumed to form in any element in which the principal stress reaches the tensile strength of the concrete in the direction perpendicular to this stress; then the isotropic model of the concrete is modified to an orthotropic one with zero stiffness in the tensile direction but with possible shear transfer across the crack. Such transfer of shear stresses across cracks is important in mass concrete because of the large aggregate size and the probability of aggregate interlocking, but the smeared crack approach can represent shear transfer only approximately, because there is no direct information on the width and distribution of the smeared cracks. The tensile strength used to determine crack initiation can be modified to account for a multiaxial stress state (4-19), and the smeared-crack model behavior can be easily incorporated into nonlinear finite element analysis procedures. This requires only modification of the tangent stiffness matrix for the current state of cracking in an element and release of stress perpendicular to newly formed cracks.

The smeared-crack approach has been criticized because the numerical results are not objective with respect to the finite element mesh (4-20). As the element size decreases, the zone of fracture decreases, and the force required to propagate the crack can decrease to a negligible value. To remedy this lack of objectivity, the theory of fracture mechanics has been used to modify the smeared-crack approach in the so-called blunt-crack approach (4-20, 4-21, 4-22). This approach recognizes that tensile cracking is a fracture process in which the concrete at the crack front exhibits strain softening (decreasing stress with increasing strain) as microcracks coalesce to extend the crack. The crack front has a characteristic width, usually related to the aggregate size in the concrete mix. The criterion for crack initiation uses the fracture energy for the material (energy released in formation of a crack with unit area), and objectivity is achieved by selecting the strain-softening modulus, and possibly the tensile strength, to give the fracture energy of the material. An important advantage of the blunt-crack model is that it includes the effect of aggregate size, which is important when attempting to compute the response of large mass concrete dams using material properties obtained from testing small specimens. In a finite element analysis the crack band-width can be assumed to be the element size (within certain limits), and the strain-softening modulus then depends on the fracture energy and tensile strength of the concrete (4-20). The blunt-crack approach has been used in a similar form in reference 4-23, and it is related to the line (or fictitious) crack theory (4-24). One problem with the smeared-crack approach is the difficulty in representing impact and sliding of sections in the dam after extensive crack propagation, because the discontinuous displacements across the crack are not well defined; in some cases a discrete-crack has been combined with a smeared-crack model to represent such behavior.

Analytical Research

Numerical computation of the nonlinear earthquake response of concrete dams has received more attention than physical testing of models and materials. An early investigation of the nonlinear response of gravity dam monoliths used a biaxial failure model for concrete (4-25). The nonlinear compressive and tensile behavior of concrete was recognized by modifying an equivalent uniaxial stress-strain relationship (tension and compression) in accordance with the current state of stress. A smeared-crack approach was used to represent tensile cracking based on a strength criterion for crack initiation. Analysis of Koyna Dam, neglecting water and foundation rock interaction effects, showed that tensile cracks formed near the top of the dam close to the change in downstream slope, but the cracks did not propagate through the cross section. Including strain rate effects in the concrete model stiffened the dam, increasing the participation of the higher-vibration modes and producing more extensive cracking. An interesting finding was that the amount of tensile cracking was very sensitive to variations in the assumed concrete tensile strength; variations such as those observed in a set of typical tensile cracking experiments produced dramatically different amounts of cracking. The study also showed that tensile cracking predicted by this material model did not dissipate a significant amount of energy, suggesting that nonlinear behavior may not reduce the earthquake forces developed in the dam. However, this conclusion may not be correct, because the fracture of concrete may not have been properly accounted for in an objective manner.

Another series of studies (4-26, 4-27) involved analysis of a finite element model of a complete gravity dam-water-foundation rock system. The concrete was modeled as a rate-and history-dependent material using an elastic-viscoplastic relationship with a Mohr-Coulomb failure surface for compression and tension. The material model used viscoplastic work to measure accumulated damage in the dam; the anisotropy due to tensile cracking was not considered. Analysis of Koyna Dam showed a substantial residual displacement with large energy dissipation during the strong-motion part of the input record. It appears, however, that the energy dissipation is from viscoplastic work as the concrete softens in tension, which would not occur if cracking were allowed; thus, the significance of these results may be questioned.

Two recent studies illustrate the application of smeared-crack models for gravity dams. In the first application the crack-band model, described previously, was incorporated into a nonlinear dynamic analysis procedure for dam-water systems (4-28). The analysis of a 400-ft-high gravity dam monolith with a full reservoir due to a horizontal ground motion with a maximum acceleration of 0.36 g showed extensive cracking of the concrete, as shown in Figure 4-2. The cracking is located near the stress concentration caused by the change in geometry of the downstream face and the dam-foundation interface. The distributed crack zone indicated in Figure 4-2 is in conflict with the experimental results, summarized in Chapter 5, which show crack propagation in a narrow zone.

In another analytical study (4-29, 4-30) the crack-band model was used with several modifications: (1) a number of features designed to eliminate crack spreading, including a special formulation of the finite element to eliminate spurious stiffness, and (2) user control over the elements that are susceptible to cracking during the response analysis. The latter modification allows control over the direction of the crack, but it also indicates the lack of theoretical knowledge of crack propagation in concrete dams. Figure 4-3 shows the crack that forms in the same 400-ft-high gravity dam monolith with full reservoir when subjected to horizontal and vertical ground motions with maximum acceleration of 0.50 g and 0.30 g, respectively. The narrow crack zone extends from the stress concentration at the downstream face, turning down near and parallel to the upstream face. In addition, a crack formed near the base of the dam. Although the analysis showed that the dam remained stable, many numerical difficulties were encountered.

As noted above, the alternate approach for modeling tensile cracking is to represent the formation, propagation, and closure of discrete cracks in the concrete. The discrete-crack approach has been used in the finite element earthquake analysis of gravity dam monoliths, neglecting interaction with the water and foundation rock (4-31). Each crack was monitored, and the topology of the element mesh was redefined to represent the current state of cracking. Analysis of Koyna Dam showed that the top part of the dam would become unstable as a tensile crack propagated across the cross section due to an artificial earthquake with a peak acceleration of 0.50 g. As in the earlier research mentioned above, the extent of cracking was very sensitive to the assumed value for concrete tensile strength. The response was also

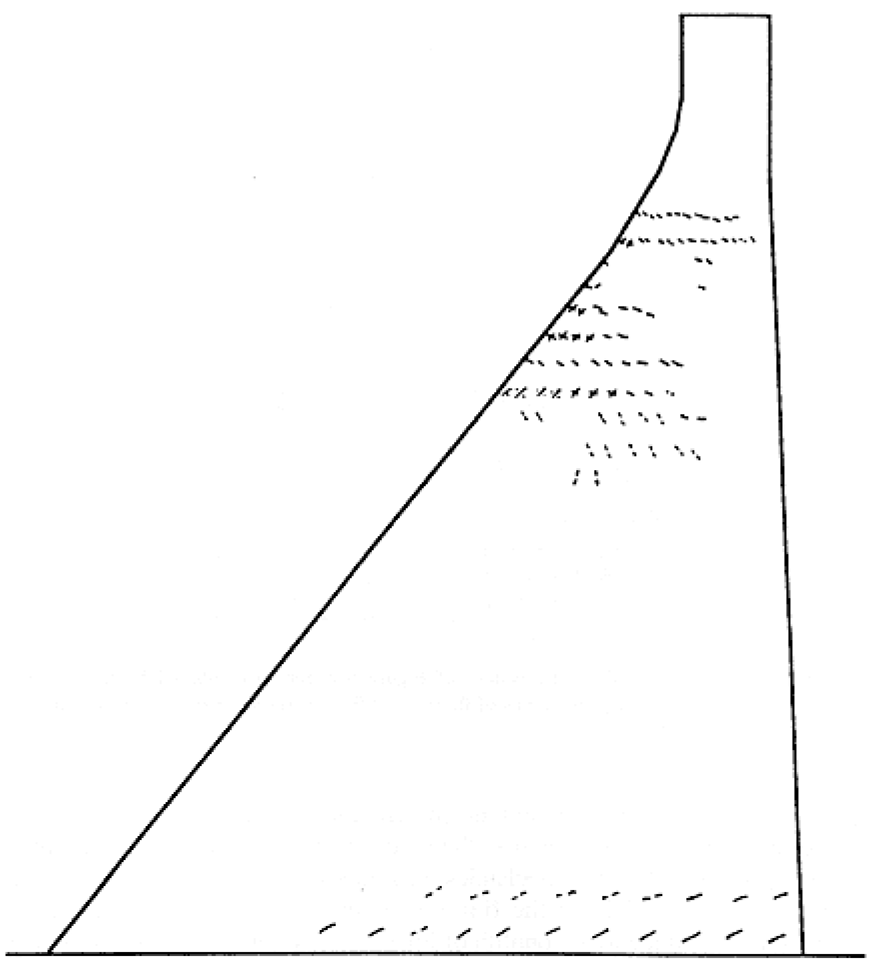


Figure 4-2 Cracking calculated in 400-ft-high monolith of Pine Flat Dam with full reservoir due to 0.36-g peak horizontal ground motion; solid lines show cracks open at one instant of time, dashed lines show cracks that opened previously and then closed (4-28).

dependent on the finite element mesh size and orientation, although the effect of aggregate interlock across the cracks was relatively small. Another approach (4-6) used fracture mechanics techniques to overcome the sensitivity of the tensile stress field to the finite element discretization near a crack. Applying the technique to a monolith of Pine Flat Dam, including compressible water in the reservoir, showed that a tensile crack propagated from the upstream face at the change in slope, but stopped short of the downstream face as the compressive stresses arrested the crack growth. The upper portion of the dam appeared to remain stable; however, the cracking seemed extensive for the relatively small peak ground acceleration of 0.1 g. Other applications of fracture mechanics concepts to concrete dams are described in references 4-32 and 4-33.

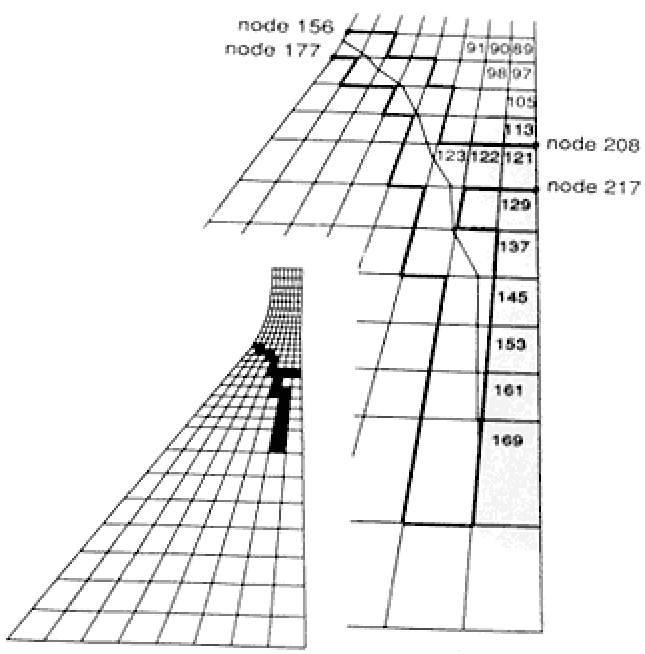


Figure 4-3 Cracking calculated in system of Figure 4-2 due to combined horizontal and vertical ground motions with peak values of 0.50 g and 0.30 g, respectively; cracked elements are shown in black (4-30).

There have been two particularly noteworthy earthquake studies of arch dams that included the nonlinear behavior of construction joints. The first used an approximate representation of the discrete cracks formed by the joints (4-34). The stiffness of the elements in the vicinity of the joints was modified to represent the current state of tensile stresses near the joints. This approach has also been used to represent opening of a horizontal joint in a gravity dam (4-35). Another study (4-36) used a simple discrete-crack model of the joint that required only one finite element through the thickness of the arch. The analytical results for a single arch rib showed the expected opening-closing behavior and large compressive stresses in the portion of the joint in contact.

Later work (4-37) extended the crack model to represent vertical construction joint opening and horizontal joint opening at lift surfaces in arch dams. In one of the most complete nonlinear analyses to date, the earthquake response of Pacoima Dam (Figure 2-4) was computed, including dam-foundation interaction (with massless foundation rock) and dam-incompressible water interaction (4-38). An example of the calculated response of Pacoima is shown in Figure 4-4. The earthquake ground motion for this study was the Pacoima Dam record scaled to a maximum horizontal acceleration of 0.50 g and vertical ground acceleration of 0.30 g. Figure 4-4 shows the joint stresses and displaced shape of the crest arch and crown cantilever at two instants of time during the response. The response results, as exemplified by this figure, showed that the vertical contraction joints in the upper part of the dam and the joint assumed at the dam-foundation interface opened under moderate ground motion. The loss of arch stiffness due to vertical joint opening also caused horizontal joint opening in the upper parts of the cantilevers. During severe ground motion some cantilever blocks lifted off their supports, and large compressive cantilever stresses developed, possibly invalidating the assumption of no joint slip and linear behavior of the concrete in compression.

Nonlinear analysis of concrete dams allows for the inclusion of other phenomena that may affect the earthquake response. One such possible effect is cavitation of the impounded water, in which gaseous regions form if the absolute pressure in the water becomes less than the vapor pressure. The possibility of cavitation has been shown analytically and observed in model tests (4-3). The formation and collapse of gaseous regions in the water would alter the hydrodynamic pressure acting on the upstream face of a dam and hence change the dynamic response. One analytical study of cavitation (4-39) for a gravity dam monolith, assuming incompressible water, showed that impact of the water resulting from collapse of the cavitation bubble can increase tensile stresses in the top part of the dam by 20 to 40 percent. In contrast, an evaluation of dam-water interaction including compressible water concluded that cavitation does not significantly affect the maximum stresses due to earthquake ground motion (4-40). Recent work including compressibility also confirms the latter conclusion, in which it was shown that cavitation has a very small effect on peak displacements and stresses in a dam (4-41). However, cavitation can double the peak acceleration at the dam crest, which may affect appurtenances and facilities at the crest. To date, there has been no research into cavitation effects for arch dams, where dam-water interaction effects may be more important than

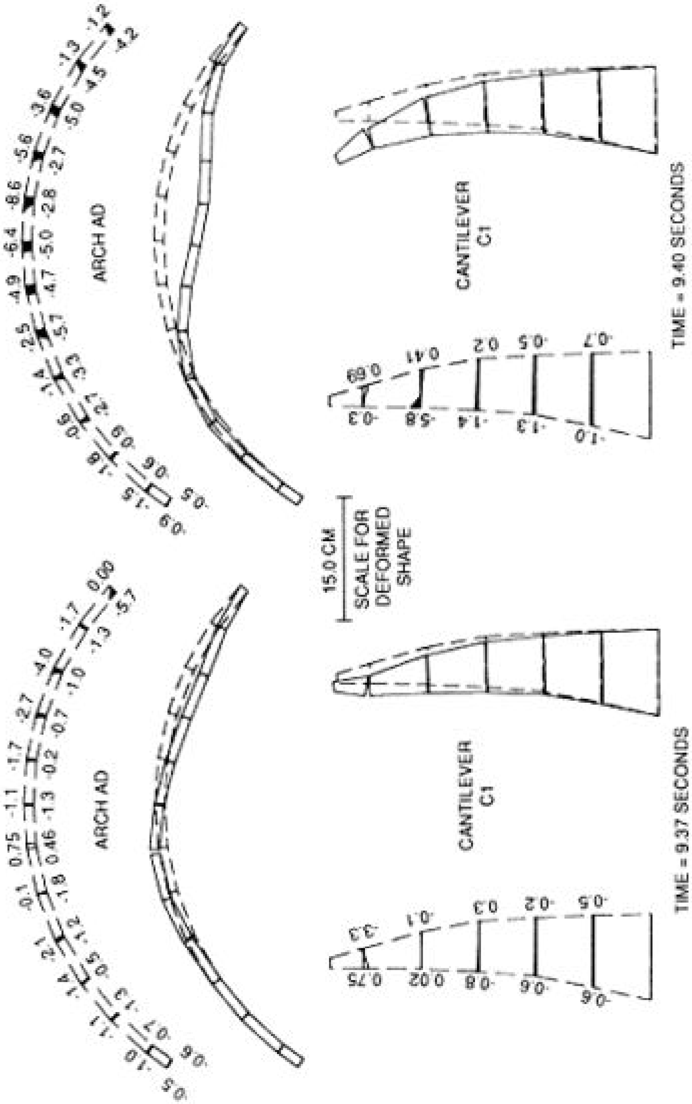


Figure 4-4 Calculated nonlinear response of Pacoima Dam to three-component Pacoima earthquake record at two instants of time. Shading at joints depicts compressive stresses; unshaded joints indicate joint openings (4-38).

in gravity dams; however, it has been noted that cavitation may have a beneficial effect by limiting the upstream hydrodynamic force acting to open the vertical joints (4-39).

RESEARCH NEEDS

Recognizing the importance of evaluating the earthquake safety of concrete dams and the limited knowledge that exists concerning the nonlinear behavior of concrete dams, the following items should be addressed in a future comprehensive research effort:

1. Material Testing of Mass Concrete

Additional data on the behavior of mass concrete under dynamic loads is urgently needed and will require an extensive physical testing program. The testing should emphasize the tensile cracking of the mass concrete under multiaxial stress states that are representative of the in situ stresses, including strain rate effects. The test program should recognize the special properties of the mass concrete used in dam construction, such as its large aggregate size. Concrete samples should be representative of the actual curing conditions in dams. To accomplish this, the testing should include laboratory samples as well as cores from actual dams, and some specimens should be taken from roller-compacted concrete dams. To perform these tests it may be necessary to develop or adapt equipment to provide biaxial cyclic loads on large specimens.

2. Development of Materials Models for Concrete

Using the data obtained from testing of mass concrete specimens, it is important to develop realistic mathematical models for tensile cracking under dynamic loads. Identification of a set of parameters that are predictive of nonlinear dynamic behavior of concrete would be an important advance. The material model should allow multiaxial stress states, including criteria for tensile cracking and propagation of cracks, strain rate effects, and shear stress transfer by aggregate interlock. The models must make reasonable trade-offs between fitting the experimental data, satisfying conditions such as objectivity, and providing computational efficiency. Further study of the smeared-crack approach and use of fracture mechanics principles should be pursued. With advances in computational techniques and computer hardware, the discrete-crack approach for dynamic response may provide an alternative.

3. Modeling of Other Nonlinear Mechanisms

Additional research is required to develop and apply models of construction joint behavior in the earthquake analysis of concrete dams. Such models must represent the redistribution of forces as the joints open and close and allow for degradation of the joint resulting from a large number of loading cycles. Models for concrete and rock abutments that include potential shear failure modes are also necessary. Although not directly related to dynamic response, analytical procedures must be used for determining the state of stress and strain in a concrete dam at any point in time—due to gravity and hydrostatic loads, shrinkage, temperature variations, temperature gradients, and creep—for accurate assessment of nonlinear dynamic response of dams at the time of an earthquake.

4. Numerical Procedures for Computing Nonlinear Response

Once realistic mathematical models for concrete, joints, and foundation rock are available, they can be incorporated into several classes of numerical procedures for nonlinear dynamic response analysis of concrete dam systems, including interaction with the impounded water and flexible foundation rock and the effects of reservoir-bottom shock-wave absorption. Research should identify methods for reducing the computational effort, while still representing the important nonlinear behavior of the system, so that nonlinear analysis can be used for routine design and evaluation of dams.

Although the finite element method and time integration of the equations of motion appear to be the most useful techniques for nonlinear dynamic analysis of concrete dams, other discretization methods and procedures for solving nonlinear equations of motion should not be precluded. The use of algorithms that take advantage of the vector and parallel processors in supercomputers should be pursued.

5. Parametric and Detailed Response Studies

With the development of efficient numerical procedures, response analyses of typical concrete dams can be used to improve understanding of the nonlinear behavior during earthquake ground motion. In particular the studies should identify the significance of tensile cracking and joint opening with respect to either precipitating failure or limiting dynamic response. It is important to determine the sensitivity of the response to parameters describing the nonlinear models. Postcracking stability in gravity and arch dams and postearthquake joint integrity in arch dams (as well as in some gravity dams) must be evaluated. In addition, the influence of the following factors should be determined: (1) ground motion characteristics, particularly amplitude, frequency content, duration, and occurrence of velocity pulses, (2) water compressibility with respect to nonlinear response, and (3) modeling issues, such as degree of spatial discretization, extent of reservoir and foundation rock domains, and radiation conditions at the boundaries.

6. Dynamic Testing of Dam Models

In parallel with the analytical studies, further dynamic testing of dam models is essential for verifying nonlinear behavior and calibrating mathematical models and numerical results. If at all possible, testing of gravity dam and arch dam models including water and foundation rock should be undertaken. New, larger earthquake simulators such as that recently installed at the research laboratory of the Ministry for Water Conservancy and Hydroelectric Power in Beijing, China, may be required for realistic testing of concrete dam systems.

7. Identification of Design Criteria

Realistic design criteria for concrete gravity and arch dams should be developed using the results of analytical and experimental studies. Although nonlinear earthquake analysis may not become a standard practice in design offices, design criteria should recognize the tensile strength of concrete and postcracking stability of gravity and arch dams and identify acceptable limits of such behavior. Comparison of the predicted nonlinear response with the more easily calculated linear response would be useful to determine the limits of assessing nonlinear behavior based on the results of linear elastic analysis. Current design criteria should be evaluated in this regard.

8. Investigation of Earthquake-Resistant Design Measures

Nonlinear response analysis capabilities will allow investigation of innovative measures for increasing the earthquake safety of concrete dams. Based on the results of nonlinear analyses, it may be possible to alter the geometry of dams and foundation abutments to improve resistance to earthquakes. Different jointing schemes, including use of joint materials that dissipate energy, should be investigated.

It must be emphasized that these research needs are not only important in improving the seismic safety of concrete dams but would also represent significant advances in other applications of earthquake engineering. Similarly, research in the nonlinear dynamic behavior of other structural systems will have a positive effect on the study of concrete dams.

5

Experimental and Observational Evidence

EARTHQUAKE EXPERIENCE

Cases with Strongest Shaking

It was noted earlier that no concrete dam has ever failed as a result of an earthquake; however, as far as can be determined, neither has a large concrete dam with full reservoir been subjected to very strong ground shaking. The closest to such an event was the previously mentioned 1967 experience at the 103-m-high Koyna (gravity) Dam (Figure 3-1) with the reservoir nearly full (3-5, 3-6). Recorded ground motions at the dam from a nearby earthquake of magnitude 6.5, probably reservoir induced, peaked at 0.49 g in the stream direction and continued strongly for 4 sec. As stated in Chapter 3, significant horizontal cracking occurred through a number of nonoverflow monoliths at a level 36 m below the crest where the downstream face changed slope, but gravitational stability prevailed, even though the water level was 25 m above the crack (Figure 5-1). Spalling of the concrete in many of the contraction joints and damage to the joint seals provided some evidence that the monoliths vibrated individually rather than as a single block. A similar experience had occurred in 1962 at the 105-m-high Hsinfengkiang (buttress) Dam (5-1). Cracking at a level 16 m below the crest and 3 m below the water surface resulted from a magnitude 6.1 earthquake, again probably reservoir induced. Ground motions were not recorded, but with the epicenter in close proximity they probably reached significant intensity.

Perhaps the strongest shaking experienced by a concrete dam to date was that which acted on Lower Crystal Springs Dam, a curved gravity structure with the modest height of 42 m (shown in Figure 5-2), during the magnitude 8.3 San Francisco earthquake of 1906 (5-2, 5-3). The dam incurred no damage, even though it stood with its reservoir nearly full within 350 m of the fault trace at a point where the slip reached 2.4 m. However, the stability of this structure exceeds that of typical gravity dams due to its curved plan and a cross section that was designed thicker than normal in anticipation of future heightening, which was never completed. Another example of a concrete dam subjected to strong shaking is the 103-m-high Pacoima (arch) Dam, shown in Figure 2-4. During the 1971 magnitude 6.6 San Fernando earthquake (5-4, 5-5), an accelerograph located on the left abutment ridge 15 m above the dam crest recorded peak accelerations of 1.2 g in both horizontal components and 0.7 g vertical with a duration of strong shaking of 8 sec. Even though amplification of this motion occurred at the recorder location, the excitation to the dam's boundaries must have been severe. However, the only visible damage was a slight opening of the contraction joint on the left thrust block, a crack in this thrust block, and slumping of an area on the left abutment; the first two effects may have been caused by the latter. The good performance of the dam can be attributed partly to the low water level, 45 m below the crest at the time of the earthquake.

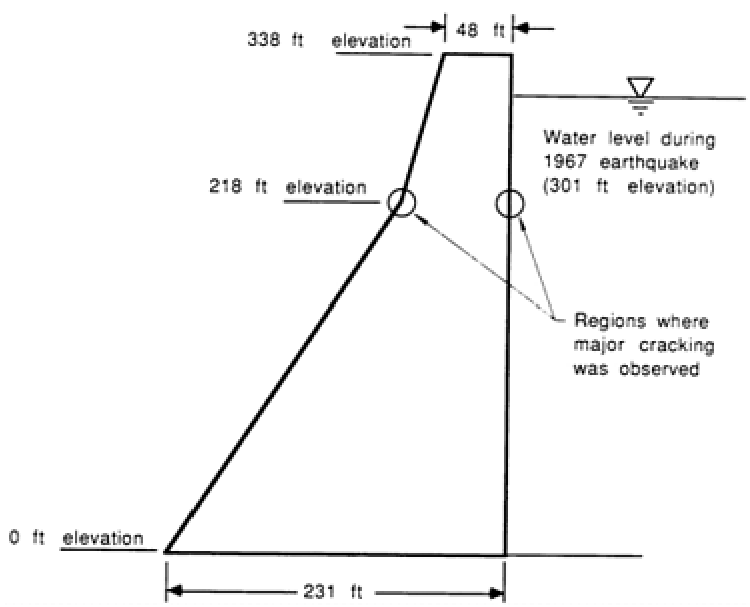


Figure 5-1 Cross section of Koyna Dam showing water level during 1967 earthquake and regions where principal face cracking was observed (3-6).

Moderate shaking, on the order of 0.3 g in a horizontal component as recorded on the abutment, failed to damage Ambiesta (arch) Dam (59 m high) during the magnitude 6.5 Friuli earthquake of 1976 (2-51, 5-6), and motion of similar intensity affected only some appurtenant structures at Rapel Dam (arch, 110 m high) during the strong (magnitude 7.8) 1985

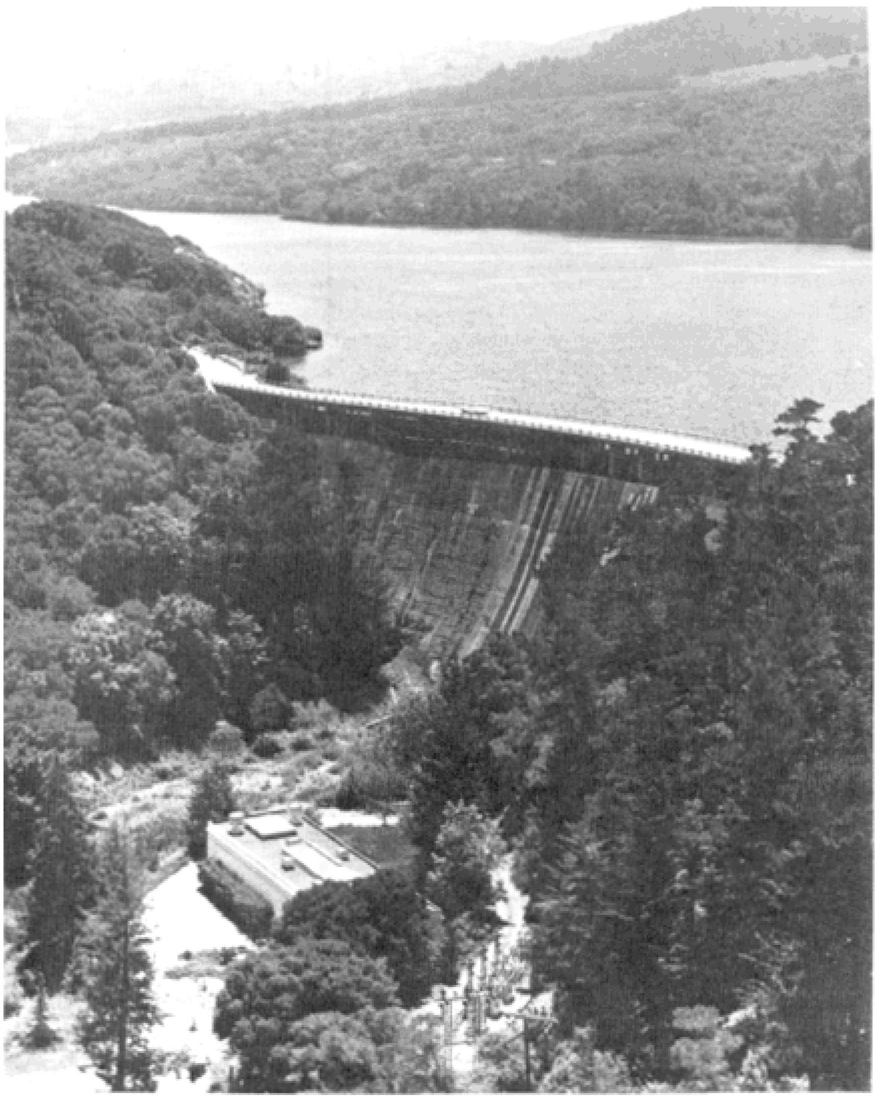


Figure 5-2 Lower Crystal Springs Dam, California; the San Andreas fault lies under the reservoir (5-3). Photo courtesy of San Francisco Public Utilities Commission.

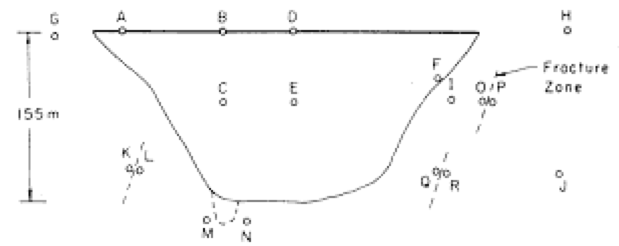
Chilean earthquake (5-7). At Rapel cracking occurred on the inner walls of the spillway intakes, and the power intakes, which had been designed as vertical cantilevers, suffered some minor damage. In another instance the magnitude 6.3 Santa Barbara earthquake in 1925 caused no damage to Gibraltar Dam (arch, 50 m high), even though the shaking was so strong that a workman on the dam reportedly had difficulty standing up (5-8). For these cases the reservoir condition was mentioned in the references only for Rapel Dam, and it was stated to be full.

Thus, although the experience outlined above (which represents the most significant earthquake events that have acted on concrete dams) is impressive, it falls short of providing complete confidence that large concrete dams with full reservoirs are safe against strong seismic shaking.

Cases with Recorded Responses

Actual recorded time histories of the response of concrete dams during earthquakes are scarce for events causing other than very small shaking. What is believed to be the largest recorded peak acceleration measured on a concrete dam was the 0.6 g value obtained on the crest of Hsinfengkiang Dam during a magnitude 4.5 shock in 1972 (5-9). Peak crest accelerations recorded at other dams have been much smaller and include 0.25 g at Nagawado (arch) Dam (2-47), shown in Figure 5-3, 0.20 g at Hoover (gravity-arch) Dam (5-10), 0.17 g at Techi (arch) Dam (2-46), 0.15 g at Yuda (gravity-arch) Dam (5-11) and at Big Dalton (multiple arch) Dam (2-45), and 0.12 g at Kurobe (arch) Dam (2-39). Among all these events, the maximum recorded foundation acceleration was 0.08 g at Hoover. Large amplification of motion from foundation to crest was detected often, indicating small damping. Such evidence exists at Ambiesta (2-43), Hsinfengkiang (5-9), Kurobe (2-39), Nagawado (2-47), Shintoyone (arch) Dam (5-12), Tagokura (gravity) Dam (5-12), Talvacchia (gravity-arch) Dam (5-13), Techi (2-46), Tonoyama (arch) Dam (5-14, 2-37), and Yuda (5-11). None of these data are from strong shaking, when the material component of the damping would be higher.

A number of measurements of motion at the dam-foundation interface during small to very small shaking have been obtained by instrument arrays located at Ambiesta (2-43), Kurobe (2-39), Nagawado (2-47), Pacoima (2-45), Tagokura (5-12), Talvacchia (5-13), Tonoyama (5-14, 2-37), and Techi (2-46); the maximum recorded interface motions were 0.04 g at Nagawado and Techi. In general, the motion varied significantly around the canyon, often showing considerable amplification in the upper abutments over that at the toe. Some of these data were for distant earthquakes and may not characterize the strong motions expected from nearby earthquakes. Nonuniformities can be present in the free-field motions and can also arise from the interaction between the dam and foundation; relative proportions of these two effects in the measured motions are unknown. Some data from Hsinfengkiang Dam, where peak accelerations at a free-field site were reduced by a third at the base of the dam, suggest that dam-foundation interaction effects can be significant (5-9). The subject of seismic input is one on which much more needs to be learned, as discussed in Chapter 2. However, in the United States only Lower Crystal Springs and Pacoima dams are well enough instrumented to record motions at both the abutments and the toe (2-69, 5-15), and it may be that the three accelerographs used on the dam-foundation interface at each of these sites are not sufficient to define the spatial distribution of earthquake motions.



Station	Peak accelerations (g)
A	0.035 (R)
B	0.247 (R)
C	0.049 (R)
D	0.196 (R)
E	0.074 (R)
F	0.036 (R)
G	0.021 (S), 0.027 (C), 0.013 (V)
H	0.027 (S), 0.031 (C)
I	0.037 (O)
J	0.021 (S), 0.019 (C)
K	0.029 (S), 0.021 (C)
L	0.020 (O)
M	--- 0.017 (C)
N	0.029 (S)
O	---
P	0.016 (S), 0.027 (C)
Q	0.025 (S), 0.022 (C)
R	0.020 (S), 0.023 (C)

Orientation
R = radial
S = stream
C = cross-stream
V = vertical
O = other horizontal

Figure 5-3 Nagawado Dam face, showing locations of accelerograph stations; peak accelerations during the 1984 magnitude 6.8 Naganokan Seibu earthquake are tabulated (2-47).

Only for Tagokura Dam have dynamic pressures in the reservoir been measured during seismic shaking (2-36, 2-38, 5-16). The events for which pressures were recorded included two small earthquakes and the larger Niigata earthquake; however, time histories from the latter were unintelligible. From records of the two smaller earthquakes, investigators noted that the dynamic pressure and crest acceleration of the dam responded nearly in phase and concluded that, since the predominant period of response appeared to be close to what was thought to be the resonant frequency of the water, compressibility effects of the water were absent, because they would be expected to cause a phase difference. However, it is possible that the response frequency was somewhat below the resonant frequency of the water, and without an analytical investigation to indicate the expected phase difference, the conclusion seems premature. In the United States, only the 219-m-high Dworshak (gravity) Dam is equipped with dynamic pressure transducers, five on the center monolith at various depths (5-17), but no records have been obtained.

Analyses to Reproduce Earthquake Response

A number of analyses have been performed on concrete dams that have received earthquake shaking with the aim of reproducing some features of their response. Analyses of Lower Crystal Springs Dam (5-2, 5-3) for the 1906 San Francisco earthquake and of Pacoima Dam (5-4) and Big Tujunga (arch) Dam (5-18, 5-19) for the 1971 San Fernando earthquake were carried out under a State of California program on the seismic safety of dams. Finite element models of the dam and foundation region (the latter assumed massless and truncated at a far boundary) with the reservoir water (assumed incompressible) represented by lumped added masses were subjected to uniform excitations at the foundation boundary. Damping at 5 percent of critical was included in the system modes. For Lower Crystal Springs Dam (Figure 5-2) an artificial accelerogram with a 0.60-g peak acceleration represented the 1906 ground motion and was applied in the stream direction. Computed tensile stresses exceeded the tensile strength of concrete only over a small region at the upper part of one abutment, indicating that some separation would have occurred, since the foundation rock was highly fractured. Thus, the computations are not at odds with the lack of observed damage from the 1906 earthquake.

Ground accelerations employed in the Pacoima analysis were those measured on the ridge reduced by a third in an approximate attempt to represent the motions that actually excited the dam. Computed arch tensile stresses indicated that some openings of the contraction joints should have taken place, but no evidence of opening and closing was noted in the postearthquake inspection. However, such openings may have left little evidence, or perhaps the assumption of uniform base input motion exaggerated the predicted response. A recent nonlinear analysis (4-38) of Pacoima Dam using the above ground motion produced significant contraction joint openings, but the same comments apply. At Big Tujunga, where the water was at an intermediate level during the earthquake, input accelerations were constructed from a seismoscope trace obtained on one abutment, resulting in peak input accelerations of 0.2

g in both horizontal components. The computations led to a comparison between a seismoscope trace recorded on the crest at the center of the dam and a computed trace at the same location; reasonable correlation was seen.

The cracking that occurred at Koyna Dam and the availability of recorded ground motions at that site have stimulated a number of analytical investigations. Two-dimensional representations have been employed on the assumption that the monoliths vibrated independently. Linearly elastic finite element analysis (3-6) in which dam-water interaction effects were approximately included showed that net tensile stresses near the point of slope change on the downstream face and at a similar elevation on the upstream face significantly exceeded the tensile strength of concrete when the recorded ground motion was used as input. Several nonlinear analyses of Koyna Dam have attempted to include cracking, as mentioned in Chapter 4. In one of these analyses (5-20) a horizontal preexisting crack was assumed at the elevation of slope change, and the portion of the dam above the crack was treated as a rigid body with only a single rotational degree of freedom. Water was included statically as a pressure on the upstream face of the dam and in the crack and also was included dynamically through an added mass. Although the dam was shown to be kinematically stable under the ground motions recorded during the earthquake, the analysis appeared to incorrectly reverse the angular velocity upon impact. Another study (4-26) employed the finite element method and smeared cracking, with stress release once the tensile stress reached a critical value that was a function of strain rate. The effect of water was neglected. With the measured ground motion as input, cracks on both faces occurred near the elevation of slope change, but the crack penetration was not very deep. A recent finite element study (4-31) attempted to model the formation and propagation of discrete cracks through mesh adaptation using a maximum tensile stress criterion. The results appeared to exhibit unstable branching of cracks, and different crack patterns were produced by mesh refinement. In an analysis of Koyna Dam using a ground motion weaker than the recorded one, cracks issued from the point of slope change and quickly reached the upstream face. Again, the effects of water, both as added mass and in crack penetration, were neglected. Based on these studies, it is evident that considerable improvement in nonlinear mathematical models of concrete dams is still needed and that the data from Koyna Dam are the most complete set from which to evaluate the performance of these models.

Analyses of Hsinfengkiang Dam were stimulated by the 1962 earthquake experience in which cracking occurred and by a valuable set of records obtained on the dam during the magnitude 4.5 shock in 1972 after the dam had been greatly strengthened and stiffened. Nonlinear analyses (5-21) have been performed to examine the rocking and sliding stability of the top portion of the dam treated as a rigid body and separated from the flexible structure below by a preexisting crack. The water at a level 8 m above the crack was included statically as an applied force and dynamically as an added mass. Separate analyses were performed for rocking and sliding, but the two mechanisms were not allowed to occur simultaneously. With these assumptions the top portion of the dam was shown to remain kinematically stable for reasonable ranges of friction coefficients, rotational impact assumptions, and ground motions. For the magnitude 4.5 shock, using the recorded base motion as input, linearly elastic finite element analysis (5-9) of an individual buttress idealized as two-dimensional successfully reproduced the time history recorded on the crest of the dam. Although the reservoir was partially full during the earthquake, it was omitted from the analysis. The computed vibration profile agreed with the measured one; both showed very large amplification of motion in the upper part of the dam, which probably was the cause of the cracking that occurred in 1962.

Other reported analyses include two performed for the 90-m-high Yuda Dam (5-11) for magnitude 5.9 and 7.4 earthquakes in 1976 and 1978, respectively, and one analysis of Ambiesta Dam (2-43) for an aftershock of the 1976 Friuli earthquakes. The linearly elastic analyses of Yuda Dam employed a two-dimensional finite element model of a 67-m-high monolith with reservoir water included; this monolith was chosen because its resonant frequency matched the observed predominant frequency of response of the three-dimensional dam. With modal damping taken to be 3 percent of critical and using accelerations recorded on one abutment during the two earthquakes as input, the computed response time histories of the crest showed similarities to those recorded on the crest of the 90-m-high monolith. However, because controlling both the predominant frequency of vibration by selection which monolith to analyze and the response amplitude level by selection of the value of damping is enough to guarantee a reasonable match, the conclusions to be drawn from this study are limited. Water compressibility was both included and neglected in the comparisons and was found to be unimportant. However, because only a short monolith was analyzed, the effect of water compressibility was limited by the reduced water depth. The finite element model of Ambiesta was subjected to nonuniform excitations as recorded by seismographs at five locations on the foundation interface. Although reservoir water was present during the aftershock, none was included during the analysis. Nevertheless, the computed profile of maximum response of the dam crest agreed with that measured on the crest, but few details were provided. Presumably, a uniform foundation input produced poorer results.

A recent investigation of the response of Nagawado Dam (arch, 155 m high) to the magnitude 6.8 Naganoken Seibu earthquake of 1984 in Japan made use of a number of accelerograms obtained on the dam and foundation rock and employed a sophisticated finite element model that included nonuniform ground motion, dam-foundation interaction, and water compressibility (2-48). The foundation rock was represented by springs and dashpots defined from two-dimensional solutions for a strip footing on an elastic half space. Modes of the dam-foundation system (dashpots omitted) were employed as generalized coordinates and were assigned damping values equal to 2 percent of critical. To represent the water, the analysis used a velocity potential formulation with compressibility included along with an approximate transmitting boundary to represent an infinite reservoir and a 33 percent pressure wave absorption condition on the reservoir floor and sides. The elastic moduli of the dam and foundation were adjusted to match measured resonant frequencies from previous forced-vibration field tests. The solution procedure defined seismic input in terms of free-field motions at the foundation interface; these motions, which were applied to both dam and reservoir, were taken as those recorded on the foundation rock a short distance from the dam at both abutments at crest level and at the canyon bottom. Interpolation was used to obtain the motions at intermediate levels. The analysis sought to reproduce the time histories recorded on the dam crest, but satisfactory results were not achieved. The major discrepancy was underestimation of the strong antisymmetric response of the dam as evidenced by a sizable record from the left quarter point on the crest. Other results using a finite element model of the foundation failed to enhance the relative amount of antisymmetric response. A number of potential sources of error existed. Those associated with definition of the free-field input included siting the accelerographs off the dam-foundation interface and the absence of records at intermediate levels and along the reservoir (the latter for use as input to the reservoir water). Siting the accelerographs off the interface was intended to reduce the contamination from the dam response, but it introduced other errors.

In summary, analyses of concrete dams that were intended to reproduce features of their observed earthquake response have in some cases been successful, although many of the studies reported used crude mathematical models such as a lumped-added-mass representation of incompressible water or spatially uniform excitations. It is possible that adjustments in the analyses were made until correlation was obtained. The study of Nagawado Dam reveals the complexity of the seismic response of a concrete dam arising from nonuniform excitations and suggests that accurately capturing free-field motions at the foundation interface of the dam and reservoir water is difficult. Certainly, more earthquake data and study are needed regarding seismic input, as well as for dam-water interaction and nonlinear response. Important data sets obtained recently at Techi Dam (2-46) and Pacoima Dam (2-45) provide other opportunities for postearthquake analysis.

FIELD VIBRATION TESTS: FORCED AND AMBIENT

The literature contains a large amount of data from forced-vibration tests and ambient measurements from actual dams, some of which should be regarded with caution. For example, modal damping values ranging from 1 to over 12 percent of critical have been reported from forced-vibration tests. Much of the high damping data (e.g., Baitings [gravity] [5-22], Upper Glendevon [gravity] [5-23], Naramata [arch] [5-24], and Tsukabaru [gravity] [5-25]) is probably not accurate due to excessive modal interference in the dam response or, perhaps for some of the older data, inadequate frequency control on the shaking machines. When these problems are absent, low damping values in the range of 1 to 3 or 4 percent of critical usually result. Examples of lightly damped gravity or gravity arch dams include Fengshuba (5-26), Fiastra (2-51, 5-27), Pine Flat (5-28, 5-29), Place-Moulin (2-51), Talvacchia (5-13), and Xiang Hong Dian (5-30, 5-31), while lightly damped arch dam examples are Ambiesta (2-43), Emosson (5-32), Hengshan (5-33), Kolnbrein (5-34), Kops (5-35), Kurobe (2-39), Liuxihe (5-33), Lumiei (2-51, 5-27), Monticello (5-36), Morrow Point (5-37, 5-38, 5-39), Quan Shui (5-40), Sakamoto (5-41), Sazanamigawa (5-42), Schlegeis (5-35), and Yugoslavia No. 1 and No. 2 (5-43). No clear trends between the amount of damping and resonance number have appeared. It is very interesting that with the large potential for radiation of energy that exists with a concrete dam, being fully embedded in foundation rock that often has material properties similar to concrete, and with the impounded water completely covering one face, so little damping is present. Of course, during strong shaking the material component of the damping would be expected to increase.

Contraction joints that can accommodate differential movements in gravity or buttress dams, if present, would complicate their behavior during forced-vibration tests, and data from Pine Flat Dam (5-28, 5-29) and Wimbleball (buttress) Dam (5-22, 5-44) indicate that some relative motion can occur across a joint. However, some of the complex behavior observed for such dams, as at Wimbleball (5-22, 5-44) and Upper Glendevon (5-23), which has been attributed to movements across joints, may actually be due to excessive modal interference arising from shaking a long structure with a single shaker. Some odd behavior observed at Pine Flat Dam (5-28, 5-29) between winter and summer forced-vibration tests at different water levels may have been due to changes in joint contact resulting from variations in temperature or water load. It should be mentioned that if only frictional forces exist across the joints (i.e., unkeyed), the friction may be overcome during the response to strong shaking, resulting in primarily individual vibrations of the monoliths or buttresses. In this case forced-vibration or ambient data would not be entirely relevant. Contraction joints may also affect the forced-vibration behavior of arch dams, as at Talvacchia Dam (5-13), where differences in resonant frequencies were observed between different tests at the same water level. In this case temperature was again suspected of playing a role.

The presence of reservoir water lowers the resonant frequencies of the system because of the added mass effect. Data on the amount of reduction comes from forced-vibration field tests or ambient measurements carried out at significantly different water depths (Alpe Gera [gravity] [5-45], Ambiesta [2-43], Big Tujunga [5-18], Fiastra [5-27], Kamishiiba [arch] [5-14, 5-42], Kolnbrein [5-34], Kops [5-35], Morrow Point [5-37, 5-38, 5-39], Pine Flat [5-28, 5-29], Talvacchia [5-13], Techi [5-46, 5-47], Tonoyama [2-37, 5-14], Sazanamigawa [5-42], Stramentizzo [arch] [5-48], Val d'Auna [gravity-arch] [5-48], Wimbleball [5-22, 5-44], Yahagi [arch] [5-49], and Yugoslavia No. 2 [5-43]). The amount of reduction in resonant frequencies depended on the amount of increase in water depth, the initial depth, the resonance number, and the type of dam. The largest reductions occurred in the lower resonances and for thinner dams and reached 20 percent in either the fundamental symmetric or antisymmetric resonance at Ambiesta, Morrow Point, and Sazanamigawa dams for increases in the water level by 22, 32, and 33 percent of the dam heights, respectively, that filled the reservoirs. Modal damping as a percentage of critical tended to remain the same or increase slightly with an increase in water depth, possibly indicating some extra energy loss due to radiation.

A number of analytical studies have been carried out in conjunction with forced-vibration field tests or ambient measurements. The most thorough studies were performed on Xiang Hong Dian (5-30, 5-31), Quan Shui (5-40), Techi (5-46), Monticello (3-26), and Morrow Point (5-39) dams, all by U.S. investigators; the layout of Xiang Hong Dian Dam is shown in Figure 5-4. Finite element models of the dam-foundation-water system were employed in all of these analyses, assuming the foundation to be massless; water compressibility was included only for Morrow Point Dam and in a few results for Monticello Dam. Parameters of the finite element models adjusted for best fit to the data included the elastic moduli of the dam and foundation, and the damping values.

In the studies of Xiang Hong Dian (gravity-arch, 88 m high) and Quan Shui (arch, 80 m high) dams, comparisons of the computed results with those measured during forced-vibration field tests were made for resonant frequencies; for resonating shapes and amplitudes of the dam, including those at the foundation interface; for frequency-response curves of the dam crest near the resonant frequencies; and for profiles and amplitudes of the dynamic water pressure at the resonant frequencies. The agreement obtained for Xiang Hong Dian Dam, in all comparisons, was quite remarkable and represents a significant step forward in understanding the dynamic behavior of concrete dams; the mode shape comparison is shown in Figure 5-5. Agreement for Quan Shui Dam, although good in some aspects, was not as consistent as that obtained for Xiang Hong Dian Dam. Possible reasons include the more complicated geometry of Quan Shui Dam, making it more difficult to model, a greater nonsymmetry that increased the interference between the

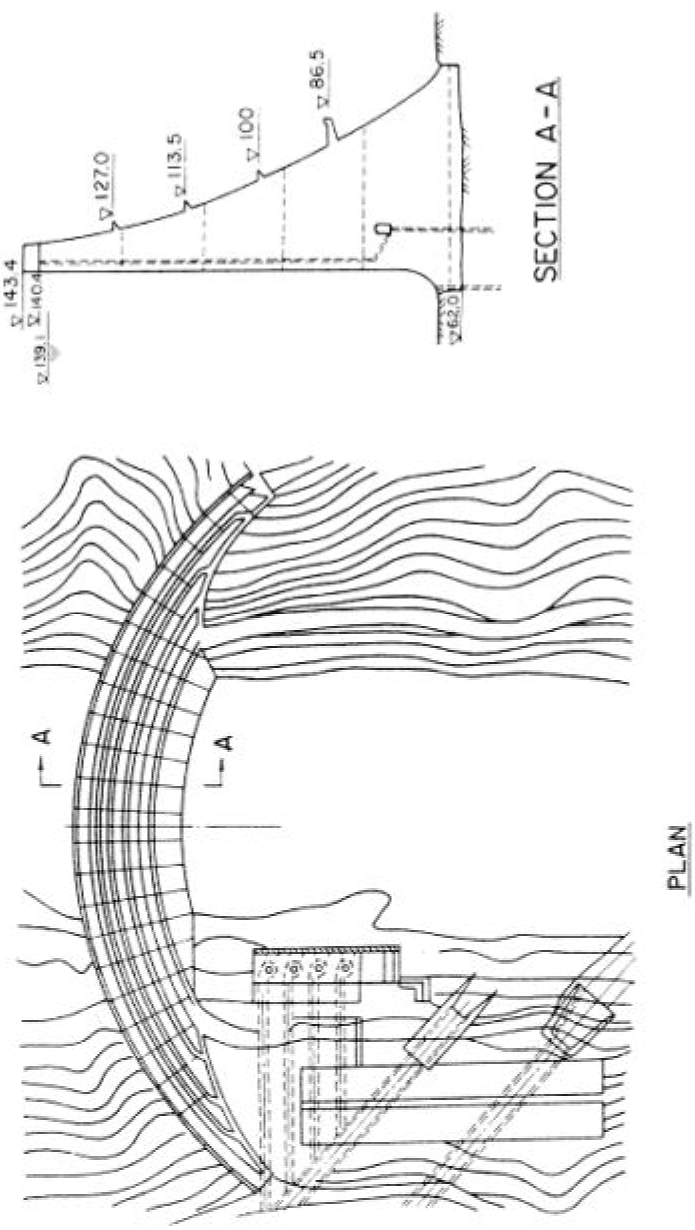


Figure 5-4 Layout of Xiang Hong Dian Dam, which has been the subject of extensive forced-vibration testing and correlation analyses (5-30).

symmetric and antisymmetric resonances, and the close proximity of the first two resonant frequencies.

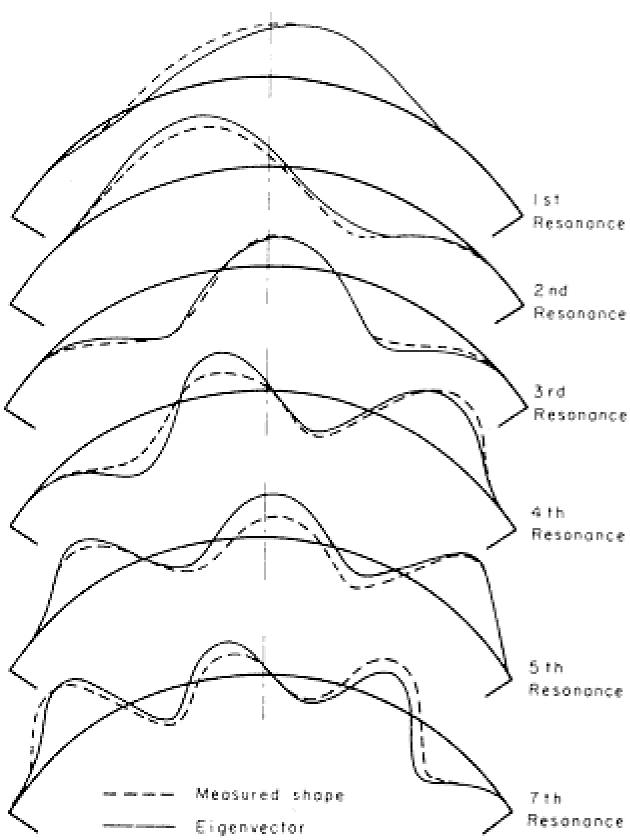


Figure 5-5 Comparison of measured vibration-mode shapes at the crest of Xiang Hong Dian Dam with shapes from finite element analyses (5-30).

For the study of Techi Dam (arch, 181 m high), the fit of the finite element model was carried out with both forced-vibration and ambient data taken at a reservoir water depth equal to 90 percent of the dam height. Good agreement was obtained between computed eigenfrequencies and measured resonant frequencies and between computed mode shapes and shapes measured under ambient conditions. Additional comparisons with the resonant frequencies from other ambient data at water depths equal to 85 and 95 percent of the dam height were good for the first resonance, but the finite element model with incompressible water predicted changes in the next three resonant frequencies for the variation in water level that exceeded the measured changes. These results demonstrate the desirability of evaluating a numerical model with data obtained at different water depths.

Comparisons between computed and measured results at Monticello Dam (arch, 93 m high) were good for resonant frequencies, resonating shapes of the dam crest, and frequency-response curves of the dam crest near the resonant frequencies. However, less-than-satisfactory agreement was obtained for amplitudes of the dynamic water pressures, including a too-rapid decay with distance from the dam in the numerical results. A repeat of the water pressure calculations with water compressibility included produced only slight differences from the results obtained with incompressible water. An attempt to isolate a resonance of the water domain by normalizing a measured dynamic-pressure-frequency response with a nearby acceleration frequency response of the dam was inconclusive.

From the studies performed on Xiang Hong Dian, Quan Shui, Techi, and Monticello dams, no obvious evidence emerged to suggest that the omission of foundation mass and the neglect of water compressibility in the analyses prevented adequate fits to the field data from being obtained. Regarding water compressibility, some effects may have been present in the measured data, especially for Techi Dam (5-50). Perhaps in the fit of the mathematical results to the data, neglecting water compressibility was compensated for in the adjustment of the values of elastic moduli and modal damping. It should be pointed out that successfully calibrating a mathematical model that uses incompressible water to forced-vibration or ambient data does not necessarily imply that neglecting water compressibility is appropriate for earthquake analysis. Because of the greater extent of the earthquake excitation over the boundaries of the water domain, water compressibility effects may be more important for earthquakes than under forced-vibration or ambient conditions.

The forced-vibration correlation study of Morrow Point Dam (Figure 5-6, arch, 142 m high) was performed with and without consideration of water compressibility, using mostly recent data obtained with full reservoir. Measurement stations at the face of the dam are shown in Figure 5-6. From this study it was concluded, in contrast to all previous experimental studies, that water compressibility can strongly influence the dynamic response of a dam. The presence of near-perfect symmetry, which eliminated interference between symmetric and antisymmetric responses, permitted calibration of the finite element model on the antisymmetric response, which, unlike the symmetric response, was little affected by water compressibility in the frequency range examined. Good matches were obtained to antisymmetric resonating shapes and frequency-response curves for both the dam accelerations and water pressure (Figure 5-7), whether or not water compressibility was included. Reasonable agreement was also achieved with some older data (5-37) for antisymmetric response at a significantly lower water level. However, use of the calibrated finite element model to predict the symmetric response data was not satisfactory, although it was somewhat better when water compressibility was included. Possibly water compressibility made the symmetric responses sensitive to the geometry and/or boundary conditions of the reservoir, which may not have been modeled accurately. Water compressibility effects were exhibited in the forced-vibration data through larger measured damping ratios for the symmetric resonances and by the presence of a "cantilever" crossover in the dynamic water pressures at the second symmetric resonance, where the resonating shape of the dam, with only a slight cantilever crossover, closely resembled the shape at the first resonance. An attempt to isolate the fundamental symmetric resonance of the reservoir water domain apparently succeeded by extracting the frequency-dependent amplitude of the water mode from the measured pressure data and normalizing it with the corresponding frequency-dependent measured motion of the dam. The isolated water resonance revealed a different behavior from that of the finite element compressible-water model. The work at Morrow Point Dam should be continued to determine if a numerical water model with response features similar to those of the actual reservoir water domain can be developed and if this will lead to better agreement between observed and computed dam responses. It should also be mentioned, as was pointed out in reference 5-39, that water compressibility effects at Morrow Point Dam may be more pronounced than

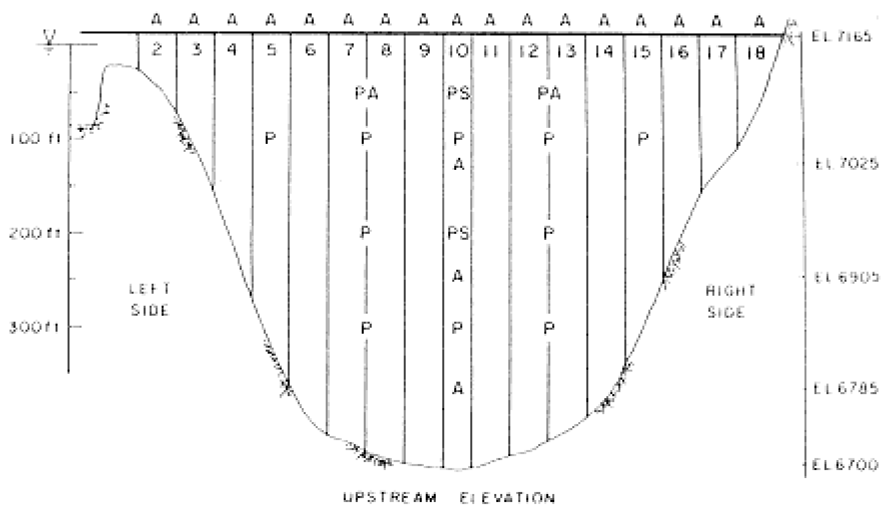


Figure 5-6 Measurement stations at the face of Morrow Point Dam where accelerations (A) and dynamic pressures (P) were recorded (5-39). (PS and PA indicate symmetrical-only or antisymmetrical-only tests.)

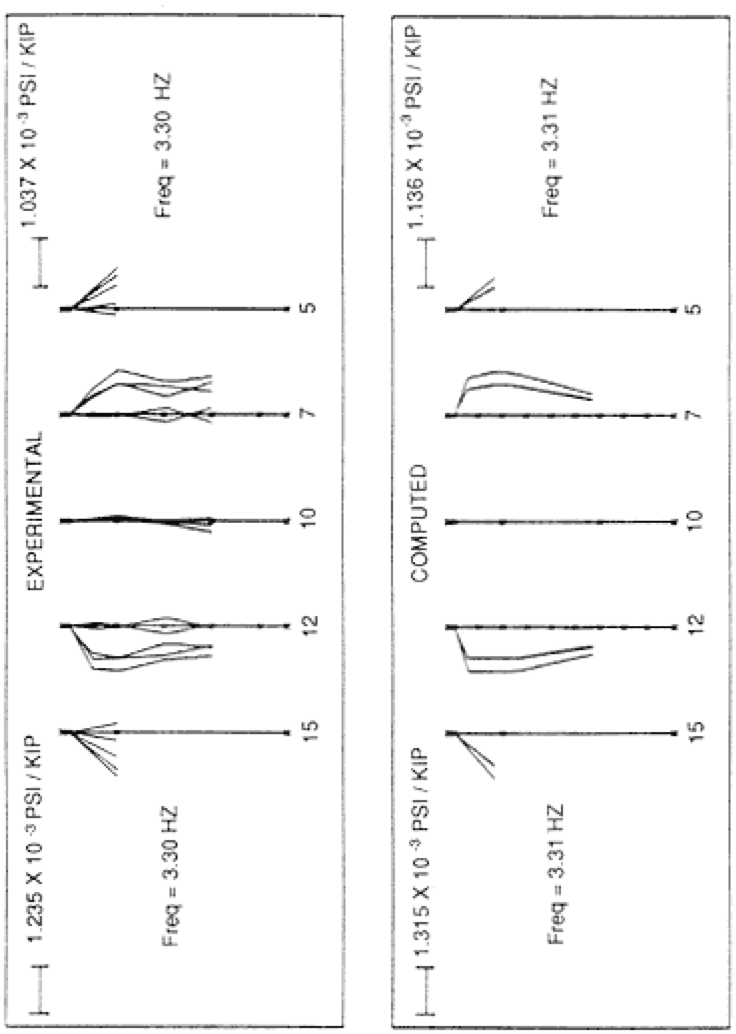


Figure 5-7 Comparison of measured and calculated dynamic pressure profiles on Morrow Point Dam in the first antisymmetric resonance (5-39).

for an "average" concrete dam because the ratio of the fundamental frequencies of the water and dam, which varies inversely with the importance of water compressibility, is lower for Morrow Point than the average.

As mentioned previously, a good test for a mathematical model is its ability to predict forced-vibration or ambient data at significantly different water depths, as was attempted at Techi (5-46) and Morrow Point (5-39) dams. For the dams listed previously for which such data are available, other analytical studies have been reported for Big Tujunga (5-18), Yugoslavia No. 2 (5-43), Pine Flat (5-28, 5-29), Wimbleball (5-22), and Kolnbrein (5-34). Analyses of Big Tujunga and Yugoslavia No. 2 dams were not very satisfactory, which may have been partially due to use of the lumped-added-mass representation of incompressible water. Studies of Pine Flat and Wimbleball dams considered only the fundamental resonance of a single monolith or buttress, and indications were that both sets of data were affected by variations in joint conditions. Remarkable analyses of Kolnbrein Dam for an empty to near-full reservoir employed a variation of the lumped-added-mass representation of incompressible water in which the added masses were adjusted for each resonance depending on the shape of vibration (distances between nodes of radial motion), so the significance of this study is not clear. Certainly, further work is needed in evaluating mathematical models using experimental data for significantly different reservoir water depths.

Another approach for evaluating a mathematical model is to compare resonant frequencies computed using experimentally determined values of elastic moduli of the dam and foundation, either by in situ geophysical means or from core tests, with those derived from forced-vibration field tests or ambient measurements. This is possible for a few dams in the United States: Pacoima (5-4), Big Tujunga (5-18), Lower Crystal Springs (5-2, 5-3), and Morrow Point (5-39). Satisfactory results were obtained only for Lower Crystal Springs and Morrow Point. Of course, problems may lie with either the mathematical models, the measured elastic moduli, or both. The redundant data obtained at Lower Crystal Springs and at Pacoima dams indicate that elastic moduli cannot be determined precisely. A number of other analyses of forced-vibration data have been carried out in which data on elastic moduli were available only for the dam (5-50).

MODEL TESTS

Dynamic tests on linearly elastic models have been used in a number of countries to determine the resonant frequencies and mode shapes of proposed or existing dams. One advantage of linear modeling is that it is possible to scale water compressibility effects properly. However, of the tests on models of actual dams that have been reported in some detail (Bai-Shan [gravityarch] [5-51], Hendrik Verwoerd [gravity-arch] [3-14, 5-52], North Fork

[arch] [5-53, 5-54], Pine Flat [5-55], and Victoria [arch] [5-56]), only the test of the Pine Flat model was in the range where water compressibility effects would be important (5-50). Analytical studies were performed for all of these model tests using data obtained under conditions of full and empty reservoir, and the agreement between computed and measured resonant frequencies and shapes ranged from poor to good. Some of this variation can be attributed to differences in the quality of the mathematical models. Best results were obtained for Bai-Shan and Hendrik Verwoerd using a finite element model of the dam and reservoir water; good agreement with measured dynamic water pressure distributions also was obtained at Bai-Shan. A recent experiment was designed to demonstrate water compressibility effects by employing a steel arch for the dam and was accompanied by numerical analysis that included water compressibility (5-57). However, insufficient published results prevent definite conclusions from being drawn. Two special small-scale experiments (5-58, 5-59), not involving dams but intended to indicate the importance of water compressibility effects in the earthquake response of concrete dams, gave inconclusive results; moreover, a number of objections have been raised regarding the validity of the experiments (5-50, 5-60).

Two recent shaking-table experiments conducted at very small scale (2-50) of a canyon, with or without a dam, and a large expanse of foundation rock sought to determine the degree of amplification of the earthquake motion in the upper portions of the abutments compared with that at the canyon bottom. Amplifications at the level of the crest equaled about 3 in one case where the abutments leveled off at crest level and about 2 in another case where the abutments extended higher than the crest. Reasonable agreement was obtained with both finite element calculations and ambient-field measurements. The presence of the dam had little effect on the rock motions. In contrast, results of a similar study employing incident waves (2-49) showed a strong effect from the presence of the dam, with maximum amplifications occurring at about midheight of the dam. Concerns regarding such model tests include boundary effects and inability to represent possible nonlinear behavior, which may occur in the actual rock mass.

Model tests are perhaps most useful when used to investigate nonlinear aspects of dam response (cracking, joint opening, sliding behavior under high compression, and cavitation in the water) and require special materials and usually a shaking table. The main difficulty with the model materials is that when the experiment is conducted under normal gravity, the strength and stiffness of the model materials compared with those of the prototype must be reduced by the product of the density and length ratios. Since the density ratio cannot practically be increased above about 2, the reduction required in strength and stiffness of model materials is large. Nevertheless, approximate small-scale versions of concrete have been developed that are typically plaster based with a high water content and lead powder added to increase density (4-3, 5-41, 5-45, 5-61 to 5-65). Water generally is used as the model liquid, but the use of a liquid having higher density can offset some of the reduction required in strength and stiffness of the model materials (5-45, 5-61). Use of a centrifuge produces the same effect to a greater degree, as has been reported once (2-49). Note that with nonlinear scaling, compressibility of water does not scale; neither does cavitation. However, separating the dam and fluid by a membrane with air access between the dam and membrane approximately accounts for cavitation at the upstream face of the dam (4-3). Such a membrane is actually necessary to prevent leakage of the fluid around the model if only the central part of the dam is being modeled and to prevent water contact with the plaster-based model material and consequent deterioration.

Although nonlinear model tests are common in Japan (5-16, 5-41, 5-62), the Soviet Union (2-49, 5-63, 5-64, 5-66), at ISMES in Italy (5-45, 5-61, 5-67 to 5-70), and at LNEC in Portugal (5-71, 5-72), very little detailed information from these sources is available in the earthquake engineering literature. What there is, though, indicates that nonlinear behavior is a very important feature in the response of concrete dams to strong ground shaking and that a dam undergoing nonlinear behavior can retain considerable stability. A recent experiment at LNEC, which was reported in some detail (2-62), described a test of a special system of joints in an arch dam designed to accommodate slip on a fault passing under the dam. Relative motions were applied pseudostatically to the perimeter of the dam, while dead and water loads were applied hydraulically. The model withstood prototype fault displacements of several meters without collapse; however, the test could not alleviate all concerns regarding water tightness.

A recent shaking-table test in Yugoslavia (5-73) investigated the effect of opening of the contraction joints on the earthquake response of arch dams. The model consisted of the central cantilever and halves of the adjacent ones and thus included two vertical joints; steel springs represented the arch support provided by the remainder of the dam. Horizontal cracks caused by the shaking initiated an overturning failure in the upper third of the central cantilever, where bending stresses had increased significantly following openings of the joints and loss of arch support. Keys were not included in the joints, so the effect of such features on the failure mechanism was not determined.

Only a few nonlinear model tests with valid scaling have been performed in the United States. A shaking-table test of a single monolith of Koyna Dam (4-3) produced a single crack extending all the way through the neck portion (Figure 5-8). The top block rocked back and forth but remained stable and continued to withstand the water load, even under intense excitations. Noticeable separations between the dam and a membrane supporting the

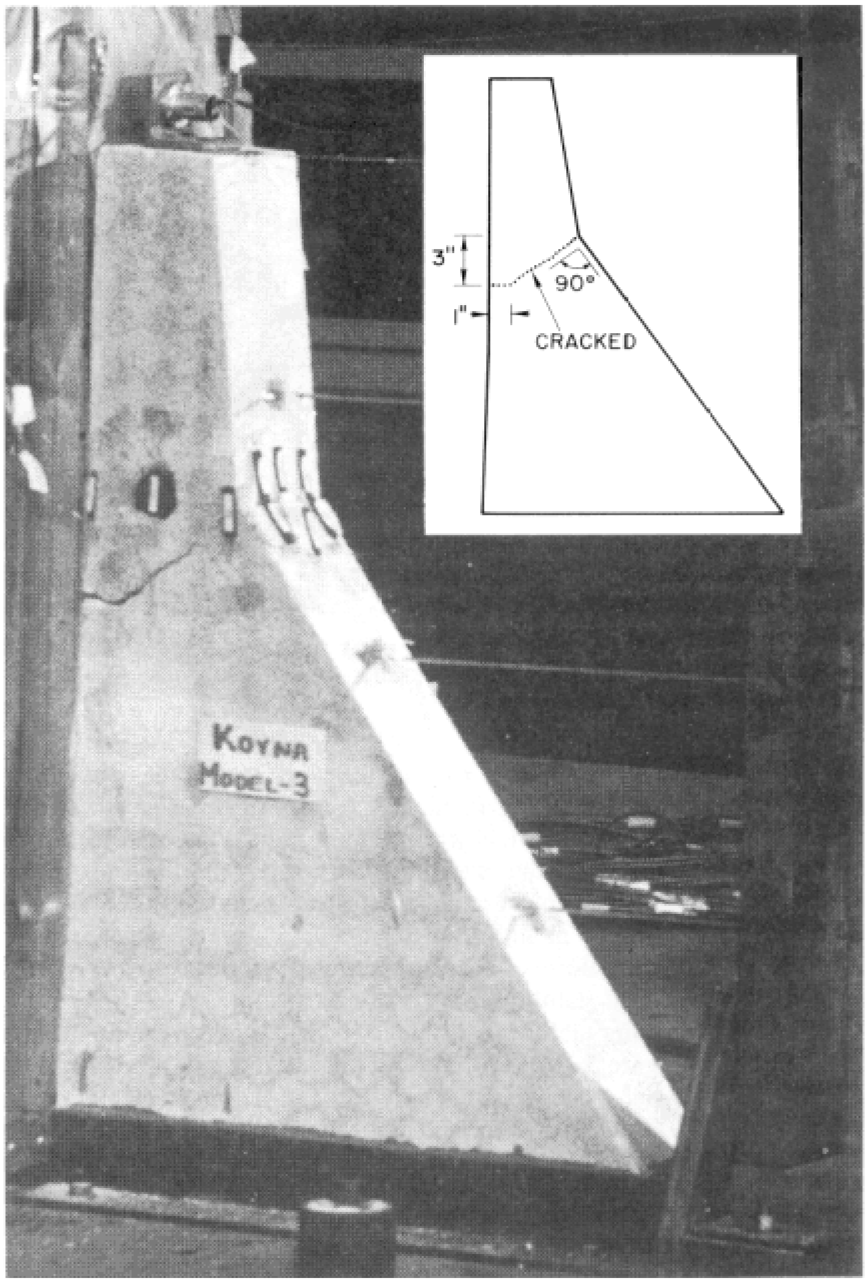


Figure 5-8 Model of single monolith of Koyna Dam; crack resulted from shaking-table test, including two-dimensional reservoir model (4-3).

water (i.e., cavitation) played a role in the response of the system. In another shaking-table test, using a jointed arch (4-3) to represent a horizontal cross section of an arch dam (shown in Figure 5-9), it was confirmed that opening of the contraction joints is an important response mechanism. Intense excitations were required to cause collapse of the arch, which was initiated by a compression failure at an abutment over the reduced contact area in a partially open joint. Both of these experiments confirmed the possible existence of continued stability in the nonlinear realm.

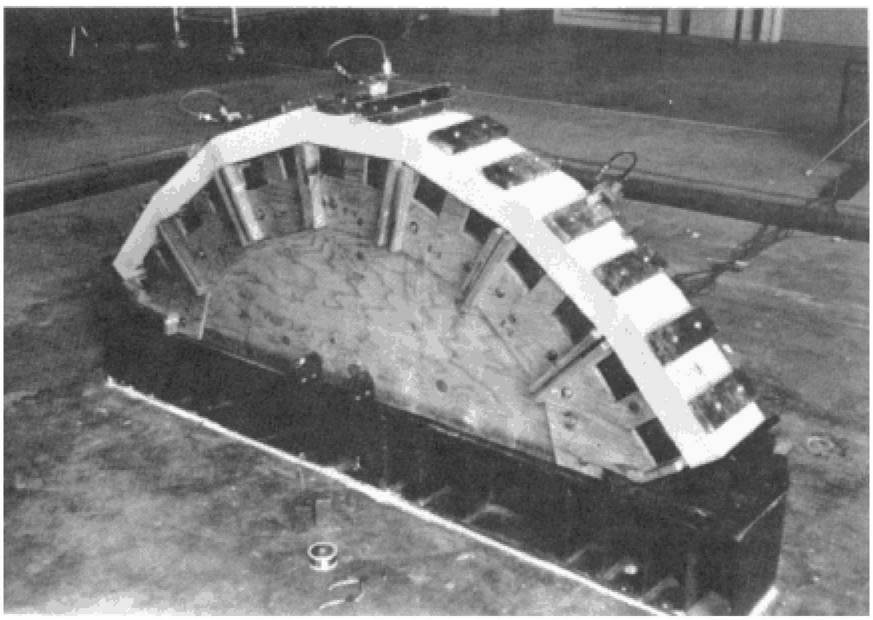


Figure 5-9 Segmented arch dam section model scaled to simulate effects of hydrostatic pressure and earthquake response inertia during horizontal and vertical shaking-table tests; it subsequently collapsed during severe shaking (4-30).

A series of shaking-table tests on three models of a single monolith of Pine Flat Dam (Figure 5-10) with full reservoir has recently been completed (5-65). Each model cracked all the way through the neck region, but no failures occurred, even during subsequent tests on the cracked models that employed severe table motions. The models remained stable because the crack profiles that developed were favorable: a V shape in one case and a dip in the upstream direction in the other two cases (Figure 5-11). Details of the cracking patterns were quite different among the models, even though the experiments were roughly similar. This suggests that cracking patterns in prototype dams may be sensitive to parameters describing the existing state of the dam and the excitation, which implies that reliable cracking

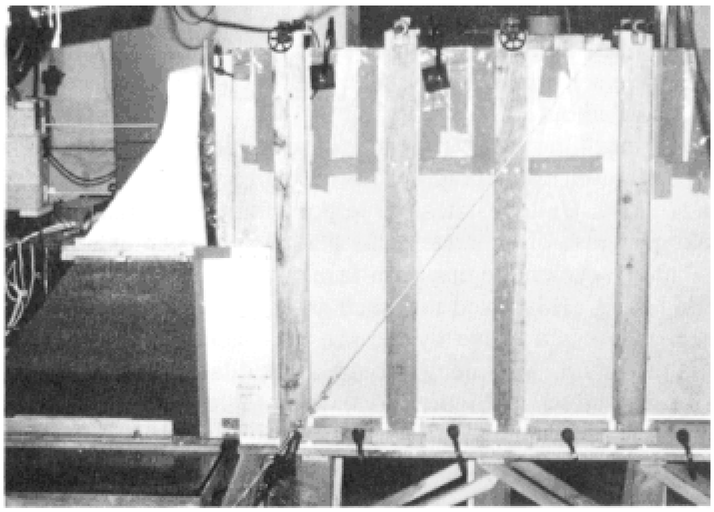


Figure 5-10 Setup for shaking-table test of scaled model of Pine Flat Dam monolith, including two-dimensional reservoir (5-65).

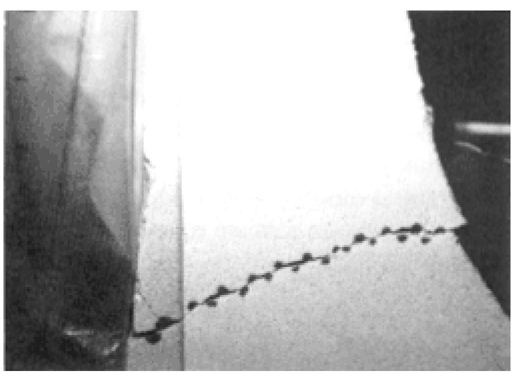


Figure 5-11 Model of Figure 5-10 shown after testing; the sliding displacement visible along the crack is equivalent to 2 ft of prototype movement (5-65).

analyses may be difficult to perform. Although the three models exhibited excellent stability during and after cracking, the possibility of developing an unfavorable crack profile, such as one that dips in the downstream direction, could not be ruled out.

Certainly, much more experimental work is needed on nonlinear models, including three-dimensional ones. This is an area where a glaring deficiency now exists in the United States. One potentially important feature of the earthquake response of concrete dams that has not been examined is water intrusion into cracks on the upstream face of a dam, especially as facilitated by crack opening. It is noted that such an occurrence was prevented in the experiment mentioned above by the use of a membrane between the dam model and the liquid. Slip along a rough crack dilates the crack (as occurred for the model shown in Figure 5-10) and would greatly facilitate water intrusion. The combination of an unfavorable crack profile with water intrusion would be an event of considerable concern. Another issue regarding such model studies is a lack of knowledge about the fracture mechanics of the plaster-based model materials; that is, how well does the critical stress intensity factor (a material property that governs crack propagation) scale?

RESEARCH NEEDS

1. Data Collection from Actual Earthquakes

The collection of data from actual earthquakes should be accelerated through increased instrumentation because of the current sparsity of records from dams and their foundation rock for moderate and intense shaking. Such records can provide a general understanding of the dam response; they are the only reliable source of information on the spatial variations of rock motions that occur in complicated site geometries and it is critical to include such variations in analyses. Also, the motion of the reservoir bottom upstream of the dam, which provides seismic excitation to the water, must be measured. Records of dynamic water pressure response would be very useful; practically none exist. Consideration should also be given to measuring movements associated with contraction joints, such as joint openings for arch and gravity dams and relative tangential motions across unkeyed joints in gravity and buttress dams. Accurate earthquake data are essential for guiding development of appropriate mathematical models.

2. Forced-Vibration Field Tests

Forced-vibration field tests, carefully carried out, are the preferable means for investigating dam-reservoir water interaction and water compressibility effects. Important work has been accomplished in this area, but in many cases correlations with analytical results have been poor. Field tests must be planned to obtain data for significantly different water levels and should include measurements of dynamic water pressure against the dam and also upstream from the dam face. Dam-foundation interaction can also be studied via forced-vibration tests and should include measurements of the rock motion at the dam-foundation interface and beyond.

3. Nonlinear Phenomena

Nonlinear phenomena play an important role in the response to strong shaking of a large concrete dam with full reservoir. A major concentrated effort is needed in this area, about which little quantitative information is available, using small-scale model tests on shaking tables. A high-frequency capability is necessary for the shaking table because of the very small scales that must be used. Difficulties associated with carrying out such experiments in a valid way (e.g., in reproducing joint keys and in allowing intrusion by the liquid into cracks) should not be underestimated. Attempts should be made to learn as much as possible from the experiences of laboratories in other countries, and cooperative research programs should be planned to take advantage of foreign facilities that have no counterparts in this country—such as large-capacity shaking tables with high-frequency capabilities.

6

Recommended Criteria for Evaluating Seismic Performance

BACKGROUND

To evaluate the seismic performance of a concrete dam subjected to moderate or strong earthquakes, it is essential to have appropriate criteria. As described in the preceding chapters, the seismic performance of concrete dams in most cases is evaluated by using an analytical model of the dam based on numerical techniques, usually the finite element method of analysis, together with an appropriate characterization of the earthquake. To assess seismic performance using results from finite element or other numerical analyses, criteria are needed that relate the numerical results from such analyses to the expected behavior of the dam.

As described in Chapter 3, when earthquakes were first considered in the design and analysis of concrete dams, earthquake effects were characterized in terms of "equivalent" static forces. The amplification of accelerations through response of the dam was assumed to be either negligible or improbable, and equivalent static forces for seismic conditions were simply added to forces determined for true static loading conditions. The analytical results for combined loading conditions including earthquake effects were not evaluated any differently than results for normal static loading conditions. Consequently, concrete dams were considered to be safe during earthquake loading conditions if computed tensile stresses were small or nonexistent, if resultant forces in two-dimensional sections through gravity dams fell within the central one-third of their bases, and if compressive stresses were computed to be less than an allowable working stress (usually 1,000 psi or less).

By the 1950s the importance of the dynamic earthquake response of typical structures such as buildings and bridges was recognized. However, it was still commonly assumed that concrete dams were stiff enough that amplification of earthquake ground motions through structural response was insignificant. Although earthquake effects were still being characterized by equivalent static forces, it was recognized that, given the relatively infrequent occurrence and short duration of earthquakes, it was appropriate to apply criteria having less conservatism for loading conditions that included earthquakes than for loading conditions without earthquakes (3-9).

However, in the late 1960s it was realized that although concrete dams are relatively stiff structures, substantial amplitudes of earthquake ground motions could occur at frequencies well within the frequency range of response for concrete dams, and the resulting response amplification should not be ignored. The dynamic response behavior of a concrete dam, together with dam-water interaction, was recognized as a key factor in correctly understanding and evaluating the dam's seismic performance. This increased understanding was facilitated in part by the occurrence of the Koyna Dam earthquake in 1967, described in preceding chapters. However, it was not until the early 1970s that analytical tools, which included methods for modeling dynamic response and reservoir water interaction, principally finite element procedures, became readily available to those performing seismic stability analyses.

Improvements in analytical procedures required corresponding modifications of the criteria used to evaluate analytical results. The criteria that had been used to evaluate results from simplified pseudostatic analyses were not generally appropriate for evaluating results from two-and three-dimensional dynamic finite element analyses. More applicable criteria began to be developed partly as a result of dynamic finite element analyses conducted for concrete dams that successfully withstood the relatively large ground motions associated with the 1971 San Fernando earthquake in California. Criteria were eventually set forth whereby computed tensile stresses large enough to indicate the initiation of cracking, and compressive stresses larger than the allowable working stress level, were understood to not necessarily indicate structural instability (6-1). The existence of foundation characteristics that could contribute to instabilities and the effects of strong earthquakes on foundation stability also became better understood during the 1970s.

In the 1980s many of the research developments related to the numerical analyses of concrete dams have focused on nonlinear material behavior and nonlinear analytical techniques, as described in Chapter 4. Although some nonlinear analyses have been performed to evaluate certain aspects of nonlinear response, the analytical techniques and models necessary to reliably and economically perform complete nonlinear analyses of concrete dams, together with their associated reservoirs and foundations, are not yet sufficiently developed to be applied during routine engineering analyses. Consequently, criteria used to evaluate numerical results from earthquake analyses of concrete dams, predominantly results from linear elastic finite-element analyses, remain largely unchanged from those developed for linear response analyses in the

1970s. These criteria and their application and adequacy are the focus of this chapter. However, some preliminaries concerning initial conditions and analytical procedures must be addressed before these criteria can be considered. Inappropriate recognition of initial conditions or misapplication of analytical procedures may produce numerical results that cannot be meaningfully related to expected seismic performance regardless of the criteria used.

PRELIMINARY CONSIDERATIONS

Initial Conditions Resulting from Static Loads

Since the initial condition of a concrete dam prior to the occurrence of an earthquake is the result of previously applied static loads, it is essential that the effects of these static loads be adequately quantified before completing a seismic evaluation. Thus, studies must be performed to determine the effects of dead loads, water loads, and temperature distributions, in appropriate combinations. In addition, the effects of any uncommon loads or conditions, such as expansion due to alkali-aggregate reaction, must be included. Without a thorough understanding of the existing static condition of a concrete dam, an evaluation of the dam's seismic performance may be meaningless.

Effects of Construction Sequence

Generally, static dead load effects are easily accounted for in the analyses of concrete dams, particularly when two-dimensional modeling is adequate. However, when three-dimensional analyses are required, spurious stresses may be indicated when the gravity loading is applied to the model as if dead load effects occur only after construction of the dam is completed, rather than on portions of the dam as they are constructed. Additional discrepancies can occur when three-dimensional modeling neglects water loads from partial filling of the reservoir prior to completion of construction. Further discrepancies can arise when the successive grouting of portions of contraction joints between partially completed monoliths of a conventionally constructed dam is ignored. Neglecting these factors in a three-dimensional analysis can lead to the indication of fictitious tensile stresses along abutments and horizontal loads or thrusts that are incorrect, particularly in the case of double-curvature arch dams.

To reduce these inaccuracies in a three-dimensional linear elastic analysis of a conventionally constructed concrete dam, alternate monoliths in the model of the completed dam can be assigned zero mass and zero stiffness. The gravity loading can be applied to the remaining monoliths, which will support the gravity load through simple cantilever action. The analysis can then be repeated with the other monoliths to obtain the gravity load state of stress for the entire dam. In some cases this simplified approach will be sufficient. However, in other cases, especially for large double-curvature arch dams, it may be necessary to perform a sequence of analyses (as many as five or more) to simulate construction of the dam in blocks, partial grouting of contraction joints, changing temperature distributions after grouting, and staged filling of the reservoir (6-2). Similarly, a series of analyses that simulate the sequence of construction for some roller-compacted concrete dams may be necessary to adequately account for construction of these dams in horizontal layers.

Temperature Effects

The effects of changing temperature distributions can also influence the seismic performance of concrete (and even masonry) dams and often are not fully considered, especially when older existing structures are analyzed. After concrete or masonry hardens, it expands and contracts in response to temperature changes. Temperature changes in concrete dams occur because of variations in the temperature of the surrounding air, variations in reservoir water temperatures, and, to a lesser extent, solar radiation at exposed surfaces. These temperature changes cause strains, which if restrained result in stresses. Thermally induced stresses and strains can be significant and as a result can affect the strain capacity available to resist earthquake inertia forces without the concrete being damaged.

For concrete gravity dams and buttress dams that are essentially two-dimensional in their behavior, concrete temperature changes can usually be ignored. However, for concrete dams whose behavior is three-dimensional, including some concrete gravity dams (see section below titled Two-Dimensional Versus Three-Dimensional Analytical Models), the effects of temperature changes must be assessed in order to adequately evaluate seismic performance. When concrete temperatures are important to consider, it often is not sufficient to characterize them as uniform distributions. Rather, it is usually necessary to consider nonuniform, linearly varying temperature distributions. In some cases, particularly for thin concrete arch dams, it may even be necessary to consider nonlinearly varying temperature distributions.

Creep Effects

Concrete is a brittle material that exhibits significant variations in its elasticity depending on its age when loaded, the rate of loading, and the duration of loading. Most of the laboratory tests conducted to determine modulus of elasticity and compressive strengths of concrete are performed with loading applied at a rate of about 30 to 40 psi/sec, such that tested samples are taken from zero load to failure in 2 to 3 min. The resulting modulus and strength values can appropriately be termed instantaneous or short-term static values. However, static loads that are actually experienced by concrete dams may exist for many years. Under sustained loads concrete exhibits pronounced creep (increasing strain with time at constant load), which affects displacements due to external loads, such as gravity and reservoir water loads, and stresses due to internal loads, such as those induced by thermal strains. Creep effects should be accounted for in all analyses, but are particularly significant if nonlinear behavior is being considered.

For linear elastic analyses it is not appropriate to represent stress-strain behavior with a single modulus-of-elasticity value obtained from short-term laboratory loads. The most common means of accounting for creep in linear elastic analyses of concrete dams is to use a sustained-load modulus of elasticity, which is less than the short-term static modulus typically measured in the laboratory. Based on results from numerous uniaxial laboratory tests (6-3), a reduction of about 25 to 30 percent from the short-term value appears to be appropriate, depending on the duration of significant components of static loads. However, more research concerning the creep behavior of the mass concrete used to construct dams is needed, particularly for multiaxial loading conditions.

Uplift

Another factor that can have a significant influence on the seismic performance of a concrete dam is uplift. Uplift is the result of interstitial water, which carries a portion of the normal compressive loads applied to mass concrete in dams and their foundations. Under static loading conditions the effect of these pore pressures is to reduce the normal compressive stresses acting within the concrete and to increase any normal tensile stresses, should they exist. The exception is an open crack, in which case water in the crack produces external loads along both faces of the crack.

The stresses that should usually be considered when evaluating the stability of concrete dams, except for thin arch dams, are "effective" stresses. At a particular location the effective stress equals the difference of the total stress due to external loads and the pore pressure or uplift at that location. Since uplift has not been shown to significantly affect the stress distribution in thin arch dams, and since the stability of arch dams is dependent on their ultimate capability to carry loads in compression, normally it is not considered necessary to reduce compressive total stresses to effective stresses when evaluating the stability of most arch dams. Except for thick arch dams, normal practice is to neglect uplift pressures and to base evaluations of numerical results on total stresses rather than effective stresses.

For concrete gravity dams subjected to static loads, uplift increases the tendency for cracking along the upstream face of the dam and reduces compressive normal stresses within the dam structure and along the dam-foundation contact, which correspondingly reduces sliding stability. To include the effects of uplift in a static analysis using the finite element method, an analysis can be performed in the usual manner, neglecting pore pressures and calculating total stresses. Pore pressures, based on appropriate considerations regarding the presence and effectiveness of drainage systems within the dam and its foundation, can then be summed with total stresses to calculate effective stresses (6-4, 6-5).

For seismic loading conditions, where a dam oscillates rapidly, the present state of practice for linear elastic analyses is to not attribute any additional significance to pore pressures beyond their effects during static loading conditions. During seismic loading conditions, as the dam moves upstream, the upstream portions of the dam carry the inertia load in compression, resulting in higher pore pressures, while the stresses in the downstream portions of the dam tend toward tension, causing reductions in pore pressures. When the movement of the dam reverses, pore pressures tend to be reduced in the upstream portions of the dam while increasing in the downstream portions. Since the increase in pore pressures in compressive zones is usually accompanied by a larger increase in total stress, higher pore pressures do not significantly affect stability during seismic loading conditions. Should cracking occur in portions of the upstream face during an earthquake, it is usually assumed that oscillations occur quickly enough to prevent significant penetration of water into the cracks. Consequently, pore pressures are usually treated as though they are constant during both static and dynamic loading conditions. However, research to more accurately characterize the effects of pore pressures during earthquakes is needed, especially when the response is nonlinear.

Deformation Modulus of Foundation Rock

The deformation of foundation rock due to loads applied from a concrete dam influences seismic response and the stresses that develop in the dam, particularly near the dam-foundation contact. The fundamental property that represents the deformation characteristics of a dam's foundation is termed the deformation modulus.

In linear elastic analyses, the deformation modulus is an effective elastic modulus that represents both elastic strain of the rock mass and also the inelastic deformation of discontinuities. Values for deformation modulus can be determined from in situ testing, laboratory testing, empirical relationships, comparisons of analytical results with measured prototype behavior, or various combinations of these methods. The appropriate approach for determining deformation moduli will vary depending on the size of the dam, the severity of anticipated loading conditions, the availability of data characterizing foundation conditions, and the status of the dam as an existing structure or one under design. Often, the deformation characteristics of the foundation underneath a concrete dam will vary spatially. In such cases, more than a single deformation modulus may be necessary to adequately represent differing deformation behavior in various zones of the foundation.

There are no known data that indicate that deformation moduli increase significantly during seismic loading conditions. Therefore, the same values of deformation moduli are usually used for seismic response analyses as for corresponding static analyses. If deformation moduli should increase by 25 percent, for example, during seismic loading conditions as compared with static loading conditions, this would not appreciably affect the frequency characteristics of the dam-foundation system, the seismic response of the system, or the stresses near the dam-foundation contact.

Two-Dimensional Versus Three-Dimensional Analytical Models

It is commonly accepted that three-dimensional analyses are necessary to adequately evaluate concrete arch dams. Probably the only situation in which two-dimensional analyses might be appropriate for an arch dam would be a "worst-case" evaluation for a gravity arch structure. Concrete gravity dams, on the other hand, are usually evaluated on the basis of two-dimensional analyses. While two-dimensional analyses are appropriate in many cases, there are situations where three-dimensional analyses should be considered.

In some of these cases concrete gravity dams, particularly roller-compacted concrete gravity dams, are located in relatively narrow sites, having crest-length-to-height ratios of 5 to 1 or less. In other cases concrete gravity dams are located at sites that are extremely irregular in shape from abutment to abutment. Such sites can provide additional restraint or can result in structural movements from applied loads that cannot be accounted for in two-dimensional analyses. Generally, two-dimensional analyses for static loading conditions are conservative; but while this conservatism may be appropriate for static loads, it may be misleading for seismic loading conditions. For example, in the case of a relatively narrow site where additional restraint is provided by the abutments, the restraint can increase compressive stresses near the heel of the dam along the upstream face during static loading conditions, thereby reducing the magnitude of tensile stresses developing during seismic conditions. Two-dimensional analyses will almost always indicate the development of high tensile stresses during large earthquakes at these locations, whereas three-dimensional analyses may indicate much lower stresses (6-4). In such cases two-dimensional analyses may predict stresses large enough to cause cracking, while three-dimensional analyses may indicate that cracking will not occur at all.

It should be noted that in relatively narrow sites the fact that a concrete gravity dam has numerous vertical contraction joints that are unkeyed is not sufficient justification by itself for performing two-dimensional analyses rather than three-dimensional analyses. In relatively narrow sites the horizontal forces perpendicular to the contraction joints can be large enough to develop significant shear friction resistance, so that adjacent monoliths do not respond to loading independently of one another, as is often assumed. These horizontal forces can result from three factors: (1) small displacements parallel to the abutments toward the bottom of the site caused by gravity, (2) twisting of adjacent monoliths caused by water loads, and (3) thermal strains.

GUIDELINES FOR EVALUATING RESULTS FROM LINEAR ANALYSES

At present, seismic safety evaluations of concrete dams are usually based on numerical results from linear dynamic finite element response analyses. The evaluations are in large part based on comparisons of computed levels of stress with levels of stress deemed to be acceptable considering concrete strengths and the probability, at least in a qualitative sense, that the postulated earthquake ground motion will occur.

The following paragraphs are intended to aid in understanding concrete behavior during earthquake loading and to provide a means for estimating reasonable concrete strengths. However, these are simply guidelines for making estimates. When performing a seismic safety evaluation of a concrete dam, it is imperative that an appropriate amount of work be conducted to investigate the characteristics and strengths of the specific concretes used or proposed for a particular dam. The following guidelines are not intended to replace those necessary investigations.

Concrete Strengths During Earthquake Loading

As previously discussed, concrete is a brittle material that exhibits significant variations in its elasticity depending on the rate of loading. For the rapid loading rates that occur during oscillatory inertia loads associated with earthquakes, the concrete modulus-of-elasticity values increase. Typically, concrete in a dam during an earthquake can experience minimum strain followed by maximum strain in 0.1 sec or less. By conducting tests under similar conditions, it has been shown that the dynamic uniaxial modulus-of-elasticity values for concrete in constructed dams are about 25 percent higher than the values obtained from typical short-term laboratory tests (5-3, 5-22).

Although failure strains remain about the same as for short-term static loads, the material stiffening that occurs during rapid loading, shown by increased modulus of elasticity, affects concrete strengths and thus the acceptable levels of computed stresses.

Compression

Concrete core samples have been obtained by drilling from a number of dams in the United States and used to perform laboratory compression tests at loading rates comparable to inertia loading rates during earthquakes. For these concretes the uniaxial compressive strengths measured during rapid loading rates were 12 to 52 percent larger than compressive strengths of comparable samples measured during typical short-term tests (4-12). Since only two of the rapid loading test results published to date indicated compressive strengths substantially higher than about 30 percent above typical standard static test values, increases in uniaxial compressive strengths of 25 to 30 percent above static strengths appear reasonable.

Tension

Core samples obtained by drilling at a number of dams have also been used to perform laboratory tensile tests at loading rates comparable to those attained during earthquakes. For these dams uniaxial and modulus-of-rupture tensile strengths measured during rapid loading rates were 31 to 83 percent larger than tensile strengths of comparable samples measured during typical short-term tests (4-12). Based on these results and others (3-18), an increase of 50 percent above static tensile strengths appears reasonable.

The general relationship between tensile strength and compressive strength of concrete is not linear. However, for the range of compressive strengths common for concretes used to construct dams, the intact uniaxial static tensile strength is approximately 10 percent of the static uniaxial compressive strength (4-12). When increased 50 percent for rapid loading conditions, intact uniaxial concrete tensile strengths approximately equal to 15 percent of static uniaxial compressive strengths are presently considered appropriate for evaluating the seismic performance of concrete dams. However, it is important to note that very few data exist concerning failure strains and stresses during rapid strain-rate multiaxial loading conditions.

Because concrete does not demonstrate a linear relationship between stress and strain, except at relatively low levels of applied loading, and because most seismic evaluations are based on linear elastic analyses rather than nonlinear analyses, some investigators have proposed the use of an apparent tensile strength rather than actual tensile strength for such evaluations (4-12, 6-6, 6-7). The apparent tensile strength is equal to the tensile stress corresponding to actual uniaxial tensile strain at failure assuming a strictly linear relationship between stress and strain. Under rapid loading conditions the apparent tensile strength is approximately 25 percent greater than the actual uniaxial tensile strength. This results in apparent rapid loading tensile strengths that are approximately 20 percent of static uniaxial compressive strengths for the range of compressive strengths common for concretes used to construct dams.

While the concept of an apparent tensile strength is valid for evaluating the results from linear elastic numerical analyses for severe seismic conditions, it is important to remember that for all concrete dams the limiting tensile strength of the concrete is that which exists across lift surfaces (the horizontal surfaces between concrete placements that are typically spaced at intervals of 1 to 10 ft over the height of the dam, depending on the type of dam and the type of construction) and across the contact surfaces between the dam and its rock foundation. Even in cases where particular efforts are implemented during construction to prepare concrete and foundation contact surfaces for bonding, uniaxial tensile strengths across bonded lift surfaces and foundation contacts should be expected to be at least 10 to 20 percent less than corresponding intact tensile strengths without lift or contact surfaces (3-18). While the decrease in tensile strength across bonded lift surfaces and foundation contacts is an independent effect and has no relationship to the nonlinearity of the stress-strain curve, under ideal conditions the two effects essentially offset one another in terms of the tensile strength appropriate for evaluating the results from linear elastic analyses. For dams where no specific construction techniques, such as high-pressure water jetting, were employed to achieve bond between lift surfaces and foundation contacts, significant portions of the surfaces will probably not be bonded, and additional reductions in the tensile strength used to evaluate seismic performance should be considered, beyond the 10 to 20 percent indicated by the limited data available for ideal conditions.

It is interesting to note that in some countries outside the United States the tensile strength of concrete is discounted or routinely assumed to be zero when evaluating the performance of concrete dams (6-8). Since computed tensile stresses are expected from linear elastic analyses of concrete dams during significant earthquake excitation, discounting tensile strength necessitates performing nonlinear analyses (6-8). However, given the amount of data suggesting that some reasonable tensile capacity can be expected in most cases, as well as the uncertainties in the results from nonlinear analyses performed to date, it is not recommended at present that the tensile capacity of concrete be ignored in favor of performing nonlinear analyses.

Shear

The shear resistance of concrete in dams, and along contacts with rock foundations, is usually assumed to follow Mohr-Coulomb relationships. Consequently, the shear resistance across a plane consists of the sum of two forces: intact shear strength (usually termed cohesion) multiplied by intact area, and frictional resistance (coefficient of friction or tan ø) multiplied by normal load. Cohesion values, or zero normal load intact shear strengths, are typically about 10 percent of static uniaxial compressive strengths based on direct shear tests of concrete core samples, and coefficients of friction are typically near 1. These static values are not usually increased to account for rapid loading rates, because there is a lack of data documenting any change in shear strength.

Evaluating Seismic Performance

At present the process of evaluating the seismic performance of concrete dams using results from linear elastic numerical analyses, finite element or other, is in most cases deterministic. The process involves comparing computed levels of stress with levels that are considered acceptable based on considerations of concrete strength and the likelihood of significant earthquakes occurring. If computed levels of stress are generally less than or equal to levels considered acceptable, the seismic performance and safety of the dam are considered acceptable. If computed levels of stress exceed acceptable levels, the seismic performance of the dam may be considered unacceptable. If computed levels of stress exceed material strengths, a rational analysis of the seismic performance of the dam is considerably more difficult and may not be possible based on the results of linear analyses alone.

An earthquake that causes the largest ground motion expected to occur at a dam site at least once during the economic life of the dam, usually taken to be 100 years, is commonly termed a design basis earthquake (DBE). A DBE should be considered a design loading condition, although in most cases it is appropriate to consider it as an unusual loading condition. When evaluating the performance of a concrete dam for a DBE, it is appropriate to apply factors of safety to concrete strengths and require that there be no excessive damage, no irreparable damage, no life-threatening uncontrolled release of the reservoir water, and no interruption to systems or components needed to maintain safe operation.

An earthquake capable of producing the largest ground motion that could ever be expected at a dam site is commonly termed the maximum credible earthquake (MCE). An MCE should be considered to be an extreme loading condition. When evaluating the performance of a concrete dam for an MCE, criteria based on factors of safety applied to concrete strengths are not applicable since extensive damage, including extensive irreparable damage, is acceptable. However, it is still appropriate to require that there be no life-threatening uncontrolled release of reservoir water and no interruption to operating systems required for safety. Criteria for concrete dams presently used by the Bureau of Reclamation (6-1), the U.S. Army Corps of Engineers (6-9), and the Federal Energy Regulatory Commission (6-10) are generally consistent with this concept.

Evaluating the seismic performance of concrete dams is considerably more complex than evaluating their static performance, particularly when the postulated earthquake motion is severe. For seismic analyses where all of the computed dynamic stresses summed with static stresses are below acceptable levels, the evaluation is straightforward. However, for MCE analyses when the ground motion is severe, such a simple evaluation usually is not possible, and considerably more effort is required to understand the dam's performance and provide a realistic evaluation. In such cases time history analyses are usually required, together with a considerable amount of postprocessing of the response data. If the computer program used to perform the analyses is not capable of calculating stresses in the planes of element faces, postprocessing should be performed to calculate face stresses in the appropriate local coordinate system. Postprocessing is also useful in determining locations (where stresses large enough to be of concern are expected to occur) that could experience large stresses at any particular time during an earthquake and the number of times the large stresses are expected to be repeated during the earthquake. In addition, the capability to produce plots showing time histories of stresses at particular locations, along with overall stress distributions at specific times, is especially useful.

Arch and Gravity Dams

When evaluating the seismic performance of a concrete dam, it is important that computed stresses at the dam faces be resolved into arch (or horizontal in the case of gravity dams) and cantilever components and that principal face stresses be calculated at significant instants of time during the earthquake. Other than foundation stability and bearing capacity of the foundation rock, the ultimate load-carrying capability of concrete dams is limited by the compressive strength of the concrete (and by sliding resistance for gravity dams). Thus, it is important to confirm that maximum computed earthquake compressive stresses do not approach the concrete's rapid-loading compressive strength. Computed compressive stresses will usually be well below this compressive strength limit; however, it is not unusual for computed tensile stresses to exceed the concrete's rapid-loading tensile strength when ground motion is severe.

Unless the numerical model developed to perform the seismic analysis accounts for the existence of vertical contraction joints, horizontal tensile stresses will usually be indicated near and above the reservoir water surface elevation. These tensile stresses may occur on only one face of the dam at any given time or on both faces simultaneously, particularly for loading conditions involving minimum concrete temperatures. Generally, these tensile stresses are not of concern unless accompanied by vertical tensile stresses acting in the cantilever directions over a significant portion of the dam. Since vertical contraction joints have little or no tensile capacity, it is reasonable to assume that the indicated horizontal tensions would be replaced by slight openings of the contraction joints. When contraction joint openings are thought to occur, the potential for and consequences of any subsequent load redistribution should be assessed. Since the analyses discussed in this section are assumed to be linear elastic, load redistributions and the resulting increases in other stresses cannot be precisely quantified. However, on a qualitative basis it is possible to estimate whether sufficient reserve load-carrying capacity exists in the adjacent sections and whether the stability of the dam can be considered adequate.

When tensile stresses that approach or exceed the rapid-loading tensile strength are computed in directions other than normal to contraction joints, cracking should be assumed to occur. However, so long as the tensile stresses occur over a very limited extent of the dam, do not repeatedly exceed the tensile strength, or are the result of modeling anomalies (such as the contact between the dam and its foundation), it is reasonable to conclude that the cracking does not necessarily indicate unacceptable performance for extreme events. However, the extent of such cracking must be estimated, and computed stress distributions must indicate that adequate compressive capability exists to accommodate subsequent load redistributions. In addition, the overall distributions of principal stresses that develop at particular instants of time should be evaluated to confirm that indicated regions of tensile cracking are not likely to join together to form surfaces along which partial sliding failures could occur, if such failures would result in a significant life-threatening reservoir release.

In the case of arch dams analyzed assuming linear elastic material behavior, the structure may exhibit a tendency toward developing a partial failure, usually resembling the shape of a semicircular or rectangular notch in the upper central portion of the dam, if the calculated seismic stresses are large. Whether such partial failures could actually occur is unknown, since they have not actually been observed. However, if the partial failures are credible, their development would primarily depend on the extent of cracking, the orientation of cracking, and whether arch action can restrain the notch-shaped portions separated by cracking.

In the case of gravity dams, unless both the upstream and downstream faces are sloped or the dam is unusually stiff, most structures exhibit a tendency toward developing horizontal cracks on both the upstream and downstream faces in the upper half of the dam when large stresses are produced by earthquake inertia loading. Whether such cracking indicates that sliding stability is significantly impaired depends primarily on the extent of the indicated cracking and the duration of time when the cracking tendency is indicated.

Potential for Sliding Failures: Except for the cases of sliding associated with the partial notch-shaped failures described above, or for sliding failures developing within or along rock foundations, which are discussed in a separate section, sliding is not a credible failure mode for arch dams. For gravity dams the potential for sliding or shear failures within the dam depends primarily on the extent of cracking that develops during an earthquake. If extensive cracking and a significant reduction in sliding resistance are indicated, sliding stability can be checked using time histories of nodal point forces, which are available as part of the output from most finite element programs presently used to perform time history dynamic analyses. If the particular finite element program used does not provide time histories of forces directly, relatively simple modifications to the program are usually possible to obtain them.

Normally, if cracking through the thickness of the dam is not indicated and if substantial intact concrete remains at cracked locations, the sliding stability will be acceptable (sliding factor of safety greater than 1.0 for extreme loading conditions). If unacceptable factors of safety are calculated at more than a few instants of time, nonlinear analyses incorporating a joint element to represent the sliding plane of interest can be performed to estimate the amount of sliding that will result, as recommended by the current criteria of the U.S. Army Corps of Engineers and the Federal Energy Regulatory Commission (6-9, 6-10). However, at present, analyses incorporating joint elements are limited to two-dimensional cases. If a dam has keyed contraction joints or there are other three-dimensional effects offering restraint to potentially unstable portions of the dam, results from two-dimensional analyses incorporating joint elements may have little practical meaning.

Potential for Overturning Failures: Because of their inherent resistance to overturning and the extremely short duration of dynamic overturning forces during earthquake excitation, overturning failures of well-proportioned concrete dams during earthquakes are not possible. However, appurtenant structures, such as parapet walls and gate support piers, could experience overturning in severe earthquakes, and reinforcement that is designed to resist the expected earthquake input should be provided for such components.

Buttress Dams

No major buttress dam has been constructed in the United States since about the mid-1970s. Given the economics of roller-compacted concrete construction, it is unlikely that any significant buttress dams will be constructed in the future. However, existing buttress dams have required, and undoubtedly will continue to require, seismic evaluations.

Certain types of buttress dams are more susceptible to damage from cross-channel earthquake ground motion than are arch or gravity dams. However, the concrete comprising buttress dams is usually well reinforced, unlike the unreinforced mass concrete used to construct arch and gravity dams. Because of this reinforcement and because so few buttress dams have been constructed during the past 20 to 30 years, there are no updated criteria intended specifically for such dams that are comparable to available criteria for arch and gravity dams. Consequently, to evaluate the seismic performance of concrete buttress dams based on results from numerical analyses, it is recommended that the most recent criteria developed by the American Concrete Institute (6-11) be used, as set forth in code requirements for reinforced concrete structures.

GUIDELINES FOR EVALUATING RESULTS FROM NONLINEAR ANALYSES

No complete nonlinear time history earthquake analysis of a concrete dam, together with its associated reservoir and foundation, has been done to date. However, nonlinear analyses for selected aspects of nonlinear earthquake response of concrete dams have been performed, and research applicable to the nonlinear behavior of concrete and nonlinear analysis techniques continues to be done.

Depending on the type of nonlinearity modeled, the criteria used to relate the results from nonlinear analyses to expected performance may require a different approach than that for linear analyses. If a nonlinear mechanism primarily affecting the distribution of loads is modeled, such as vertical contraction joints, results can be evaluated in much the same manner as results from linear elastic analyses. However, for nonlinear mechanisms providing for inelastic deformations, such as cracks forming potential sliding surfaces, results that include accumulated displacements must be evaluated. For these cases acceptable limits of accumulated displacements must be established for which the dam would still be considered stable and capable of safely resisting the loads acting after the earthquake has ended.

No detailed criteria for use in evaluating the results from nonlinear seismic analyses of concrete dams are known to have been developed. Part of the difficulty in setting forth detailed criteria for nonlinear seismic analyses is that few definitive data exist concerning the development of failure mechanisms in concrete dams. Research to identify all of the credible potential failure mechanisms for concrete dams and to determine how failures could develop during earthquakes is needed. Such research would not only lead to the development of rational criteria for evaluating the results from nonlinear analyses of concrete dams but would also provide a basis for improving criteria used for evaluating results from linear analyses.

GUIDELINES FOR EVALUATING FOUNDATION STABILITY

In addition to failures resulting from exceeding the bearing capacity of the foundation rock, two types of potential foundation instability during seismic events can be evaluated using results from finite element analyses. The first type is potential sliding along the contact between the dam and foundation rock, and the second is potential sliding of rock blocks or wedges within the foundation and in contact with the dam. These potentially unstable blocks or wedges are formed by intersecting planes associated with rock discontinuities, such as faults and rock joints, and have sliding planes that daylight downstream from the dam.

Sliding Stability Along Concrete-Foundation Contacts

Usually, sliding stability along the concrete-foundation contact of a concrete arch dam is not a problem because of the wedging produced by arch action. However, gravity and buttress dams usually do not have the benefit of this wedging action, although they sometimes are designed to be curved in plan to provide increased stability. Additionally, there can be cases where the geometry of the abutment surfaces is not conductive to sliding stability, adequate drainage is not provided along the contact, and the concrete is not thoroughly bonded to the foundation rock. In such cases the results obtained from time history finite element analyses, together with results from static analyses that include the effects of uplift, can be used to calculate factors of safety against sliding along the abutments.

Since loads acting at nodal points can be obtained from finite element analyses on an element-by-element basis, time histories of loads acting on a portion of the foundation surface corresponding to the bottom faces of one or more elements can be readily determined. For most of the locations along the modeled abutments, loading on a particular portion of the foundation surface originates from elements in the modeled dam whose bottom faces are in direct contact with the foundation. Depending on how the dam is modeled, one or more elements in the dam may have an edge in contact with the foundation as well. In addition to these edge forces, nodal forces corresponding to any applied loads at the contact must be added to the element nodal loads to satisfy equilibrium of forces at the node points.

In the case of an arch dam or a gravity dam constructed with keyed contraction joints, instability at a particular location will cause the transfer of excess driving forces acting on the unstable portion to adjacent portions, provided the adjacent portions have sufficient reserve load-carrying capability. Therefore, even if one or more local instabilities exist, the dam could remain stable. While such load transfers may be acceptable for infrequent, extreme loading conditions, such as earthquakes, they should generally be considered unacceptable for design loading conditions.

Sliding Stability Within Foundations

Potential foundation instabilities formed by intersecting rock discontinuities (joints, shear zones, bedding planes, etc.) can also be evaluated using results obtained from time history finite element analyses combined with results from static analyses (6-12). Typically, two modes of instability can be considered: a block or wedge of rock formed by rock discontinuities underneath the dam sliding along the surface of one plane of the discontinuity, and a block or wedge of rock underneath the dam sliding along the line of intersection of two of these discontinuities. In order for sliding to occur, the direction of sliding must intersect a free surface downstream from the dam; sliding instability is not likely if the direction of sliding is into the dam itself.

As for the evaluation of sliding stability along the foundation contact, if all of the calculated factors of safety are greater than or equal to an acceptable value, sliding stability can be considered adequate. If some of the calculated factors of safety are less than an acceptable value, adequate stability may still exist. However, the adjacent portions of the dam must be capable of bridging the unstable foundation block and transferring excess driving forces to adjacent portions of the foundation. Depending on the number of time intervals when instability is indicated by a linear time history analysis and the extent of the instability, a time history sliding-block analysis may be required to fully evaluate stability. While load transfers involving significant blocks within the foundation may be acceptable for infrequent, extreme loading conditions, they should generally be considered unacceptable for design loading conditions.

STABILITY FOLLOWING FAULT DISPLACEMENT

Generally concrete dams have not been sited at locations where there were faults classified as active. However, as described in Chapter 2, two gravity dams (Morris in California and Clyde in New Zealand) have been constructed across faults in the foundation rock, using sliding joints in the structure to accommodate possible fault movement. It is worth noting that the fault under Morris Dam has since been classified as inactive.

In the future there may be sites considered suitable for concrete dams where there is some possibility of fault displacements occurring underneath the dam. In addition, there may be situations where an existing dam was constructed over a fault thought to be inactive and later determined to be active. For these cases nonlinear analyses and criteria will be required to assess the capability of concrete dams to safely withstand fault displacements. Only one such analysis is known to have been performed (2-61). Although portions of the analysis were conducted using a nonlinear finite element model, the analysis was greatly simplified.

Research is needed to develop criteria that can be used to evaluate the results from numerical analyses simulating fault displacements. With sufficient research, defensive design measures could be identified that would allow concrete dams to safely withstand fault displacements. Defensive measures have been satisfactorily incorporated into the design of embankment dams and have been accepted as providing adequate protection against failure. Conceptually, defensive measures for concrete dams also could be developed that should be just as acceptable and safe.

EVALUATION OF CRITERIA

Present Criteria

Most of the seismic evaluations of concrete dams performed in the United States are based on criteria set forth by the Bureau of Reclamation (6-1), the U.S. Army Corps of Engineers (6-9), or the Federal Energy Regulatory Commission (6-10). The concepts discussed in this chapter are generally consistent with these criteria. However, there are portions of the presently used criteria that should be reviewed and possibly revised.

For example, past research and observations of prototype behavior have clearly shown that pseudostatic stability analyses using simple seismic coefficients will not realistically predict the response of concrete gravity dams to strong earthquakes. Yet some of the criteria presently in use continue to allow or even require pseudostatic analyses for the evaluation of the seismic response of gravity dams, even though simple methods that provide more reliable results are available (3-29, 3-30). Except for gravity dams that are extraordinarily stiff and less than about 100 ft in height, the results from pseudostatic analyses are not likely to be comparable to expected prototype responses to earthquakes. Because the results from these analyses are rarely meaningful, pseudostatic analyses using seismic coefficients should be discontinued, even for preliminary screening studies. For preliminary studies, starting with an earthquake design spectrum, a rational simplified analysis (3-29, 3-30) can be performed manually or implemented on a small computer.

Another aspect of some of the criteria presently being used that needs further review is the treatment of uplift pressures. Several methods of accounting for the effects of uplift have been proposed and are presently in use. However, there are some significant differences between the various approaches, and these differences should be resolved. Comprehensive uplift measurements from actual dams, similar to those assembled by the Bureau of Reclamation (6-13), could be used to help resolve these differences.

Similarly, various criteria contain differing requirements concerning levels of tensile stress considered acceptable, particularly along the interface between the dam and its foundation. For instance, some criteria require that the interface between foundation rock and concrete in gravity dams always be assumed to have zero tensile strength. Although there are obviously cases where zero tensile strength across the interface should be assumed, this assumption is not appropriate in all cases. Most well-designed, well-constructed concrete gravity dams analyzed two-dimensionally for their response to significant earthquake excitation will exhibit the tendency to develop tensile stresses near and along the foundation contact. If the dam is well proportioned, if an appropriate amount of foundation preparation has been performed, and if appropriate construction techniques have been employed, sufficient tensile capacity across the interface may exist to resist these tensile stresses. The possibility that a discontinuity exists in the rock foundation within the first few feet below a dam may be itself be insufficient justification for requiring an assumption of zero tensile strength across the interface for two reasons. First, a thorough geologic investigation of a site, even for existing dams, may provide evidence that continuous rock discontinuities that could be potentially troublesome do not exist. Second, even if such a discontinuity did exist, its effect on the stability and seismic performance of a concrete gravity dam may be quite different from the effect of an unbonded foundation interface.

Deformation-Based Criteria

As discussed in the preceding sections, presently accepted criteria used in the United States to evaluate the seismic performance of concrete dams are based on comparing levels of computed stresses with concrete strengths. This is a longstanding practice that is not likely to change in the near future. However, as discussed in the section titled Creep Effects and Concrete Strengths During Earthquake Loading in this chapter, the behavior of mass concrete is highly strain-dependent. In fact, its behavior is more strain-or deformation-dependent than stress-or load-dependent. In addition, the primary unknowns calculated from finite element analyses are deformations and strains. Calculated stresses are secondary variables, the computation of which often involves extrapolation of strains and can result in some inconsistencies.

Although the use of stress-based criteria is appropriate for the linear elastic analyses commonly performed at present to assess the seismic performance of concrete dams, it may be inappropriate to use such criteria to evaluate the results from analyses using nonlinear techniques that will continue to be developed. Since the behavior of mass concrete is more strain-dependent than stress-dependent, strain-based criteria will likely be necessary in the future to adequately evaluate results from nonlinear analyses. Evaluation of results from linear analyses could also be improved through application of suitable strain-based criteria. Consequently, as additional research is performed concerning the behavior of mass concrete, researchers should consciously begin to form the basis for developing deformation/strain-based criteria suitable for evaluating the performance of mass concrete in dams.

Probability-Based Criteria

It is evident from the discussions in the preceding sections that the criteria presently used to evaluate the seismic performance of concrete dams are deterministic. Since characterizing the occurrence of earthquakes is most meaningful in terms of probabilities of occurrence, use of probability-based criteria to evaluate the seismic performance of concrete dams is an appropriate approach. At present the uncertainties (expressed in probabilistic terms) of site characteristics, material flaws, construction flaws, and other factors influencing the seismic performance of dams are too significant to provide for meaningful probabilistic evaluations. However, some work of this type has been done, and conceptual principles have been considered (6-8). It is expected that such work will continue and that probabilistic safety evaluations will become more common in the future.

RESEARCH NEEDS

1. Criteria and Guidelines

Guidelines similar to those described in this chapter have been used for evaluating the seismic performance of concrete dams for at least the past 10 years. However, the criteria used remain relatively crude in comparison with the complexity of dam-reservoir-foundation systems. To address this disparity some investigators have chosen to attempt more complex analytical solutions, but without improvements in the ability to relate numerical results to prototype behavior; more complex analytical solutions remain outside the realm of application. In fact, analytical techniques may have already advanced beyond our present ability to develop input data consistent with the detail, preciseness, and complexity of those solutions.

Before the reliability of evaluating the seismic performance and safety of concrete dams can be significantly increased, the criteria used must be improved. Criteria presently in use are considered adequate only in the context of the present limited understanding of seismic excitation, material properties of dam-reservoir-foundation systems, and resulting system response. Consequently, significant enhancements in criteria cannot be devised without preceding improvements occurring in the understanding of prototype seismic response of dam-reservoir-foundation systems. To achieve better understanding of the seismic response of concrete dam systems and develop subsequent improvements in criteria, more complete data concerning prototype seismic response of concrete dams are needed. The data needed include:

—significant levels of ground acceleration at various locations along the abutments of concrete dams and at locations upstream and downstream from the dams,
—hydrodynamic pressures at various locations along the upstream face of concrete dams,
—response accelerations at various locations along the dams,
—joint openings of contraction joints near the upstream and downstream faces in the upper central portions of concrete dams, and
—dynamic uplift pressures, particularly along the foundation-dam interface of concrete gravity dams.

2. Continued Research

In addition to collecting and evaluating data from prototype behavior, much can be gained by continuing research at suitably equipped research centers. Therefore, the following research is also recommended:

—further characterization of the creep behavior of the mass concrete used to construct dams, together with analytical improvements in accounting for creep in numerical analyses,
—studies of stress-strain relationships during rapid multiaxial loading conditions, especially including tension,
—identification of failure mechanisms for concrete dams and their foundations during earthquakes and delineation of the conditions that would cause various failure mechanisms to develop, and
—development of defensive design measures to safely accommodate fault displacements.

3. Creep Behavior and Stress-Strain Relationships of Mass Concrete

Conducting research in the above areas not only would allow further improvements in analytical techniques but would also provide for improvements in criteria relating analytical results to acceptable prototype behavior. Research to further characterize creep behavior and stress-strain relationships of mass concrete during multiaxial loading conditions would be of particular value for developing deformation or strain-based criteria. Such criteria should be a priority for research and development because it offers the potential for more realistic evaluation of mass concrete behavior under a variety of loading conditions, especially those including seismic events.

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