Earthquake analysis, design, and safety evaluation of concrete dams

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ABSTRACT: Summarized is the current state of knowledge about earthquake response analysis of concrete dams, and the application of this information to the earthquake-resistant design of new dams and to the seismic safety evaluation of existing dams. The limitations of the traditional design procedures and the standard finite-element method are identified, the factors that should be considered in dynamic analysis are discussed, and procedures for simplified response spectrum analysis and refined response history analysis are summarized. The application of these linear analysis procedures to seismic design and safety evaluation of dams is discussed, followed by the limitations of the presently available nonlinear analysis procedures. At the end of the paper, several areas of research are identified in order to improve present methods for the seismic analysis, design, and safety evaluation of concrete dams.

1 INTRODUCTION

The consequences of a large dam failing can be disastrous, so the seismic design of dams is an important part of earthquake engineering. Although no concrete dams have failed because of earthquakes, it is important to recognize that these structures have not been seriously tested--in the sense that very rarely has a large earthquake occurred close to a major concrete dam with a full reservoir. However, earthquakes with magnitudes of approximately 6.5 did occur close to Koyna Dam--a large concrete gravity dam--in India in 1967 and Hsinfengkiang Dam--a strengthened concrete buttress dam--in the People's Republic of China in 1962. Both dams were overstressed by the earthquake motions and both were damaged to an alarming degree. Pacoima Dam, a concrete arch structure, sustained damage to one abutment during the 1971 San Fernando earthquake; its reservoir was only partly full at the time. The experience with the earthquake performance of these dams indicates that concrete dams are not immune to earthquake damage as had commonly been presumed. Thus, it is essential that increasing attention be given to the earthquake safety of these structures.

The ability to evaluate the effects of earthquake ground motion on concrete dams is essential to assess the safety of existing dams, to determine the adequacy of modifications planned to improve old dams, and to evaluate proposed designs for new dams to be constructed. The prediction of the performance of concrete dams during earthquakes is one of the more complex and challenging problems in structural dynamics. The following factors contribute to this complexity:

- Dams and reservoirs are of complicated shapes, as dictated by the natural topography of the site.
- The response of dams may be influenced significantly by variations in the intensity and characteristics of the ground motion over the width and height of the canyon. However, for lack of appropriate instrumental records, the spatial variations of the ground motion cannot be defined with confidence at this time.
- The response of a dam is influenced, generally to a significant degree, by the earthquake-induced motion of the impounded water; by the deformability of the foundation rock; and by the interaction of the motions of the water, foundation rock, and the dam itself.

- During intense earthquake motions, vertical construction joints may slip or open; concrete may crack; and the stored water may locally separate from the upstream face of the dam, resulting in cavitation. These phenomena are nonlinear and extremely difficult to model and account for reliably.

Realistic analyses of the seismic response of dams were not possible until the development of the finite element method, recent advances in dynamic analysis procedures, and the availability of large-capacity, high-speed computers. Thus, much of the research did not start until the mid-1960's. Initially, all nonlinear effects, including those associated with construction-joint opening, concrete cracking and water cavitation, were ignored, and the interaction effects of the impounded water and foundation rock were either neglected or grossly simplified. Subsequently, special techniques were developed for incorporating the interaction effects in linear analyses. These refined analysis procedures have been implemented to varying degrees for the different types of dams.

The greatest success has been achieved for gravity dams, primarily because they are generally amenable to two-dimensional analysis. The reliability of the resulting analytical procedures has been established by demonstrating that they predict results generally consistent with the limited field observations--e.g. the earthquake damage experienced by the Koyna Dam and the responses measured during forced-vibration tests on a few dams. Parametric response studies have also made it possible to demonstrate the principal effects of the interaction of impounded water and foundation rock on the response of gravity dams.

The analysis and design of arch dams have not achieved the same degree of progress primarily because these must be treated as three-dimensional systems. Substantial progress has been made recently in developing rational procedures for evaluating the hydrodynamic effects and for including the interaction effects of the impounded water. However, reliable techniques for considering the effects of interaction between the dam and the supporting foundation rock remain elusive. Contributing to the difficulty is the fact that the spatial variation of the ground motion along the dam boundary cannot be defined reliably.

Very little research has been directed specifically to the earthquake response of buttress dams, perhaps because this type of dam is used infrequently compared to gravity or arch dams. As stated in the Bureau of Reclamation publication (1974) Design of Small Dams:

> "Buttress dams comprise flat deck and multiple arch structures. They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel required usually offset the savings in concrete. A number of buttress dams were built in the 1930's when the ratio of labor cost to materials cost was comparatively low. This type of construction usually is not competitive with other types of dams when labor costs are high."

However, occasionally there is interest in the earthquake analysis of such structures if the seismic safety of an old buttress dam needs to be evaluated. The analysis technique and computer program EACD-3D developed for three-dimensional earthquake response analysis of concrete dams, which is described in this paper, can be applicable to buttress dams. However, for lack of comprehensive parametric response studies, little is known about the significance of various factors, such as dam-water interaction and dam-foundation rock interaction, in the earthquake response of buttress dams. Consequently, the earthquake response of this type of dam is not included in this paper.

The objective of this paper is to summarize the current state of knowledge about earthquake response analysis of concrete dams and how this information can be applied to the earthquake-resistant design of new dams and to the seismic safety evaluation of existing dams. The paper is divided into four parts: Parts I and II are devoted to gravity dams and arch dams, respectively. In each part, for the particular dam type, the limitations of the traditional design procedures and the standard finite-element method are identified, the factors that should be considered in dynamic analysis are discussed, and procedures for simplified response spectrum analysis and refined response history analysis are summarized. The purpose of Part III is two-fold: (1) to present the application of simplified and refined linear analysis procedures in the earthquake-resistant design of new dams and in the seismic safety evaluation of existing dams; and (2) to identify some of the limitations of presently available nonlinear analysis procedures in predicting the extent of cracking and damage that a concrete dam may experience during very intense ground shaking. In Part IV several areas of research are mentioned in order to improve present methods for the seismic analysis, design, and safety evaluation of concrete dams.

The problem of earthquake response analysis of concrete dams has been the subject of numerous research investigations in the past twenty years. However, no attempt is made to establish the interrelationship of the material presented in this paper to the work of other researchers. This paper is based almost exclusively on the results of studies carried out at the University of California at Berkeley during the past ten years.

PART I: GRAVITY DAMS

2 EVALUATION OF TRADITIONAL ANALYSIS AND DESIGN PROCEDURES

2.1 Traditional analysis and design procedures

Although new design criteria are now available (U.S. Bureau of Reclamation 1974 and 1976), traditionally, concrete gravity dams have been designed and analyzed by very simple procedures (U.S. Army Corps of Engineers 1958 and U.S. Bureau of Reclamation 1966). The earthquake forces are treated simply as static forces and are combined with the hydrostatic pressures and gravity loads. The analysis is concerned with overturning and sliding stability of the monolith treated as a rigid body and with stresses in the monolith which are calculated by elementary beam theory.

In representing the effects of horizontal ground motion--transverse to the axis of the dam--by static lateral forces, neither the dynamic response characteristics of the dam-water-foundation rock system nor the characteristics of earthquake ground motion are recognized. Two types of static lateral forces are included. Forces associated with the weight of the dam are expressed as the product of a seismic coefficient--which is typically constant over the height with a value between 0.05 to 0.10--and the weight of the portion being considered. Water pressures, in addition to the hydrostatic pressure, are specified in terms of the seismic coefficient and a pressure coefficient which is based on assumptions of a rigid dam and incompressible water. Finally, interaction between the dam and the foundation rock is not considered in computing the aforementioned earthquake forces.

The traditional design criteria require that an ample factor of safety be provided against overturning, sliding and overstressing under all loading conditions; compressive stresses should be less than the allowable values. Tension is often not permitted; even if it is, the possibility of cracking of concrete is not serious considered. It has generally been believed that stresses are not a controlling factor in the design of concrete gravity dams so that the traditional design procedures are concerned most with satisfying the criteria for overturning and sliding stability.

2.2 Earthquake performance of Koyna Dam

Koyna Dam is one of a few concrete dams that has experienced a destructive earthquake (Chopra and Chakrabarti 1973). Constructed during the years 1954 to 1963, it is a straight gravity structure made of rubble concrete. It is about 2800 feet long and 338 feet high above the deepest foundation. The traditional design procedure with a seismic coefficient of 0.05 was employed in designing the dam. The earthquake of December 11, 1967, with maximum accelerations around 0.5 g caused significant structural damage to the dam, including horizontal cracks on the upstream and downstream faces of a number of nonoverflow monoliths around the elevation at which the slope of the downstream face changes abruptly. The overflow monoliths were not damaged. Although the dam survived the earthquake without any sudden release of water, the cracking appeared serious enough that it was decided to strengthen the dam by providing concrete buttresses on the downstream face of the nonoverflow monoliths.

Assuming linear structural behavior, the dynamic response of the tallest nonoverflow monolith of Koyna Dam to the Koyna ground motion was analyzed. The response results indicated larger tensile stresses in the upper part of the dam, especially around the elevation at which the slope of the downstream face changes abruptly. These stresses, which exceed 600 psi on the upstream face and 900 psi on the downstream face (Figure 1), are

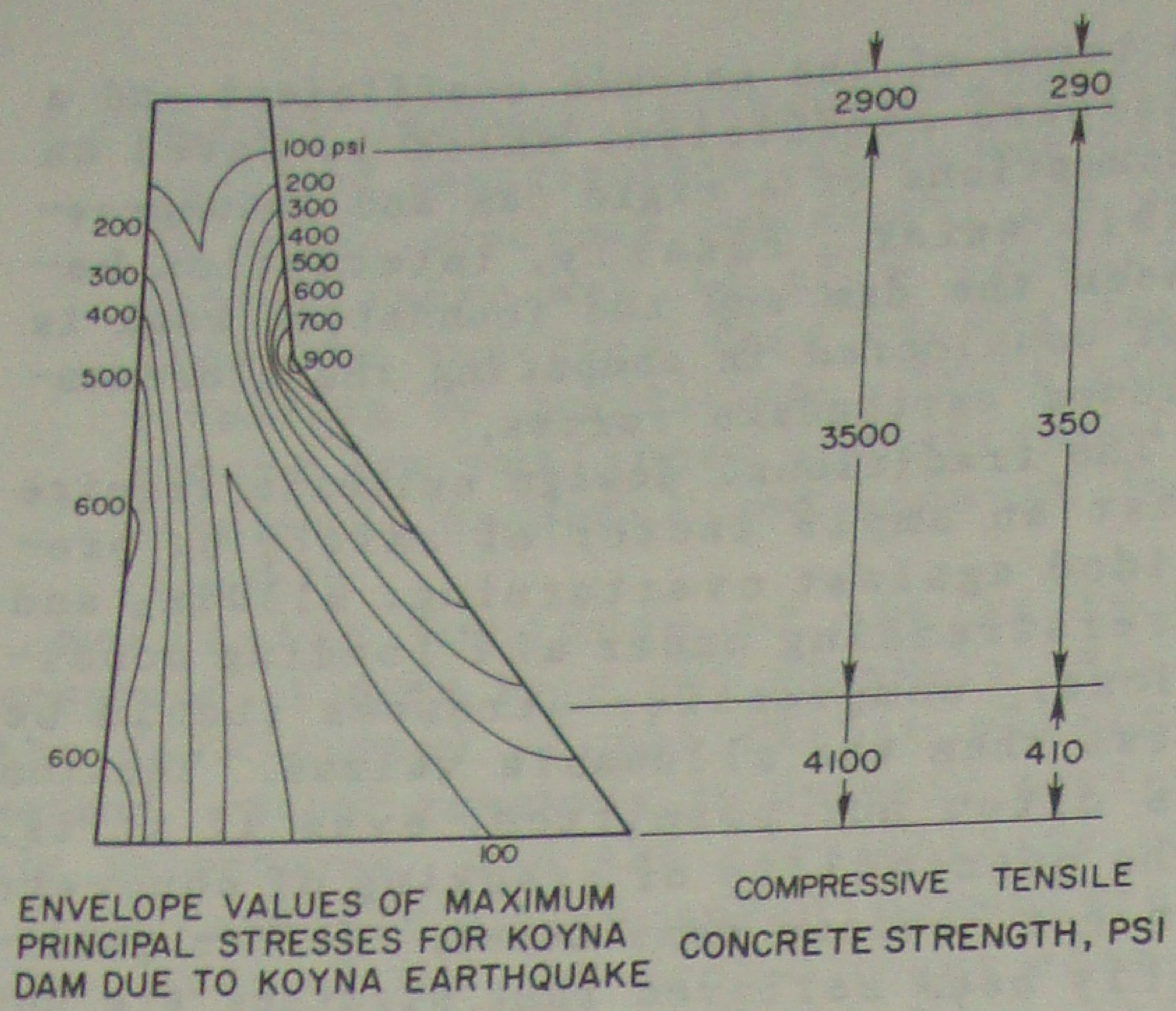


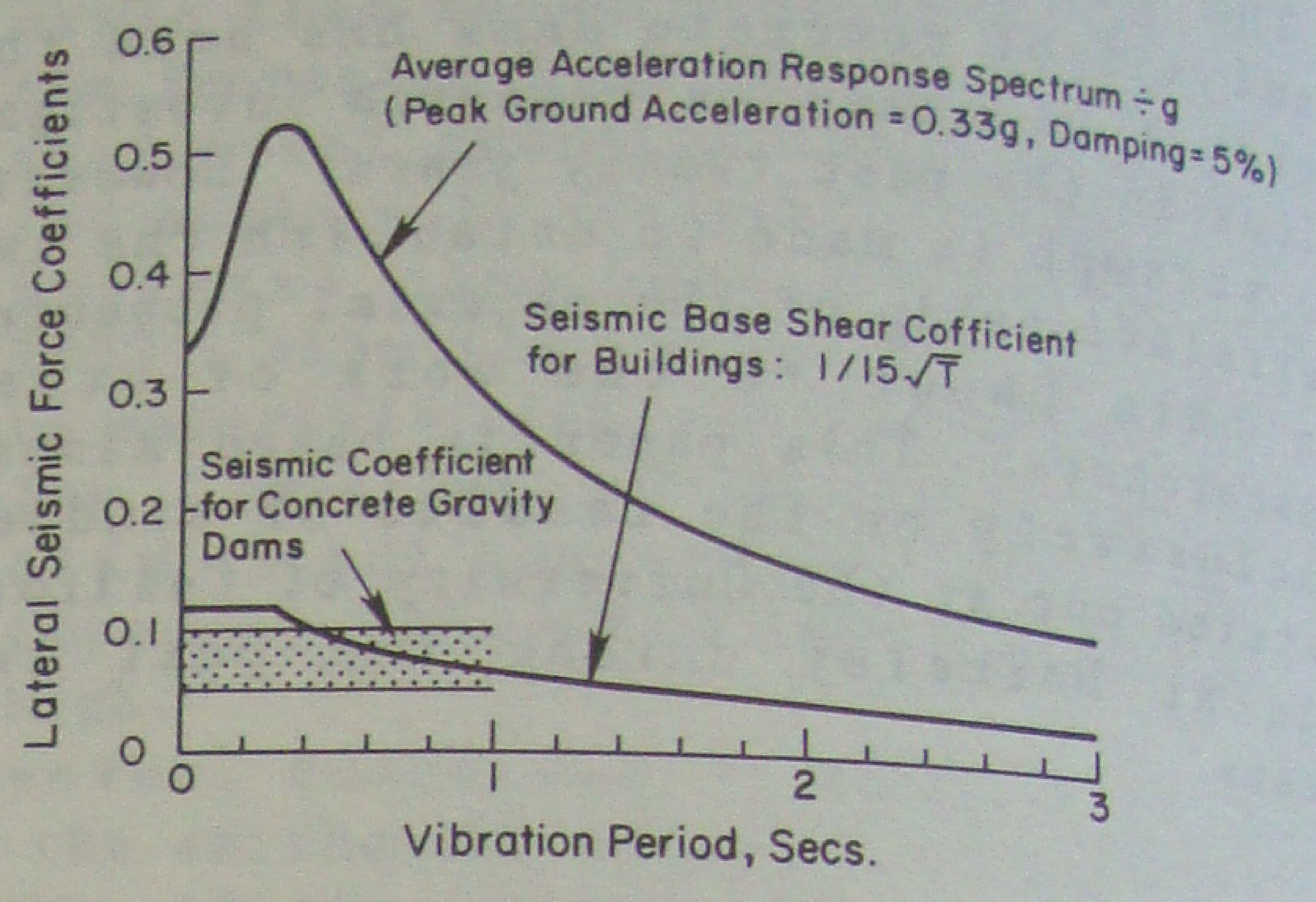
Fig.1 Comparison of stresses in Koyna Dam, predicted by linear analysis, with tensile strength of concrete

approximately two to three times the tensile strength (350 psi) of the concrete used in the upper parts of the dam. Hence, based on the analytical results and strength data, significant cracking can be expected at locations consistent with the damage caused by Koyna earthquake. The maximum compressive stress in the monolith exceeds 1100 psi (not shown in Figure 1), which is well within the compressive strength of concrete. A similar analysis of the overflow monoliths predicted that the earthquake should have caused little or no cracking in these monoliths, which is also consistent with the actual damage.

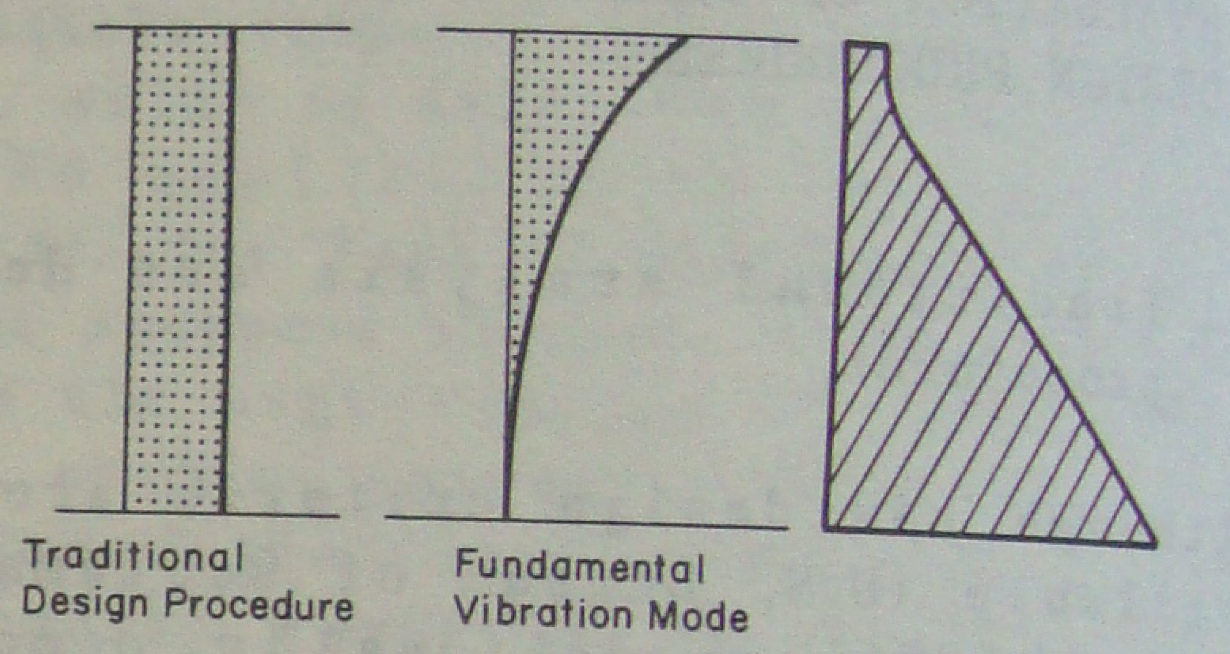
2.3 Limitations of traditional design procedures

It is apparent from the preceding analysis that stresses in gravity dams due to standard design loads have little resemblance to the dynamic response of such dams to earthquake ground motion. In the case of Koyna Dam, the earthquake forces included in the design loads were based on a seismic coefficient of 0.05, uniform over the height. The criterion of no tension was satisfied in designing the dam and obviously no cracking was anticipated. However, the Koyna earthquake caused significant cracking in the dam. This discrepancy is the result of not recognizing the dynamic response of dams to earthquake motions in computing the earthquake forces included in the traditional design methods.

The typically used values, 0.05 to 0.1, for the seismic coefficient are much smaller compared to the ordinates of pseudoacceleration response spectra for intense earthquake motions in the range of vibration periods up to 1 sec (Figure 2a), which is about the largest



(a) Comparison of Earthquake Response Spectrum and Design Coefficients



(b) Distribution of Seismic Coefficients over Dam Height (Lateral Forces = Seismic Coefficients X Weight / Unit Height)

Fig.2 Comparison of traditional design procedures with dynamic effects in earthquake response of concrete gravity dams (Chopra 1978)

possible vibration period for a concrete gravity dam. It is of interest to note in Figure 2a that the seismic base shear coefficient values for dams are similar to those specified for buildings. However, building code design provisions (International Conference of Building Officials 1982) are based on the premise that buildings should be able to:

"1. Resist minor earthquakes without damage; 2. Resist moderate earthquakes without structural damage, but with some nonstructural damage; 3. Resist major earthquakes . . . without collapse but with some structural . . . damage."

Whereas these may be appropriate design

objectives for buildings, major dams should be designed more conservatively and this is reflected in the aforementioned design criteria used in traditional methods for design of dams. What these traditional methods fail to recognize, however, is that in order to achieve these criteria, dams should be designed for the larger seismic coefficients corresponding to pseudoacceleration response spectra for elastic structures (Figure 2a).

The effective forces on a dam due to horizontal ground motion may be expressed as the product of a seismic coefficient, which varies over the height, and the weight of the dam per unit of height. For short vibration-period structures, such as concrete gravity dams, these lateral forces are essentially due to response in the fundamental mode of vibration, and the seismic coefficient varies roughly as shown in Figure 2b. In contrast, traditional analysis and design procedures ignore the vibration properties of the dam and adopt a uniform distribution for the seismic coefficient, resulting in an erroneous distribution of lateral forces and hence of stresses in the dam.

One of the erroneous results of specifying a heightwise-uniform seismic coefficient has been the practice of decreasing the concrete strength with increase in elevation within some dams, e.g. Koyna Dam (Figure 1) and Dworshak Dam. This practice seems to have been motivated by the observation that traditional analyses predict that the stresses are largest near the base of the dam and they decrease at higher elevations. However, as indicated by dynamic analyses (Figure 1) and the location of earthquake-induced cracks in Koyna Dam, the larger stresses actually occur in the upper part of the dam near the upstream and downstream faces. Therefore, these are the regions where the highest strength concrete should be provided if the designer chooses to vary the concrete strength over the dam.

Another undesirable consequence of specifying a heightwise-uniform seismic coefficient is the failure to recognize the detrimental effects of the block of concrete added near the dam crest for reasons other than structural: to provide freeboard above the maximum water level, to resist the impact of floating objects, and to afford a roadway. This added mass has little, if any, adverse effect on the stresses predicted by traditional analyses because of the relatively small values for the seismic coefficient which are not increased near the crest (Figure 2b). However, as indicated by dynamic analyses, this added mass can lead to a dramatic increase in the dynamic stresses--approximately doubling them in the example presented in Figure 3, wherein the critical stresses

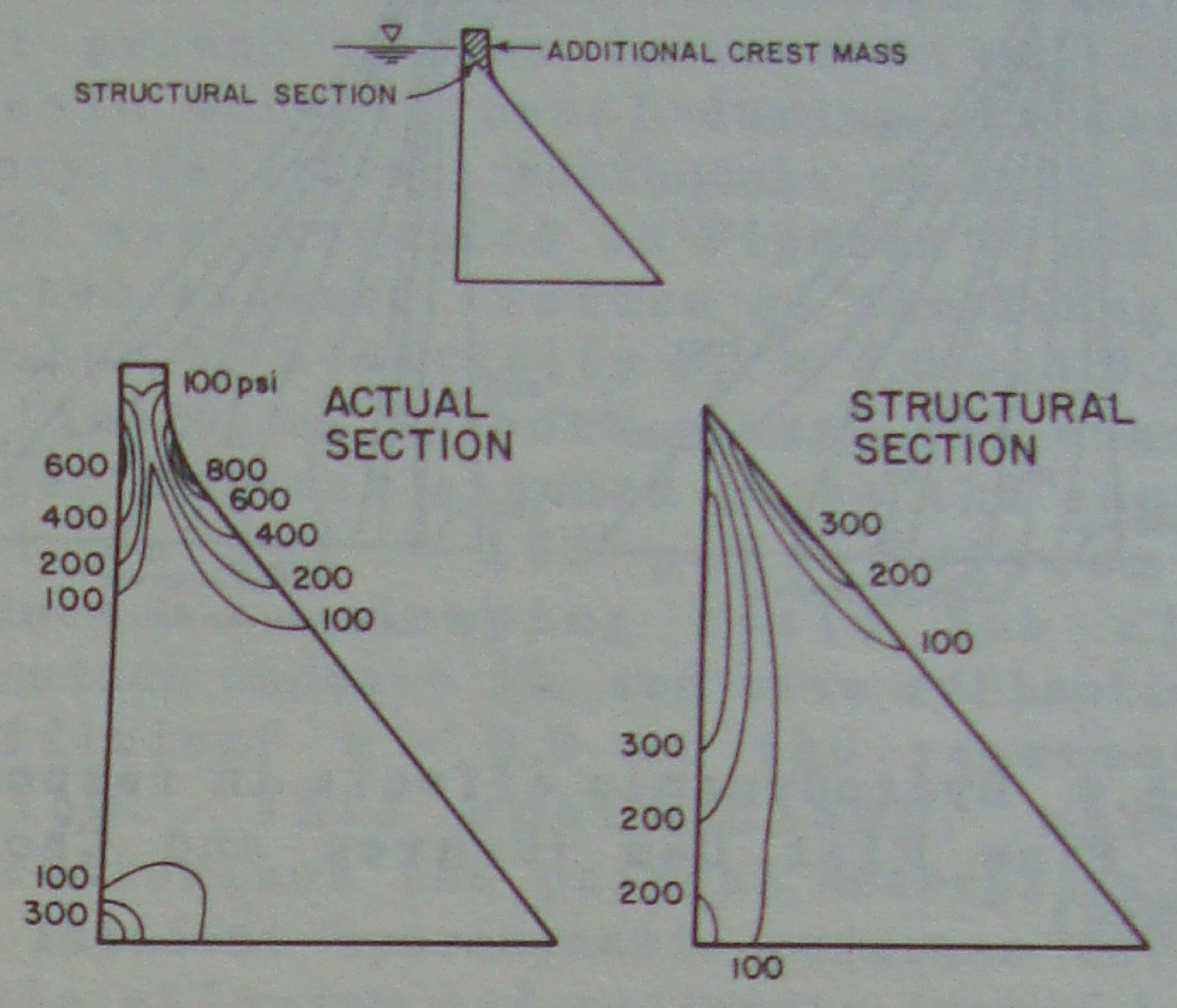


Fig.3 Increased stresses due to additional, nonstructural mass at the dam crest (Chopra and Chakrabarti 1972)

in Pine Flat Dam due to Koyna earthquake are shown.

The traditional design loadings for gravity dams include water pressures in addition to the hydrostatic pressures. A number of formulas, differing somewhat in detail and numerical values but not in the underlying assumptions, are in use (U.S. Army Corps of Engineers 1958 and U.S. Bureau of Reclamation 1966). One of these specifies the additional water pressure $p_e = c\alpha wH$, where c is a coefficient which varies from zero at the water surface to about 0.7 at the reservoir bottom, α is the seismic coefficient, w is the unit weight of water, and H is the total depth of water. For a seismic coefficient of 0.1, the additional water pressure at the base of the dam is 7 percent of the hydrostatic pressure; pressure values at higher elevations are similarly small. These small additional water pressures have little influence on the computed stresses and hence on the geometry of the dam section that satisfies the standard design criteria.

The aforementioned formula for additional water pressures due to earthquake motion is based on analysis treating the dam as rigid and water as incompressible.

When the compressibility of water and dam-water interaction resulting from deformations of the dam are included in the analysis, hydrodynamic effects are generally important in the response of concrete gravity dams. This is apparent from Figure 4 wherein the "envelope"

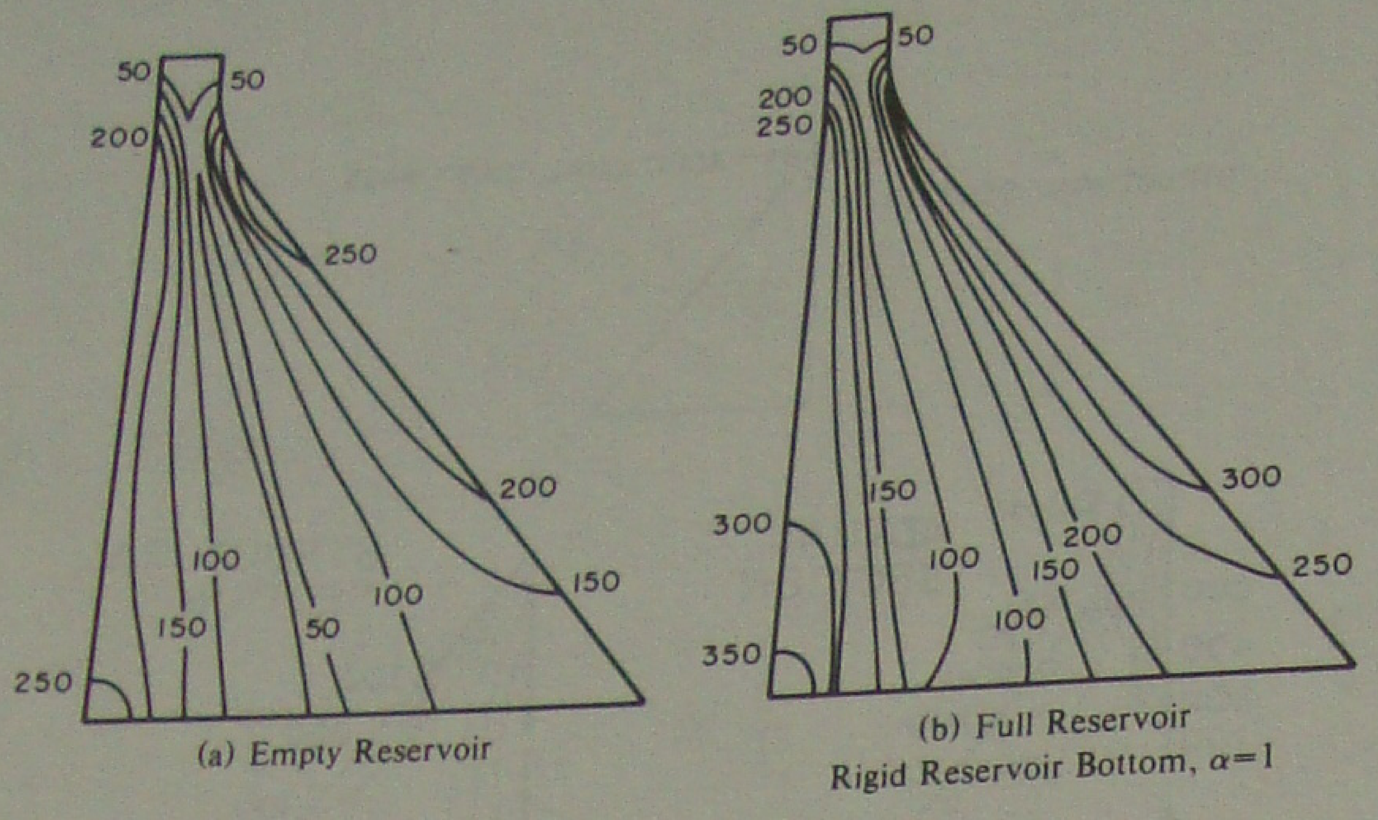


Fig.4 Hydrodynamic effects in response of Pine Flat Dam (Fenves and Chopra 1985a)

values of the maximum principal stresses (in psi) in Pine Flat Dam, due to the upstream components of Taft ground motion, computed for two conditions, are presented; hydrodynamic effects were included in one and neglected in the other. It is apparent that the tensile stresses in the dam are larger by approximately 30% when hydrodynamic effects are included; even larger increases in stresses--about 50%--due to hydrodynamic effects have been noted in other cases. It is obvious, therefore, that the hydrodynamic effects are grossly underestimated in the traditional design loadings.

Foundation rock flexibility is not considered in computing the earthquake forces in traditional design loadings. However, when dam-foundation rock interaction is properly included in the dynamic analysis, these effects are generally significant and they usually reduce the stresses, as seen in Figure 5, wherein are shown the envelope values of maximum principal stresses (in psi) in Pine Flat Dam (with full reservoir and nonabsorptive reservoir bottom) due to upstream component of Taft ground motion.

3 EVALUATION OF THE STANDARD FINITE-ELEMENT METHOD

It is apparent from the preceding section that traditional seismic coefficient

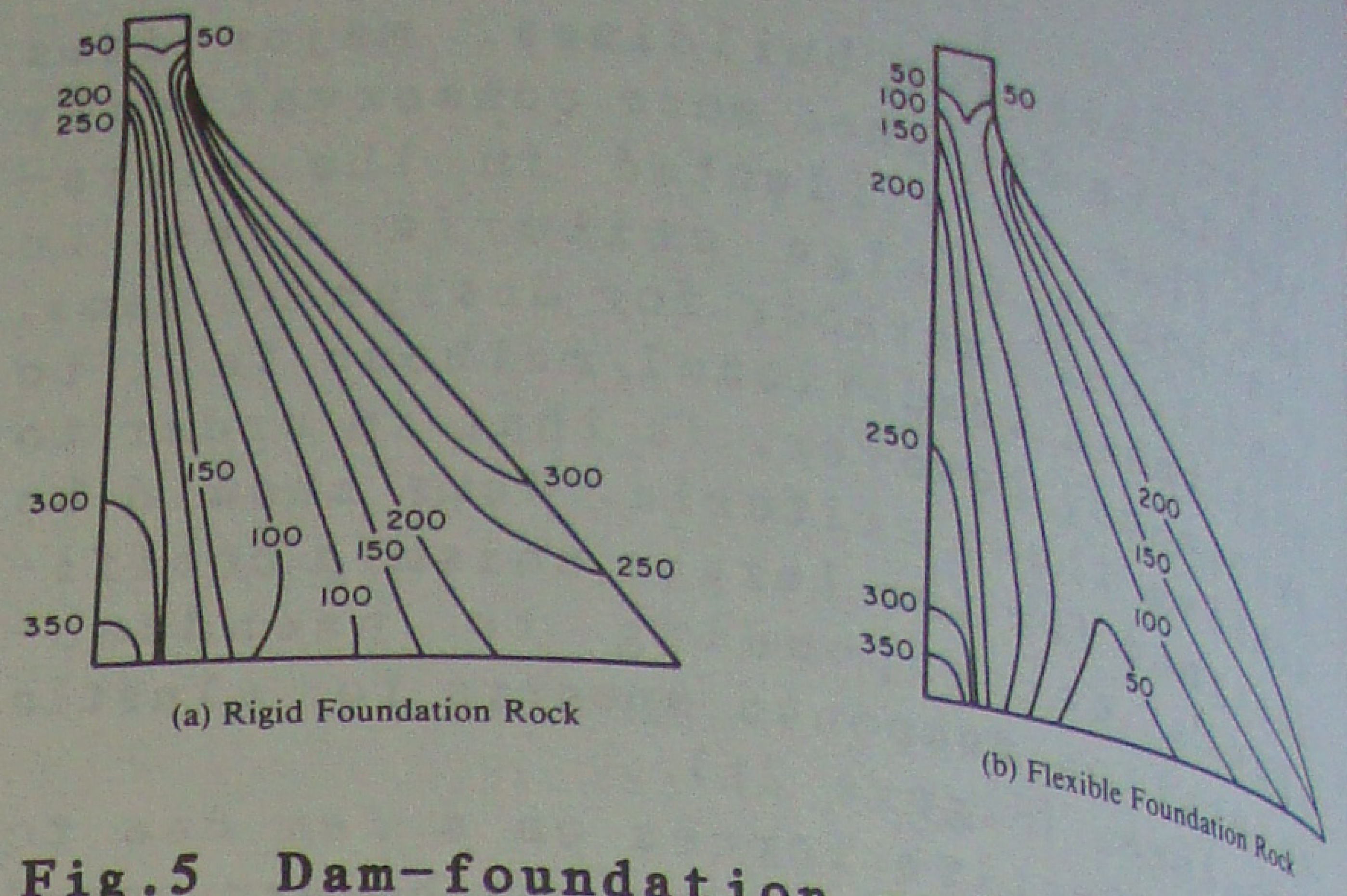


Fig.5 Dam-foundation rock interaction effects in response of Pine Flat Dam (Fenves and Chopra 1985a)

methods must be abandoned in favor of dynamic analysis procedures in order to reliably predict the earthquake response of dams. The procedures for analysis of dams began to change with the development of the finite-element method, advances in dynamic analysis procedures, and availability of large capacity, high speed computers. For example, a dynamic, finite-element analysis procedure, including an added mass representation of hydrodynamic effects, is described in a USBR publication (U.S. Bureau of Reclamation 1976). While such an analysis overcomes some of the afore-mentioned limitations of the traditional procedures, the modelling of dam-foundation rock interaction and of dam-water interaction is usually deficient.

Until a few years ago, the standard approach to accounting for dam-foundation rock interaction was to directly analyze a finite-element idealization of the combined dam-foundation rock system. This is accomplished by including a finite-sized portion of the foundation rock in the system to be analyzed. Such an approach has two drawbacks. Firstly, the boundary hypothesized at some depth to define the foundation rock region included in the analysis is usually assumed to be rigid. For concrete dam sites where similar rocks usually extend to large depths and there is no obvious "rigid" boundary such as soil-rock interface, the location of the rigid boundary introduced in the analysis is often quite arbitrary, resulting in distortion of the foundation interaction effects. Secondly, the earthquake is usually represented in the standard finite-element analysis as motion of the rigid boundary on which the finite-element model is supported. Because very little is known about earthquake motions at great depths, the input motion at the assumed rigid base may be determined by deconvolution of the free-fluid motions specified at the ground surface where most strong-motion accelerograms are recorded. The deconvolution process involves restrictive assumptions on the nature and direction of seismic waves.

The added mass representation of hydrodynamic effects employed in standard finite-element analysis is based on two assumptions that are not satisfied in actuality: that the dam is rigid, and the water incompressible. Although this concept has long been used in practical dam analysis, the range of conditions for which it is valid was not well understood, and during the past two decades extensive research has been devoted to this question. These studies have demonstrated that the computed dam response may be in significant error if the dam-water interaction arising from dam flexibility is not considered (Chopra 1970).

Although studies conducted as early as 1968 and 1970 concluded that water compressibility effects are significant in the response of concrete gravity dams (Chopra 1968 and 1970), there continues to be much interest in research (Hall 1986) and in practical applications (Tarbox et al. 1979) to neglect water compressibility in earthquake analysis of concrete dams, perhaps because such an assumption leads to great simplification in the analysis. In order that such approximate analysis is not applied to situations for which it may not be valid, the significance of water compressibility effects are investigated further and the range of conditions for which these effects may be neglected are identified.

The key parameter that determines the significance of water compressibility in the earthquake response of gravity dams is $\Omega_r = \omega_1^r/\omega_1$ where ω_1^r is the fundamental natural vibration frequency of the impounded water idealized by a fluid domain of constant depth and infinite length and ω_1 is the fundamental natural frequency of the dam alone. It has been demonstrated (Chopra 1968, Fenves and Chopra 1983) that the effects of water compressibility become insignificant in the response of gravity dams to harmonic ground motion if $\Omega_r > 2$. Recognizing that most concrete gravity dams have similar cross-sectional geometry, it has been shown that Ω_r is proportional to $1/\sqrt{E_s}$ where E_s is the Young's modulus for the dam concrete. Thus, the physical implication of the criterion $\Omega_r > 2$ is that the impounded water affects the dam response essentially as an incompressible fluid if the dam is flexible enough. In order to further investigate this question in the context of earthquake response of dams, the response of Pine Flat Dam is examined for two values of the elastic modulus E_s.

The linear responses of the tallest (400-ft high) nonoverflow monolith of Pine Flat Dam on rigid foundation rock to Taft ground motion (recorded at Taft Lincoln School tunnel during the Kern County, California earthquake of July 21, 1952) are presented in Figures 6 and 7 for two assumed values of the Young's modulus of elasticity E_s = 4.0 and 0.65 million psi. In both cases, the other properties are assumed to be the same: unit weight = 155 pcf, Poisson's ratio = 0.2, damping ratio for all vibration modes = 5%, and wave reflection coefficient α = 0.9 at the reservoir bottom. This coefficient, which characterizes the partial absorption of the incident hydrodynamic pressure waves by the alluvium and sediments invariably present at the bottom of a reservoir, is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom.

Hydrodynamic and water compressibility effects in the earthquake response of dams arise partly from the modification of the frequency response functions of the dam (Fenves and Chopra 1983 and 1985b) and partly from the change in the response spectrum ordinates corresponding to the modified resonant periods and damping. The lengthening of the fundamental period due to dam-water interaction and water compressibility is apparent from Figure 6. The maximum crest displacement due to upstream ground motion increases from 0.70 in. to 1.55 in. because of interaction between the dam and water, assumed to be incompressible, for reasons discussed elsewhere in detail. The maximum crest displacement is reduced to 1.09 in. because of water compressibility and reservoir bottom absorption effects. The reductions in the response contributions of the higher vibration modes are especially pronounced because of hydrodynamic radiation damping at the higher excitation frequencies. Similarly, the response to vertical ground motion increases from 0.10 in. to 0.17 in. because of dam-water interaction and further to 1.04 in. because of water

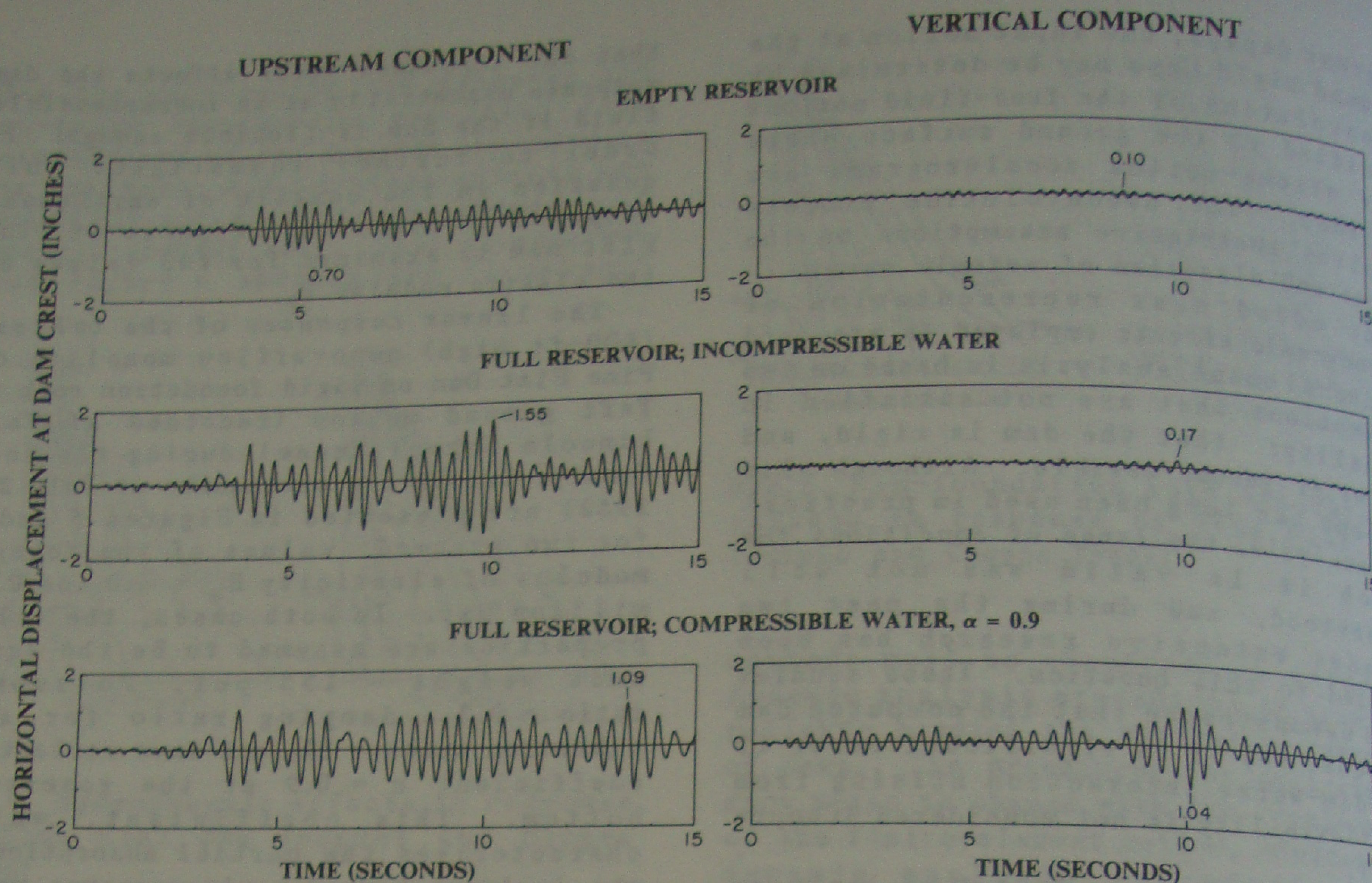


Fig.6 Displacement response of Pine Flat Dam (E_s = 4 million psi) due to upstream and vertical components, separately, of Taft ground motion

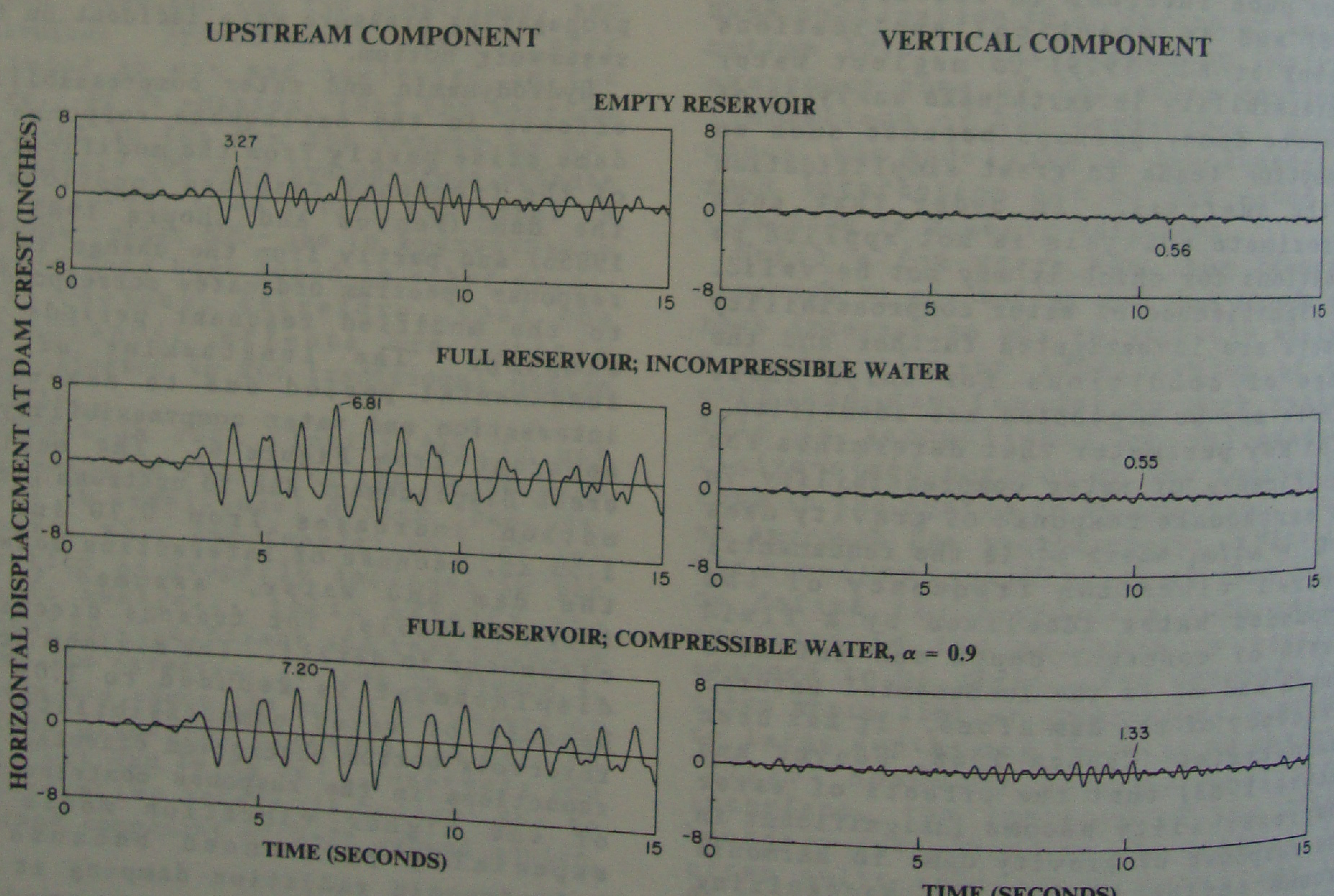


Fig.7 Displacement response of Pine Flat Dam (E_s = 0.65 million psi) due to upstream and vertical components, separately, of Taft ground motion

compressibility. The reasons for the much larger effect of dam-water interaction in the response to vertical ground motion, especially when water compressibility is considered, are documented elsewhere.

As mentioned earlier, the effects of water compressibility become smaller in the earthquake response of a dam as the Young's modulus E_s for the dam concrete decreases. This is demonstrated in Figure 7 wherein the earthquake response of the dam is presented, assuming its elastic modulus to be less than 1 million psi, which is unrealistically small. Water compressibility effects are now seen to be much smaller than in the response of the dam with higher E_s (Figure 6). These effects are insignificant in the dam response to upstream ground motion (compare Figure 7b to 7c). However, even for this very low elastic modulus, water compressibility has strong influence on the dam response to vertical ground motion. Water compressibility would be significant in the response of most gravity dams because E_s is generally much higher--in the range of 2 to 5 million psi--and, for dams with full reservoir, Ω_r would be smaller than 2. Thus, the added mass representation of hydrodynamic effects, which is based on the assumption of incompressible water and is typically used in standard finite-element analysis, would generally lead to erroneous results.

4 REFINED ANALYSIS PROCEDURES AND COMPUTER PROGRAMS

During recent years, extensive research has been devoted to evaluating the significance of hydrodynamic and foundation interaction effects in the earthquake response of concrete gravity dams. These studies have led to several conclusions (Fenves and Chopra 1985a): (1) The earthquake response of dams is increased significantly because of the impounded water, with the magnitude of the hydrodynamic effects being especially large in the response of the dam to vertical ground motion. (2) Neglecting the wave absorptive effects of the reservoir bottom sediments leads to an unrealistically large response of dams, particularly due to vertical ground motion. (3) The assumption of incompressible water commonly employed in practical analysis will generally lead to erroneous results. (4) Neglecting dam-foundation rock interaction arising from foundation rock flexibility will generally lead to an overestimation of dam response. It is apparent from these conclusions that, in order to obtain reliable results, the following factors should be considered in analyzing the earthquake response of dams: dam-water interaction, reservoir bottom absorption, water compressibility, and dam-foundation rock interaction.

Analysis procedures and computer programs have been developed for earthquake analysis of concrete gravity dams idealized as two-dimensional systems (Fenves and Chopra 1984a and 1984b) or treated as three-dimensional systems (Fok et al. 1986). The two-dimensional analysis is recommended whenever it is appropriate for the dam to be analyzed because it is computationally efficient and it rigorously considers all the aforementioned factors known to be significant in the earthquake response of dams, whereas dam-foundation rock interaction and earthquake excitation is usually treated in an overly simplified manner in the three-dimensional analysis.

At small vibration amplitudes a concrete gravity dam will behave as a solid even though the construction joints between the monoliths may slip (Rea et al. 1975). However, during large-amplitude motion, the behavior of a dam depends on the extent to which the inertia forces can be transmitted across the joints. For dams with straight joints, either grouted or ungrouted, the inertia forces that develop during large-amplitude motion are much greater than the shear forces that the joints can transmit. Consequently, the joints would slip and the monoliths vibrate independently, as evidenced by the spalled concrete and water leakage at the joints of the Koyna Dam during the Koyna earthquake of December 11, 1967 (Chopra and Chakrabarti 1973). A two-dimensional, plane stress model of the individual monoliths appears to be appropriate for predicting the response of such dams to moderate or intense earthquake ground motion. On the other hand, for dams with keyed construction joints, it may be inappropriate to assume that the monoliths vibrate independently. For such a dam, a two-dimensional, plane strain model may be better, especially if it is located in a wide canyon.

On the other hand, roller-compacted-concrete gravity dams which are built without transverse joints may be idealized as plane strain systems, a model that is especially appropriate if the dam is located in a wide canyon. However, three-dimensional effects may be significant if the dam is located in a narrow canyon.

Analytical procedures and a computer program have been developed for two-dimensional analysis of a concrete gravity dam supported on the horizontal surface of underlying flexible foundation rock and impounding a reservoir of water (Figure 8). All response results

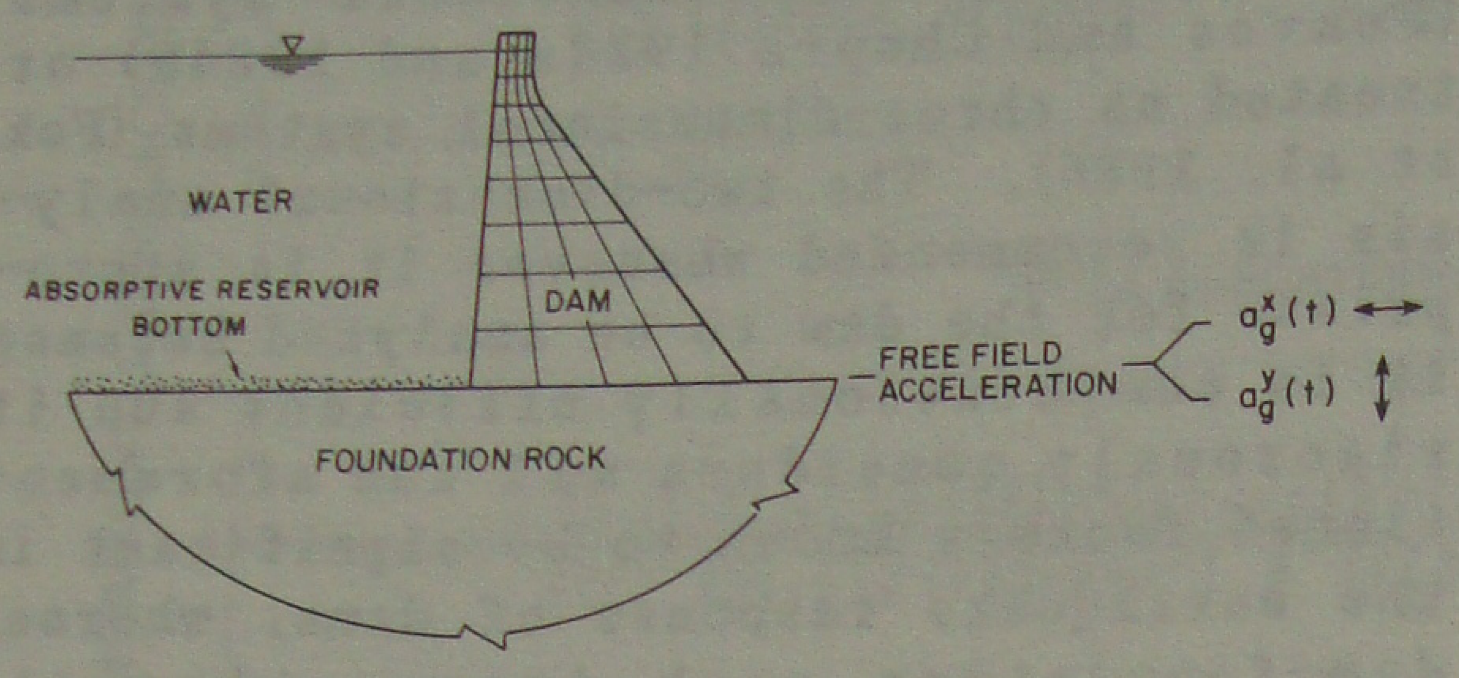


Fig.8 Dam-water-foundation rock system (Fenves and Chopra 1984a)

presented in the preceding section were obtained from this computer program. The selected monolith in generalized plane stress or dam cross-section in plane strain is idealized as a two-dimensional finite-element system. The finite-element idealization makes it possible to model arbitrary geometry and elastic material properties of the dam. Hence, nonoverflow sections, overflow sections, and appurtenant structures can be modelled satisfactorily. However, certain restrictions are imposed on the geometry of the dam to permit a continuum solution for hydrodynamic pressure in the impounded water. For the purpose of determining hydrodynamic effects, and only for this purpose, the upstream face of the dam is assumed to be vertical. This assumption is reasonable for actual concrete gravity dams because their upstream face is vertical or almost vertical for most of the height, and the hydrodynamic pressure acting on the dam face is insensitive to small departures of the face slope from vertical, especially if these departures are near the base of the dam, which is usually the case. The water impounded in the reservoir is idealized as a fluid domain of constant depth and infinite length in the upstream direction. The foundation rock underlying the dam and reservoir bottom materials is idealized as a homogeneous, isotropic, viscoelastic half-plane and treated as a continuum. As mentioned earlier, the semi-infinite extent of the idealized foundation is necessary to properly account for the dam-foundation rock interaction effects, especially the radiation damping.

The viscoelastic half-plane idealization of the foundation-rock region is not appropriate for representing the effects of interaction between the impounded water and the foundation rock. These interaction effects are dominated by the overlying reservoir bottom materials that may consist of variable layers of alluvium, silt and other sediments, possibly deposited to a significant depth, which are highly saturated and have a small shear modulus. A hydrodynamic pressure wave impinging on such materials will partially reflect back into the water and partially refract, primarily as a dilatational wave, into the layers of reservoir bottom materials. Because of the considerable energy dissipation that results from hysteretic behavior and particle turbulence in the layer of saturated materials, the refracted wave is essentially dissipated before reaching the underlying foundation rock. The dissipation of hydrodynamic pressure waves in the reservoir bottom materials is modelled approximately by a boundary condition at the reservoir bottom that partially absorbs incident hydrodynamic pressure waves (Fenves and Chopra 1984a).

Over a long time, sediments may deposit to a significant depth at the bottom of some reservoirs. The thickness of the sediment layer can be recognized by defining the reservoir bottom at the surface of the sediments, which correspondingly reduces the depth of the fluid domain. However, the computer analysis does not consider the influence of the reservoir bottom materials on the static stresses and vibration properties of the dam because these effects should be small, as the materials are very soft, highly saturated and exert forces only on the lower part of the dam.

The earthquake excitation for the dam-water-foundation rock system is defined by the two components of free-field ground acceleration in a cross-sectional plane of the dam: the horizontal component transverse to the dam axis, and the vertical component. The free-field ground acceleration is assumed to be identical at all points on the base of the dam.

A general analytical procedure based on the substructure method has been developed (Fenves and Chopra 1984a) to evaluate the earthquake response of concrete gravity dams, idealized as described earlier, including all the aforementioned factors which are significant in dam response. The analytical procedure has been implemented in the computer program EAGD-84: Earthquake Analysis of Gravity Dams - 1984 (Fenves and Chopra 1984b). The development of an appropriate idealization of the system is discussed, the required input data to the computer program are described, the output is explained, and the response results from a sample analysis are presented. The computer program is available from NISEE, Davis Hall, University of California, Berkeley, California.

This computer program enables the designer to conveniently perform a complete analysis of the dynamic response of the dam to the simultaneous action of the horizontal and vertical ground motion components. The dynamic stresses are combined with the initial, static stresses in the dam due to the weight of the dam and hydrostatic pressures. However, the user may perform a separate static analysis including thermal, creep, construction sequence, and other effects and input the resulting initial stresses in the computer program. The output from the computer program includes the complete time-history of (1) the horizontal and vertical displacements at all nodal points and (2) the three components of the two-dimensional stress state in all the finite elements. From these results, the designer can plot the distribution of stresses in the dam at selected time instants, and the distribution of envelope values of maximum principal stress in the dam, as shown in Figure 1 for Koyna Dam (tension is positive). Such results aid in identifying areas of the dam that may crack during an earthquake. The computer program is therefore a convenient tool in predicting the earthquake performance of designs proposed for new dams and in evaluating the seismic safety of existing dams.

5 SIMPLIFIED ANALYSIS PROCEDURE

While the substructure analysis procedure and the EAGD-84 computer program that implements the procedure are appropriate for analyzing the safety of existing dams against future earthquakes in the final stage of the evaluation process and for new dams in the final stage of the design process, it should be simplified for convenient application in the preliminary evaluation or design stage. In response to this need, a simplified procedure was developed in 1978 in which the maximum response due to the fundamental mode of vibration was represented by equivalent lateral forces, which were computed directly from the earthquake design spectrum without a response history analysis (Chopra 1978). Recently, this simplified analysis of the fundamental mode response has been extended to include the effects of dam-foundation rock interaction and of reservoir bottom materials (Fenves and Chopra 1986), in addition to the effects of dam-water interaction and water compressibility considered in the earlier procedure. Also included now in the simplified procedure are the equivalent lateral forces associated with the higher vibration modes which are computed by a "static correction" method which is based on the assumptions that: (1) the dynamic amplification of the modes is negligible; (2) the interactions among the dam, impounded water, and foundation rock are not significant; and (3) the effects of water compressibility can be neglected. These approximations provide a practical method for including the most important factors that affect the earthquake response of concrete gravity dams.

The simplified analysis procedure is organized in three parts (Fenves and Chopra 1986):

1. Computation of equivalent lateral forces associated with the fundamental vibration mode, considering the aforementioned factors, and evaluation of stresses due to these forces applied at the upstream face of the dam. The stresses are determined by static analysis of the dam using elementary formulas for stresses in beams or the finite-element method.
2. Computation of equivalent lateral forces associated with the higher vibration modes by the "static correction" method, and computation of the resulting static stresses.
3. Combination of the initial, pre-earthquake stresses and the earthquake-induced stresses associated with the fundamental and higher vibration modes.

This response spectrum method is available as a sequence of computational steps, along with the standard data necessary to implement the procedure. The use of the simplified procedure is illustrated by examples and is shown to be sufficiently accurate for the preliminary phase of design and safety evaluation of gravity dams.

PART II: ARCH DAMS

6 EVALUATION OF TRADITIONAL ANALYSIS AND DESIGN PROCEDURES

6.1 Traditional analysis and design procedures

Traditionally, the dynamic response of the system has not been considered in defining the earthquake forces in the design of arch dams. For example, in a 1965 publication of the U.S. Bureau of Reclamation, it is stated:

> "The occurrence of vibratory response of the earthquake, dam and water is not considered, since it is believed to be a remote possibility."

Thus, the forces associated with the inertia of the dam were expressed as the product of a seismic coefficient--which is constant over the surface of the dam with a typical value of 0.10--and the weight of the dam. Water pressures, in addition to the hydrodynamic pressure, are specified in terms of the seismic coefficient and a pressure coefficient which is the same as that defined in Part I of this paper for gravity dams. This pressure coefficient is based on assumptions of rigid dam, incompressible water, and a straight dam. Finally, the effects of foundation rock flexibility are not considered in computing the aforementioned earthquake forces.

The traditional design criteria require that the compressive stress should not exceed one-fourth of the compressive strength or 1000 psi, and the tensile stress should remain below 150 psi.

6.2 Limitations of traditional procedures

As mentioned in Part I of this paper in the context of gravity dams, the seismic coefficient of 0.1 is much smaller than the ordinates of the acceleration response spectra for intense earthquakes (Figure 2). Thus, the earthquake forces for arch dams are grossly underestimated in the traditional analysis procedures.

The effective earthquake forces on a dam due to horizontal ground motion may be expressed as the product of a seismic coefficient, which varies over the surface, and the weight of the dam per unit surface area. The seismic coefficient associated with earthquake forces in the first two modes of vibration of the dam (fundamental symmetric and antisymmetric modes of a symmetric dam) varies as shown in Figure 9. In

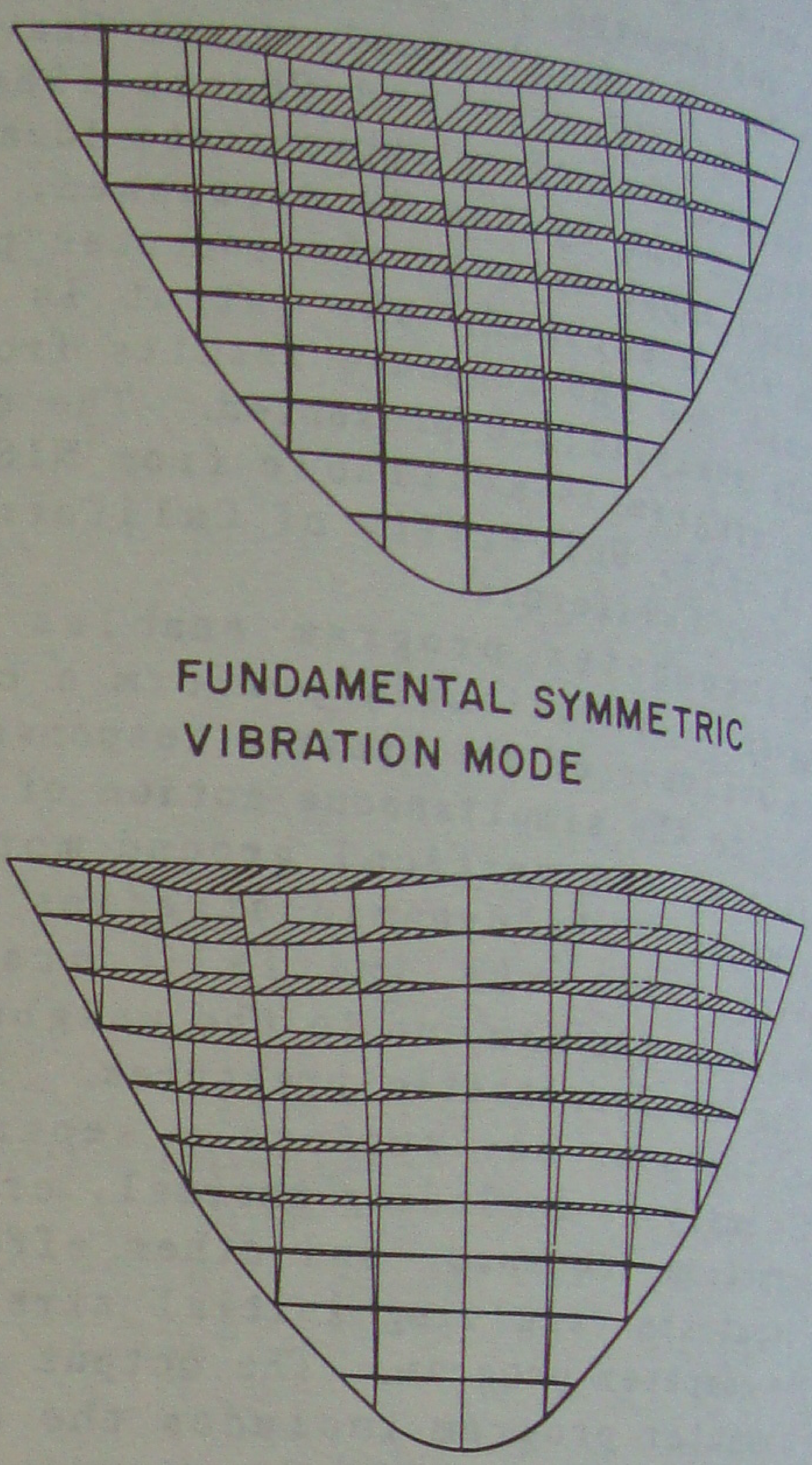


Fig.9 Distribution of seismic coefficients over the dam surface in the first two vibration modes of an arch dam (U.S. Bureau of Reclamation 1977)

contrast, traditional design procedures ignore the vibration properties of the dam and adopt a uniform distribution for the seismic coefficient, resulting in erroneous distribution of lateral forces and hence of stresses in the dam.

As mentioned in Part I of this paper, the additional water pressures included in traditional analysis procedures for gravity dams are unrealistically small and have little influence on the computed stresses and hence on the geometry of the dam that satisfies the design criteria. This observation is equally valid for arch dams because the additional water pressures considered for arch dams are essentially the same as for gravity dams. However, when dam-water interaction and water compressibility effects are properly considered in the analysis, the hydrodynamic effects are generally important in the response of arch dams, more so than for gravity dams. This is apparent from Figure 10 wherein the

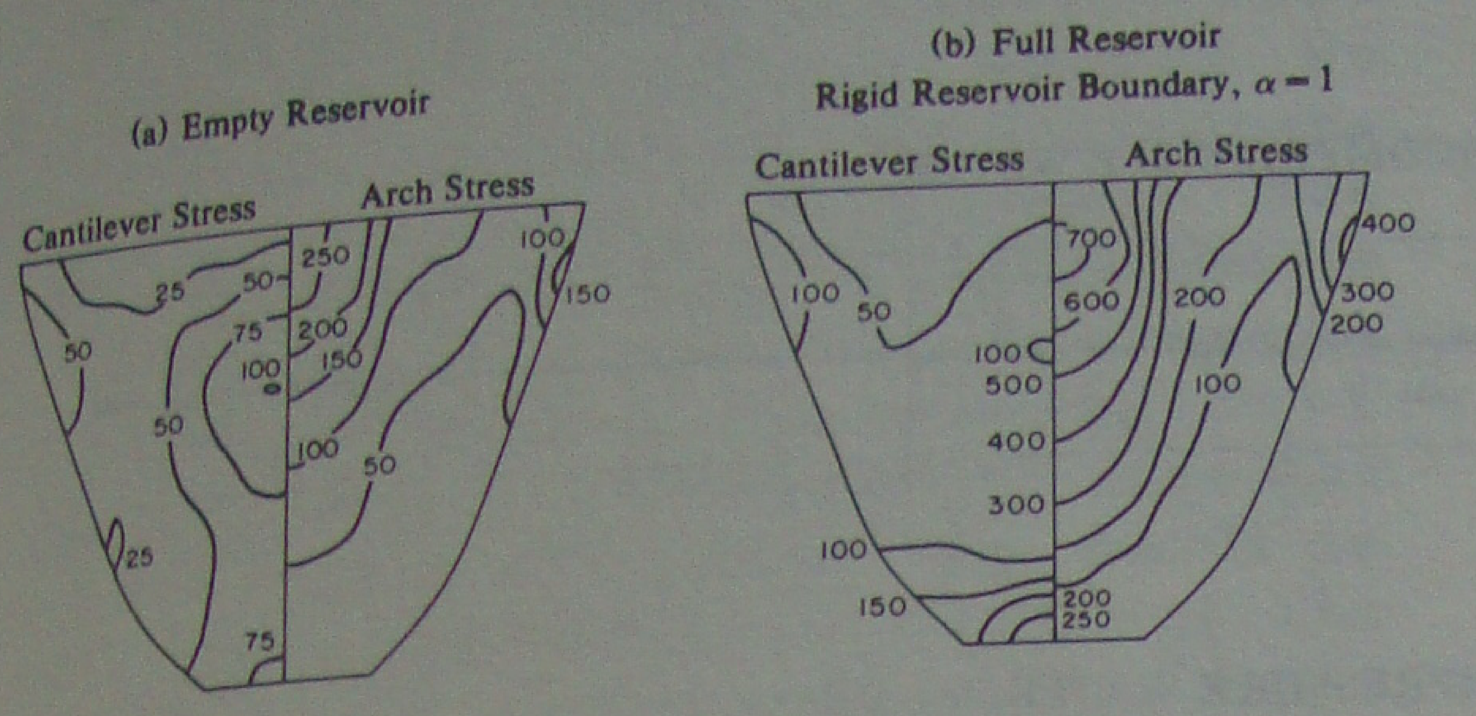


Fig.10 Hydrodynamic effects in response of Morrow Point Dam (Fok and Chopra 1986b)

envelope values of the stresses on the upstream face of Morrow Point Dam due to the upstream component of Taft ground motion are presented for two conditions; hydrodynamic effects were included in one and neglected in the other. It is apparent that the tensile stresses in the dam due to upstream ground motion are more than doubled when hydrodynamic effects are included; even larger increases occur in the stresses caused by the other two components of ground motion. Therefore, it is obvious that the hydrodynamic effects are grossly underestimated in the traditional design loadings.

7 EVALUATION OF THE STANDARD FINITE-ELEMENT METHOD

Recognizing the aforementioned limitations of traditional analysis procedures, dam designers started using dynamic analysis procedures. For example, a dynamic finite-element analysis procedure, including an added mass representation of hydrodynamic effects, is described in Section 4-56 of a 1977 USBR publication. While this procedure overcomes many of the deficiencies of the traditional procedure, it does not properly consider the dam-foundation rock or dam-water interaction effects.

A portion of the foundation rock is usually included in a finite-element analysis of the earthquake response of arch dams. The principal decision required in defining the finite-element idealization is the three-dimensional extent and boundary conditions of the foundation rock to be included in the analysis. For arch dam sites where similar rocks typically extend to considerable distances, wave-transmitting boundaries are necessary if the finite-size foundation rock region is to represent the unbounded extent in the field. Such transmitting boundaries have been developed for two-dimensional analysis (Lysmer and Waas 1972) with seemingly ad hoc extensions proposed for three-dimensional analyses. These transmitting boundaries have not been employed in the analysis of dams and instead an extremely simple idealization for the foundation rock is usually used (Clough 1980). Only the foundation rock flexibility is considered; i.e. the inertial and damping effects of the foundation rock are ignored in considering dam-foundation interaction effects. Since there is no wave propagation mechanism in the massless found ation rock, the design earthquake motion can be specified directly at the base of the finite-element idealization of the dam-foundation rock system.

In the standard finite-element analysis the hydrodynamic effects are approximated by an added mass of water moving with the dam, computed from the analysis of hydrodynamic pressures due to upstream-downstream vibration of a straight rigid dam--which obviously ignores the curvature of the dam--neglecting water compressibility (Westergaard 1933). The resulting added mass is taken to be valid for both "symmetric" and "antisymmetric" vibration modes of the dam, which is obviously inappropriate.

Just as in the case of gravity dams, water compressibility effects are significant in the earthquake response of arch dams with realistic values of E_s, the elastic modulus for concrete, but negligible if E_s is small enough. This is demonstrated in Figures 11 and 12 wherein is presented the earthquake response of Morrow Point Dam, assuming its elastic modulus E_s to be 4.0 million psi--a typical value--and 0.5 million psi--an unrealistically small value--respectively. It is apparent that neglecting water compressibility would be inappropriate in the first case but would be reasonable in the latter case in determining the response of the dam to upstream or cross-stream ground motions. However, even for this unrealistically small E_s value, water compressibility has a significant influence on the response of the dam to vertical ground motion.

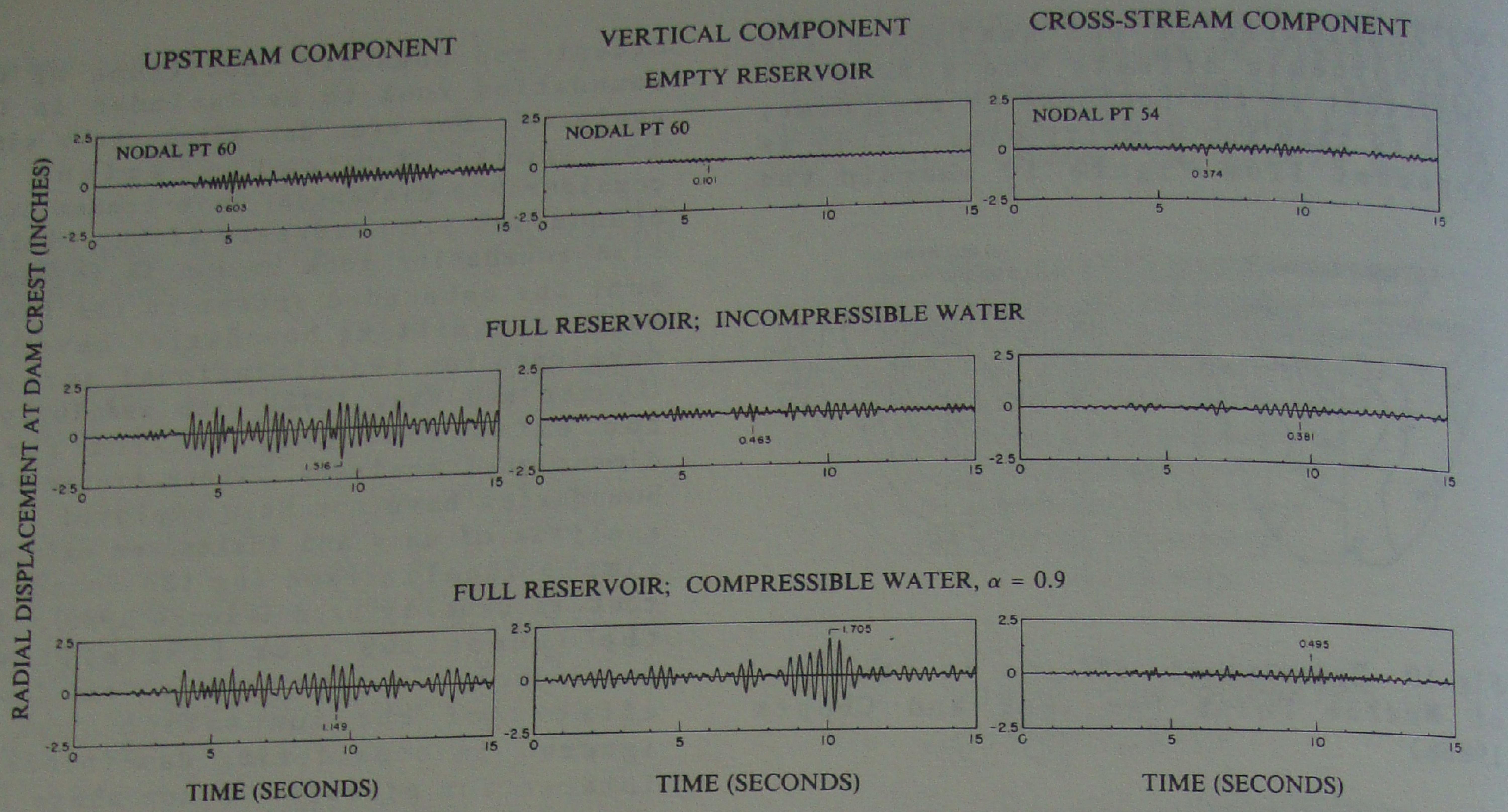


Fig.11 Displacement response of Morrow Point Dam (E_s = 4 million psi) due to upstream, vertical and cross-stream components, separately, of Taft ground motion (Fok and Chopra 1987)

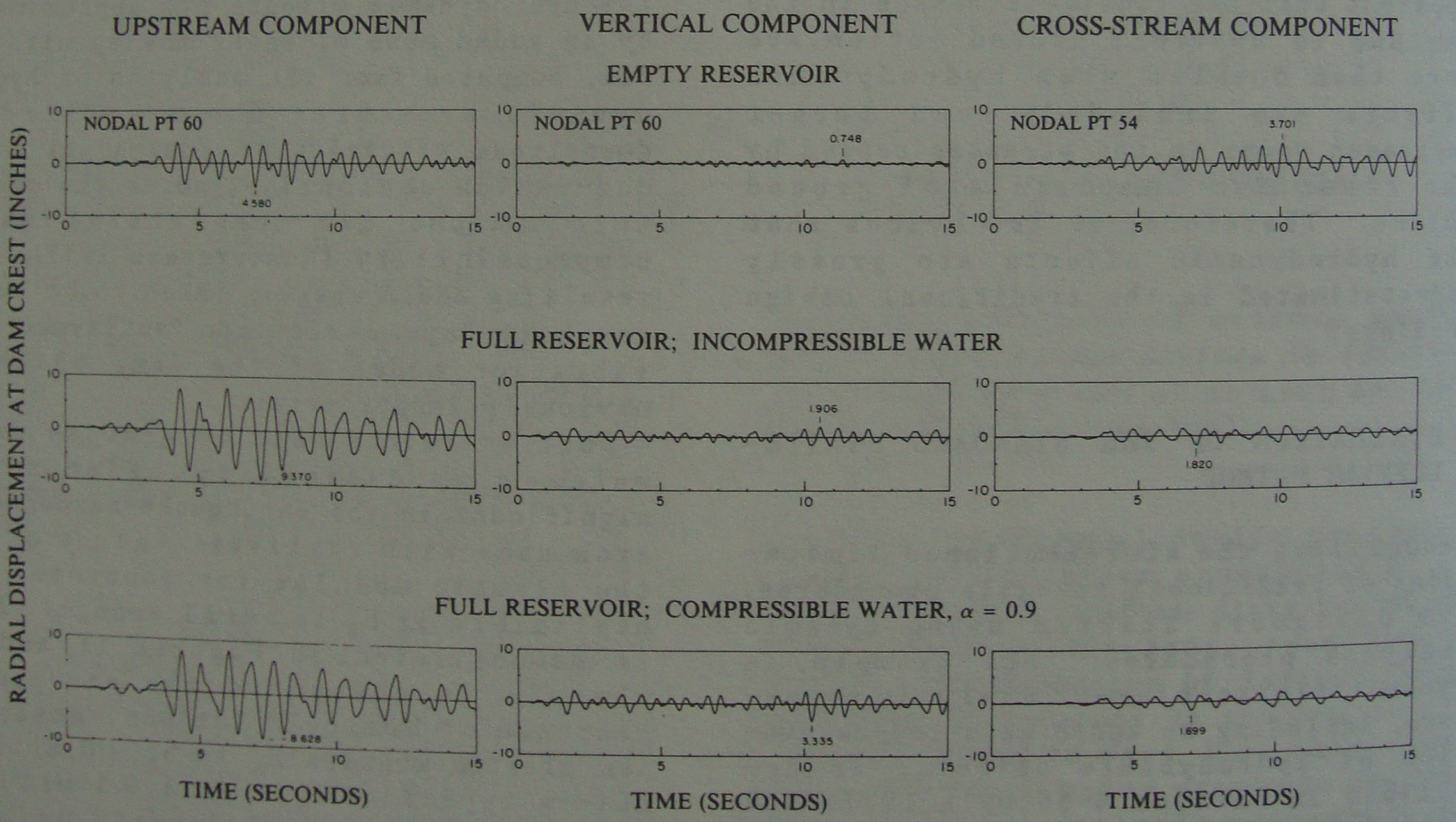


Fig.12 Displacement response of Morrow Point Dam (E_s = 0.5 million psi) due to upstream, vertical, and cross-stream components, separately, of Taft ground motion (Fok and Chopra 1987)

For Morrow Point Dam and three of its variations considered elsewhere (Fok and Chopra 1987), the effects of water compressibility are significant for E_s in the range of values from 2 to 4 million psi, which is typical of mass concrete used in arch dams. Thus, water compressibility effects should be considered in analyzing the earthquake response of concrete arch dams. The added mass representation of hydrodynamic effects, whether the added mass is determined from a two-dimensional hydrodynamic analysis assuming a straight dam (U.S. Bureau of Reclamation 1977) or from a three-dimensional analysis of the fluid domain (Kuo 1982), neglecting water compressibility in both cases, will generally lead to erroneous results for dam response.

8 ANALYSIS PROCEDURES AND COMPUTER PROGRAMS

Considering all the factors shown by recent research (Fok and Chopra 1986b) to be significant, a procedure has been developed to analyze the earthquake response of arch dams (Fok and Chopra 1986a). These include the effects of dam-water interaction, water compressibility, and reservoir boundary absorption. These hydrodynamic effects are more significant in the earthquake response of a slender arch dam than for a massive gravity dam. Considering that an arch dam resists the pressures of the impounded water and other loads in part by transmitting them through arch action to the canyon walls, the effects of dam-foundation rock interaction are likely to be significant in the earthquake response of arch dams, perhaps more so than in the case of gravity dams (see Part I). However, for reasons to be discussed subsequently in this section, the analysis procedure has, so far, not been developed to properly account for these effects; it only considers foundation rock flexibility with the inertia and damping effects of the foundation rock being ignored. By this procedure, it is possible to perform a three-dimensional analysis of a concrete arch dam supported by flexible foundation rock in a canyon and impounding a reservoir of water (Figure 13). The system is analyzed under the assumption of linear behavior for the concrete dam, impounded water, and foundation rock. Thus, the possibility of water cavitation, concrete cracking, or opening of the vertical (radial) construction joints of the dam during vibration is not considered.

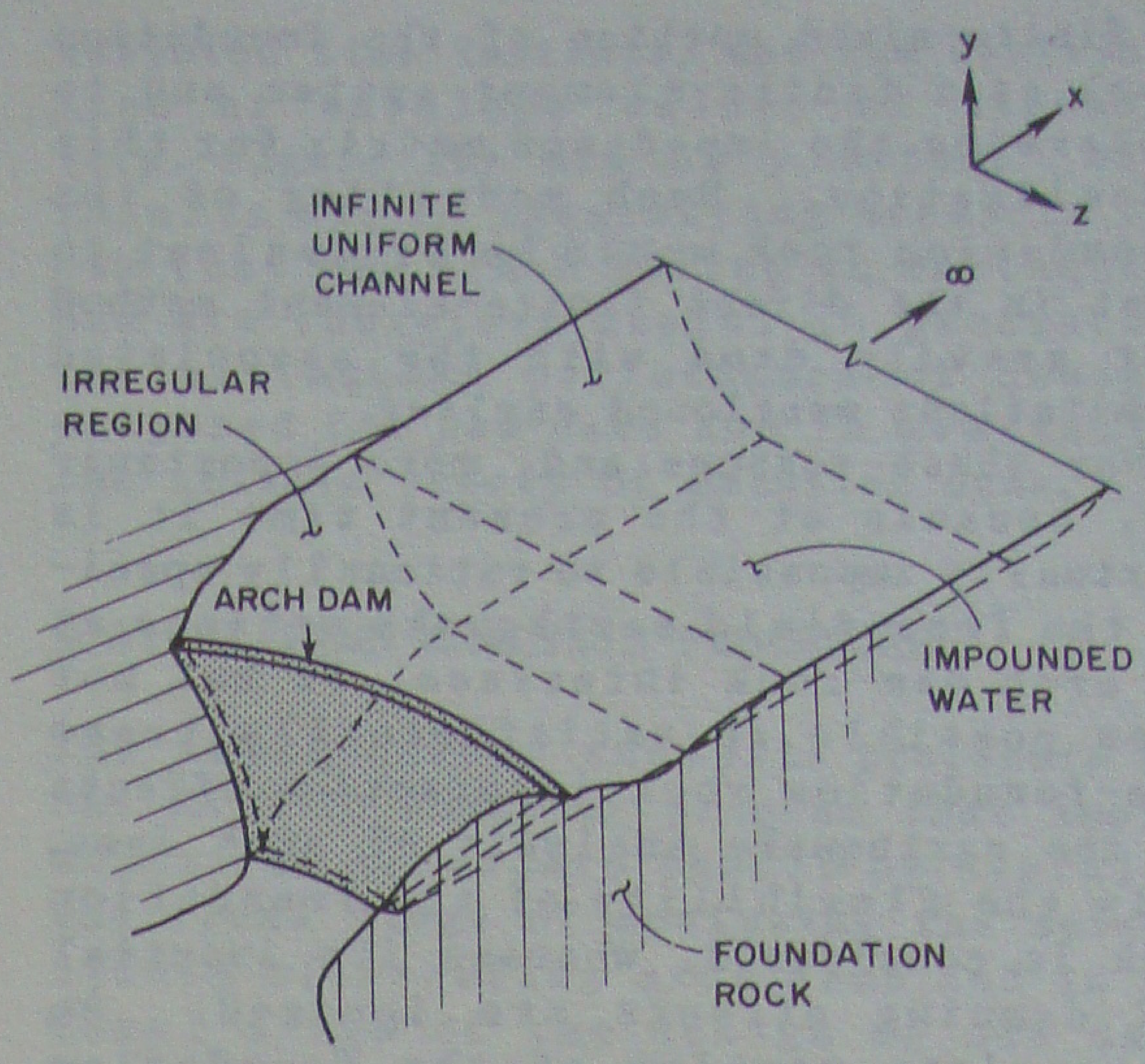


Fig.13 Arch dam-water-foundation rock system (Fok and Chopra 1985)

8.1 Arch dam

The dam is idealized as an assemblage of finite elements with a major part of the dam represented by thick shell finite elements, and the part of the dam near its junction with foundation rock represented by transition elements designed to connect thick shell elements in the dam to three-dimensional solid elements employed in idealizing the foundation rock (Fok and Chopra 1985).

8.2 Foundation rock

Required in the substructure method for analysis of earthquake response of dams is the frequency-dependent stiffness (or impedance) matrix for the foundation rock, defined at the nodal points on the dam-foundation rock interface. This matrix for a viscoelastic half-plane was determined for two-dimensional analysis of concrete gravity dams supported on the horizontal surface of foundation rock (Figure 8). However, such a foundation model is inappropriate for analysis of arch dams because they are usually built in narrow canyons with the dam boundary in contact with the foundation rock extending over the height of the dam (Figure 13).

An alternative approach is to idealize

a finite-sized portion of the foundation rock as a finite element system and to determine the impedance matrix for this idealization. Such modelling of the foundation rock would be equivalent to that in the direct finite-element method for gravity dams with the associated limitations mentioned earlier.

For these reasons and, more importantly, because at the present time it is virtually impossible to rationally specify the free-field earthquake motions at an arch dam-rock interface, it has not been possible to satisfactorily treat dam-foundation rock interaction effects in the earthquake analysis of arch dams. Only the flexibility of the foundation rock is considered, whereas its inertial and damping effects are ignored. An appropriate portion of the foundation rock region is idealized as an assemblage of three-dimensional solid finite elements, with the finite element meshes of the dam and foundation rock matching at their interface. Recommendations for the shape and size of the foundation rock region to be considered have been presented in (Fok and Chopra 1985).

8.3 Impounded water

The reservoir behind a dam is of complicated shape, as dictated by the natural topography of the site. Typically, the impounded water extends to great distances, up to a few tens of miles, in the upstream direction. Finite-element idealizations necessary to properly represent the complicated geometry of the impounded water would be exorbitantly expensive, to the point of becoming impractical, if they extend to large distances in the upstream direction.

An effective approach is to idealize the fluid domain as shown in Figure 13, with a finite region of irregular geometry adjacent to the dam connected to an infinite uniform channel--a region that extends to infinity along the upstream direction (x-axis) with uniform y-z cross-section. This restriction of a uniform cross-section for the fluid domain upstream of some cross-section is imposed because it is then possible to efficiently recognize the infinite extent of the reservoir in the upstream direction.

The finite region of irregular geometry is idealized as an assemblage of three-dimensional finite elements, with the finite element mesh compatible with that of the dam at its upstream face. For the infinite channel, a finite element discretization of the cross-section, compatible with the discretization of the irregular region over the common cross-section, combined with a continuum representation in the infinite direction provides for the proper transmission of pressure waves.

8.4 Absorptive reservoir boundary

As presented in detail for gravity dams (see Part I), the effects of the alluvium, silt, and other sediments present at the reservoir bottom and sides are modelled by an absorptive reservoir boundary characterized by a wave reflection coefficient α.

8.5 Ground motion

In earthquake response analysis of dams by the substructure method, the earthquake input is specified as the free-field ground motion at the dam-foundation rock interface (Fok and Chopra 1986a). This free-field ground motion was assumed to be uniform across the base in two-dimensional analyses of concrete gravity dams (Fenves and Chopra 1984a). This approach of specifying the same motion over the entire dam-foundation rock interface is not especially appropriate for arch dams because the dam boundary in contact with the foundation rock extends through the height of the dam, and the free-field motion is expected to vary significantly over the height. Nonuniform boundary motions can be included in finite element analysis of structures (Clough and Penzien 1975). The principal difficulty, however, is in rationally defining the variations in motions over height because no measurements of actual ground motion variations have been obtained in arch dam locations. Another possible approach is to define the earthquake input as a rigid-body translation of the basement rock on which the finite element model of the dam and foundation is supported. However, very little is known about earthquake motion at depth because most of the available strong motion records are from accelerographs located at the ground surface or in basements of buildings.

From the preceding discussion it is clear that it is difficult to define a suitable earthquake input mechanism for an arch dam. Neither of the two approaches can be justified rationally. Thus, the earthquake excitation has been defined in an extremely simple, approximate manner. As mentioned earlier, a sufficient portion of the foundation rock is included to represent only the static foundation flexibility effects, the foundation rock is assumed to be massless for the dynamic analysis, and the earthquake input is specified as spatially-uniform motion of the basement rock. Since there is no wave propagation mechanism in the massless foundation rock, the specified basement rock motion is transmitted without modification to the dam-foundation rock interface. In the context of the substructure method of analysis, the above-mentioned approximation is equivalent to specifying the same free-field ground motion throughout the dam-foundation rock interface with the foundation rock assumed to be massless in computing the foundation impedance matrix.

8.6 Computer program

The response analysis procedure has been implemented in the computer program EACD-3D: Earthquake Analysis of Concrete Dams - Three-Dimensional, to numerically evaluate the earthquake response of arch dams, idealized as described in the preceding sections. The computer program and its usage has been documented in (Fok et al. 1986), wherein the development of an appropriate finite element idealization of the system is discussed, the required input data to the computer program are described, the output is explained, and the response results from a sample analysis are presented. The computer program is available from NISEE, Davis Hall, University of California, Berkeley, California. With the computer program, a complete dynamic analysis of the response of an arch dam to the simultaneous action of upstream, vertical, and cross-stream components of ground motion can be performed. The dynamic stresses are combined with the initial, static stresses in the dam due to its weight and hydrostatic pressures. However, the user may perform a separate static analysis, including thermal, creep, construction sequence, and other effects and input the resulting initial stresses in the computer program. The output from the computer program includes the complete time history of (1) the displacements at all nodal points, and (2) the stress state in all the finite elements. From these results, the stress distribution in the dam at selected time instants, and the distribution of envelope values of stresses (e.g. Figure 10) may be plotted. Such results aid in identifying areas in the dam that may crack during an earthquake and are therefore useful in predicting the earthquake performance of designs proposed for new dams and in evaluating the seismic safety of existing dams.

9 SIMPLIFIED ANALYSIS PROCEDURE

The substructure analysis procedure and the EACD-3D computer program that implements the procedure should be useful in the final stage of analyzing the seismic safety of existing arch dams and in the final stage of the design of new dams. However, it is perhaps too complicated in the preliminary evaluation or design stage. At this early stage of an investigation, a simplified method would be useful which can consider all the significant interaction effects and provide an estimate of the maximum response directly from the earthquake design spectrum.

As presented in Part I, such a simplified procedure has been developed for concrete gravity dams. While many of the basic concepts underlying the procedure may be applicable to arch dams, the extension of the method is likely to be very difficult because of several factors that distinguish arch dams from gravity dams: arch dams must be treated as three-dimensional systems; their geometry varies considerably from one site to another, precluding the possibility of defining "standard" values for vibration properties and parameters; and their response is generally not dominated by the fundamental mode of vibration.

PART III: SEISMIC DESIGN AND SAFETY EVALUATION

10 EARTHQUAKE-RESISTANT DESIGN OF NEW DAMS

Concrete dams should be designed to elastically resist the relatively frequent, moderate intensity earthquakes. However, some damage, which is limited enough so that it is economically repairable and does not impair the ability of the structure to contain the impounded water, may be permitted in the rare event that very intense ground shaking occurs.

10.1 Design earthquakes

Definition of the ground motions at the site in question for design of the dam should, of course, be based on the history of seismic activity in the area, distance of the site to active faults, length of potential fault breaks, ground motions recorded at similar nearby sites during past earthquakes, etc. Ground motions should be selected which are representative of: (1) moderately intense shaking expected to occur during the useful life of the structure, and (2) the most intense shaking that can occur at the site. The ground motions should be defined for both levels of shaking, either by simulated motions (Jennings et al. 1969) or by appropriately scaling and/or modifying suitable existing accelerograms. The upstream, cross-stream and vertical components of ground motion are needed in analysis of arch dams, whereas it is usually sufficient to define only the upstream and vertical components in two-dimensional analysis of gravity dams. For computation of earthquake forces in the preliminary phase of elastic design, response spectra for the horizontal components of ground motion will be needed. Instead of the response spectrum corresponding to the hypothesized ground motion, a smooth spectrum (Newmark et al. 1973) with appropriate intensity is more suitable, for it does not contain the irregularities of individual response spectra.

10.2 Elastic design

A two-stage procedure is appropriate for the analysis phase of elastic design of concrete dams: (1) simplified analysis procedure in which the response is estimated directly from the earthquake design spectrum, considering only those factors that are most important in the earthquake response of dams and yet simple enough so as not to require the use of elaborate computer programs; and (2) a refined response history analysis procedure for finite-element idealizations of the dam. The former is recommended for purposes of preliminary design, and the latter for accurately computing the dynamic response and checking the adequacy of the structure designed for the preliminary design forces. The simplified analysis procedure summarized in section 5 is appropriate for gravity dams but a corresponding procedure remains to be developed for arch dams. As mentioned earlier, computer programs EAGD-84 and EACD-3D are available to implement refined response history analysis procedures for gravity, arch, as well as buttress dams.

For purposes of preliminary design of gravity dams, the earthquake forces should be determined from the simplified analysis procedure for the selected earthquake design spectrum for 5% damping, an appropriate value for concrete dams. The stresses can then be calculated from the same simple formulas for direct and bending stresses in beams that are employed in the traditional design procedures. The dam should be designed--dimensions established and required concrete strength in tension and compression determined--for these loads, considering both upstream and downstream directions for the earthquake forces. The design should provide against overstressing in compression and tension, i.e. the compressive and tensile stresses should not exceed the compressive and tensile strengths, respectively. Usually, the concrete strength requirements will be controlled by the tensile stresses because they will be similar in magnitude to the compressive stresses, whereas the tensile strength of mass concrete is an order of magnitude less than the compressive strength. The traditional overturning and sliding stability criteria have little meaning in the context of the oscillatory response of dams during earthquakes; these should not be used until realistic criteria have been developed. The end result of this phase of the design process is a preliminary design of the dam.

It is apparent from the preceding discussion that the key property which determines the capacity of concrete dams to withstand earthquakes is the tensile strength of concrete. Tensile strength can be determined from three types of tests: direction tension, splitting tension, and flexural tests. Results of these tests differ, and results of tests on cores taken in the field differ compared to tests on laboratory specimens. The direct tension test is difficult to accomplish and grossly underestimates the tensile strength of concrete if the specimen is allowed to surface dry. The flexural test, together with its usual linearly derived modulus of rupture, provides a basis to determine the tensile strength. The modulus of rupture should be multiplied by a factor which accounts for the nonlinear behavior of concrete and depends on the shape of the specimen (Raphael 1984). On the other hand, the splitting tension test is easiest to accomplish and gives the most reliable results. However, tensile strength obtained from the splitting tension test should be multiplied by about 4/3 to account for the nonlinear behavior of concrete near failure before using it to interpret results of linear finite-element analysis (Raphael 1984).

Because the tensile strength of concrete depends on the rate of loading (Hatano and Tsutsumi 1969, Hatano 1960), the aforementioned tests should be conducted at loading rates the concrete may experience during earthquake motions of the dam. Lacking the facility to perform dynamic tests, it is recommended that the tensile strength of concrete for judging the seismic safety of a concrete dam should be the static value augmented by a multiplier of about 1.5.

The tensile strength should obviously be determined from appropriate tests on specimens of concrete for the particular dam. However, a preliminary estimate of the tensile strength can be obtained from Figure 14 which presents four plots of

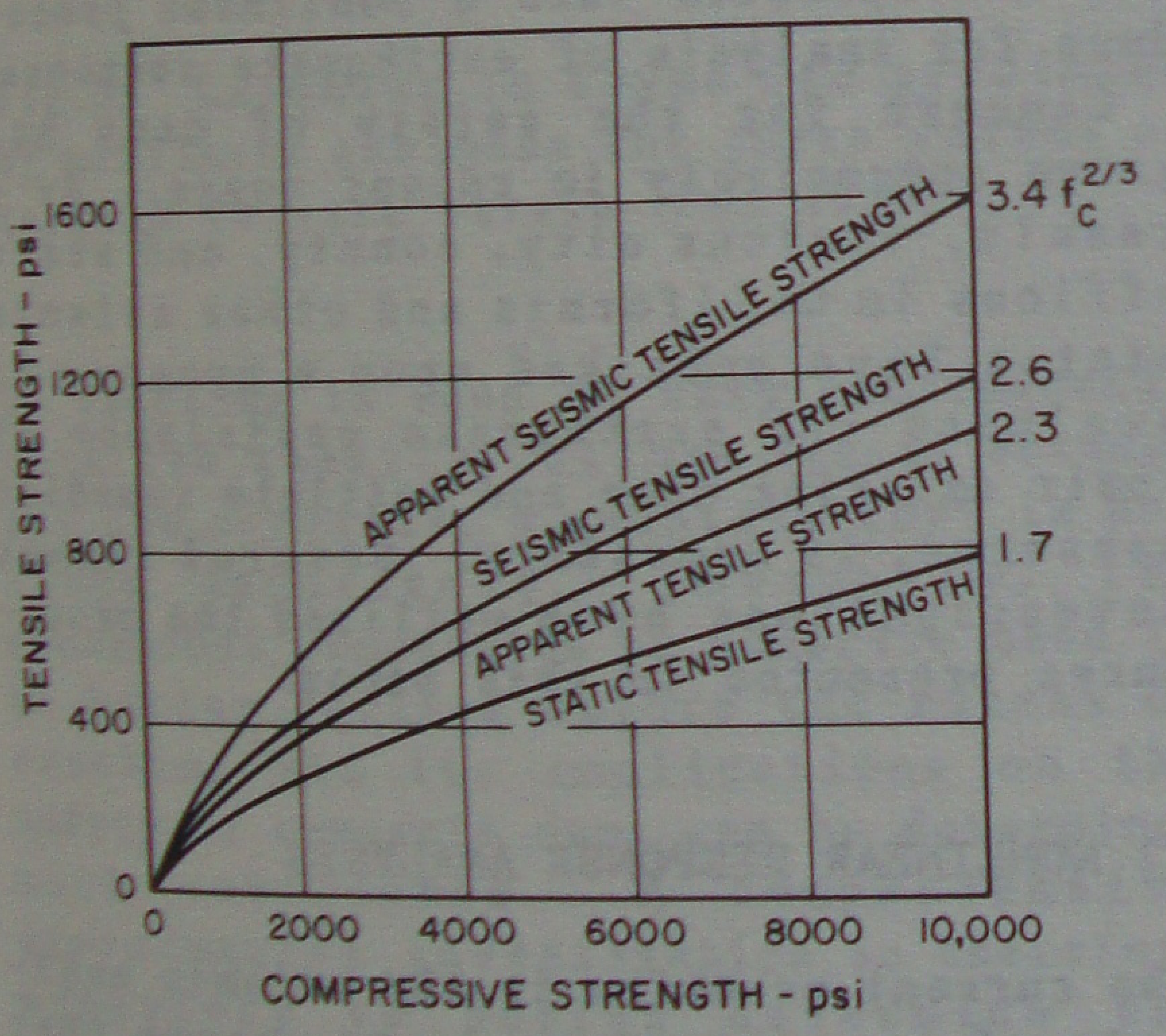


Fig.14 Design chart for tensile strength (Raphael 1984)

tensile strength as a function of compressive strength, to be used depending on need. The lowest two plots, $f_t = 1.7f_c^{2/3}$ and $f_t = 2.3f_c^{2/3}$, are for long-time or static loading. The lowest plot represents actual tensile strength, whereas the second plot takes into account the nonlinearity of concrete and is to be used to interpret the stresses computed by linear finite-element analysis. The third and fourth plots, $f_t = 2.6f_c^{2/3}$ and $f_t = 3.4f_c^{2/3}$, are the actual and "apparent" tensile strengths under seismic loading.

Permitting significant tensile stresses, up to the tensile strength of concrete, is of course a major departure from the standard design criteria wherein little tension is permitted. However, evidence is available to support the recommended design criteria that significant dynamic stresses in tension can be carried by sound concrete. In addition to the data from laboratory tests mentioned earlier, evidence of the dynamic tensile strength of concrete was provided by the performance of dams during earthquakes. Dynamic analyses indicated that Pacoima Dam should have developed maximum tensile stresses in the order of 750 psi during the San Fernando earthquake of 1971, yet no evidence of cracking could be found on either face of the dam (Swanson and Sharma 1979). As mentioned earlier, elastic analyses of Koyna Dam indicated tensile stresses almost three times the tensile strength of concrete, resulting in significant cracking of the dam. However, the dam survived the earthquake without any sudden release of water. Perhaps most interesting is the lack of damage to Crystal Springs Dam--a curved concrete gravity dam located approximately 1,000 ft from the San Andreas fault--during the great San Francisco earthquake of 1906 (Wulff and Van Orden 1979).

The adequacy of the preliminary design of the dam should be checked with the aid of a refined, accurate analysis procedure. Using a computer program such as EAGD-84 for gravity dams or EACD-3D for arch dams, the response of the preliminary design of the dam to the selected ground motion should be determined, resulting in more accurate values for the stresses. Based on these results, the preliminary design of the dam should be revised, if necessary, to satisfy the same design criteria mentioned earlier. Often, the design modification may involve increasing only the concrete strength.

However, as mentioned earlier in this paper, the design stresses in gravity dams can be significantly reduced by modifying the usual designs to reduce the weight near the crest of the dam. Instead of the solid concrete block added near the crest in typical designs of dams to support the roadway and to resist the impact of floating objects, lightweight structural systems would be preferable.

Similarly, auxiliary structures usually appended on top of dams should be located with discretion so that they have a minimum of adverse effect on stresses in dams (Chopra and Chakrabarti 1973). Possible modifications in the geometry and mass distribution of arch dams which may lead to reduction of earthquake-induced stresses need to be investigated.

10.3 Designing for intense motions

Earthquakes of large magnitude causing very intense ground shaking, although rare, can occur in California and other highly seismic areas of the world, and they must be considered in the design of major dams. A conservative criterion, which would perhaps result in uneconomical designs, is to require that the dam remain elastic even during the most intense shaking that may occur. With the design spectrum and ground motion chosen to be representative of such shaking, the elastic design procedure presented in the preceding section would apply. However, as indicated by the earthquake performance of Koyna Dam, concrete dams can continue to contain the impounded water even after they have undergone significant cracking. In designing concrete dams for the very intense ground shaking that may occur only rarely, it is therefore reasonable to permit limited cracking that is economically repairable and does not impair the ability of the structure to contain the impounded water.

However, as will be mentioned later in this paper, at the present time it is not possible to analytically predict with a high degree of confidence the extent of cracking, cantilever joint opening, and damage that a concrete dam may experience during very intense ground shaking. Until reliable prediction procedures are developed, the design of dams for very intense earthquake motions will continue to rely almost exclusively on what is known about their actual performance during past earthquakes.

11 SEISMIC SAFETY EVALUATION OF EXISTING DAMS

The procedure presented in the preceding section for design of new concrete dams to be built in seismic regions is also applicable--albeit with some obvious changes in interpretation--to evaluation of the safety of existing dams. Most of these dams were designed using the traditional procedures which, as shown earlier, are unrealistic. Currently, there is considerable interest in evaluating the safety of these structures with the aid of the better analysis techniques and new information on tensile strength of concrete that are now available. The analysis procedures and design criteria presented earlier for design of new dams are applicable as analysis procedures and evaluation criteria for determining the safety of existing dams. The simplified procedure for calculating earthquake forces and resulting stresses is also appropriate for a preliminary evaluation of the safety of an existing dam. If the results indicate the need for a more thorough evaluation, the refined computer analysis procedures mentioned earlier should be employed. If such elastic analyses for the most intense ground shaking indicate tensile stresses much larger than the tensile strength of concrete, cracking of concrete can be expected. However, as mentioned earlier, the extent of cracking and its implications to the safety of the dam cannot be determined with a high degree of confidence until the mechanical properties of concrete are better determined and incorporated into a nonlinear procedure for analysis of earthquake response.

Concern for the safety of dams has grown appreciably in recent years. As a result, various city, county, and state offices in California and other seismic states have embarked upon a program to evaluate the earthquake resistance of their existing dams and initiate remedial measures, if necessary. Several recent investigations have utilized the procedures presented in this paper.

12 NONLINEAR RESPONSE ANALYSIS

The current practice in the seismic analysis of concrete dams is to assume that the structure as well as its interaction mechanisms with the impounded water and foundation rock are linearly elastic. Ignored in such analyses, which have been presented in this paper, are the nonlinear effects associated with the possible opening or slippage of vertical construction joints, cracking of concrete, and the local separation of water from the upstream face of the dam. Such linear analysis procedures provide good estimates of the response of dams to moderately intense ground motions, in which case these nonlinear mechanisms are not likely to develop. However, they may not satisfactorily represent the true behavior of dams during intense earthquake motions.

As discussed earlier, linear analysis of Koyna Dam predicted locations in the dam where cracking will be initiated consistent with the damage caused by Koyna Earthquake. However, such an analysis cannot predict the extent of cracking. In principle, nonlinear analysis--including possible cracking of concrete--of gravity dams subjected to earthquake ground motions can be performed (Pal 1976). However, the predictions of the extent of cracking as obtained from these analyses are quite sensitive to the assumed mechanical properties of concrete. This is illustrated in Figure 15 where the extent

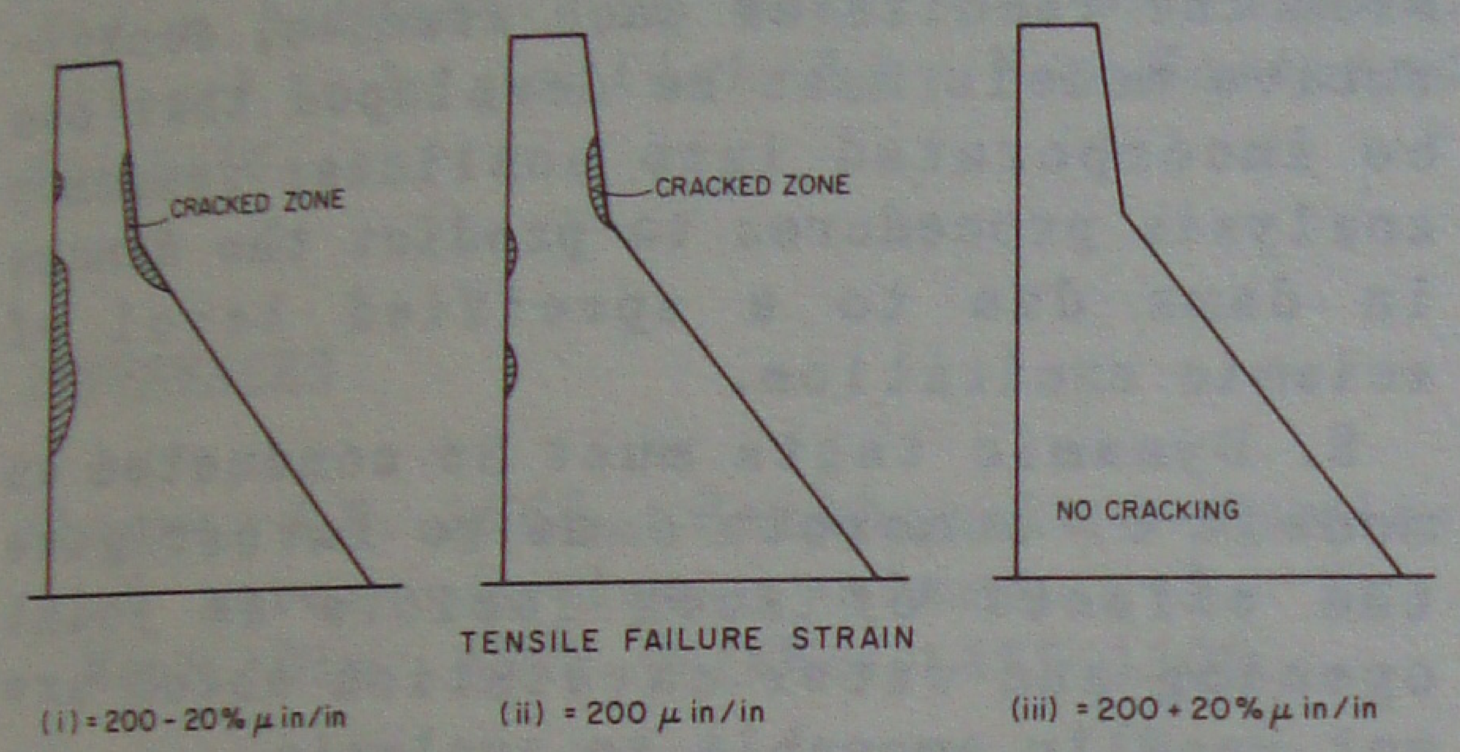


Fig.15 Cracking in Koyna Dam predicted by nonlinear analyses for three values of tensile failure strain (Pal 1976)

of cracking is seen to vary considerably with the tensile failure strain assumed for the concrete, a quantity that is not known accurately. Obviously, the mechanical properties of concrete need to be better defined before the extent of cracking and its implications on the safety of concrete dams can be determined with confidence. A comprehensive experimental investigation is therefore necessary to better determine the constitutive and strength properties of multiaxially loaded mass concrete under dynamic, reversible, cyclic strains and stresses representative of earthquake conditions. Thereafter, these properties should be incorporated into a nonlinear response analysis procedure.

The ground motion experienced by Pacoima Dam during the San Fernando earthquake of 1971 must have been very intense: accelerations of 1.25 g in a horizontal direction and 0.7 g in the vertical direction were recorded by an accelerograph located at the edge of a narrow, badly fractured ridge adjacent to the dam, and accelerations were estimated at the base of the dam in the range of 0.6 to 0.8 g. Analyses indicated that the dam must have developed maximum tensile stresses on the order of 750 psi in the arch direction, which is in excess of the tensile strength of concrete (Swanson and Sharma 1979), yet no evidence of cracking could be found on either face of the dam--except for an opening of the vertical, radial construction joint between the arch and the left abutment thrust block--suggesting that the tensile arch stresses must have been relieved at the radial construction joints. No calculations have yet been made that account for the nonlinear effects associated with such joint openings in a realistic fashion.

Linear analyses of dam response to intense earthquake motions indicate the possibility of cavitation, i.e. local separation of water from the upstream face of the dam. Results of recent nonlinear analyses suggest that the effects of cavitation are not as significant as they might have been presumed (Zienkiewicz et al. 1983). Therefore, it may not be necessary to consider this nonlinear effect in practical applications.

PART IV: CURRENT RESEARCH NEEDS

Federal and state agencies concerned with the construction of dams have revised their design standards and engineering companies have updated their procedures to acknowledge the research accomplishments of the past decade, some of which have been summarized in this paper. Static force methods involving seismic coefficients have given way to dynamic analysis procedures. As shown in this paper, these procedures should consider the following factors: dam-water interaction, reservoir boundary absorption, water compressibility, and dam-foundation rock interaction. In order to produce safe and economical designs of concrete dams, the most reliable techniques considering the above-mentioned factors should be used to evaluate proposed design for new dams to be constructed; furthermore, the reliability of present design methods should be improved.

In response to growing public concern for the safety of dams and reservoirs, major federal dam building agencies and states such as California and Utah have adopted programs for evaluating the safety of existing dams. Since most of these dams were designed by methods that are now considered oversimplified, there is considerable interest in reevaluating the original designs using current procedures. As a result of such safety evaluations, structural modifications have been made to some dams, and restrictions on reservoir water levels have been imposed in some cases. Since the economic impact of such modifications and restrictions is generally substantial, it is important to improve the reliability of present methods of safety evaluation.

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

1. Major dams in seismic areas of the United States must 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, the response of the dam, and the hydrodynamic pressures exerted on the dam. Because of the urgent need for such data, dams in highly seismic regions of other countries should also 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. Forced-vibration tests should be conducted on selected dams at different water levels, and the resulting hydrodynamic pressures and motions of the structures and their foundations must be recorded and analyzed.

3. Existing analytical methods for computing the response of all types of dams to earthquakes should be refined, and their reliability should be assessed by comparing their results with the responses recorded during forced-vibration tests and actual earthquakes. These methods and the computer programs needed for their implementation should be developed in a form convenient for application in engineering practice.

4. The present methods for earthquake analysis of arch dams urgently need improvement as they may be seriously deficient in their treatment of the earthquake excitation and of the interaction between the dam and the supporting foundation rock.

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

6. Analysis procedures should be developed that can determine the dynamic sliding and rocking response of gravity dam monoliths. Utilizing these procedures, rational criteria for sliding and overturning stability of dams should be determined.

7. Appropriate equipment must be developed for testing multiaxially loaded mass concrete specimens under cyclic deformations of the type induced by earthquakes, and comprehensive experimental studies should be undertaken to define the stress-strain characteristics and strength of mass concrete better. From the results of such studies, constitutive models must be developed that can be incorporated into nonlinear response analysis procedures to predict the damage in dams due to a specified level of seismic excitation.

8. Dynamic tests must be conducted on models of concrete dams to investigate the effects of such factors as joint opening and water cavitation which are not readily amenable to analysis.

9. Analysis procedures must be developed that can account for the effects of concrete cracking, joint opening, and water cavitation, and parametric response studies must be carried out to evaluate their importance. From the results of these studies and of the experimental program suggested in item 8, the conditions must be defined under which these nonlinear effects are sufficiently important to warrant their consideration in design, and appropriate design guidelines must be developed.

10. Static tests to failure must be conducted on large-scale models of dams, especially arch dams, to define their behavior in the range approaching failure.

11. The results of these studies should be used to evolve rational and practical methods for designing concrete dams to be built in seismic regions and for evaluating the safety of existing dams.

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