Bull. Ind. Soc. Earthq. Tech., Paper No. 251, Vol. 23, No. 4, December 1986, pp. 147—199

EARTHQUAKE ANALYSIS, DESIGN AND SAFETY EVALUATION OF CONCRETE GRAVITY DAMS*

ANIL K. CHOPRA**

INTRODUCTION

President Shamsher Prakash, respected Dr. Jai Krishna, ladies and gentlemen, I am grateful to the Indian Society of Earthquake Technology for providing me with this opportunity to present the results of some of the research I have conducted in collaboration with my graduate students. The subject I have chosen to present to you is the earthquake response of concrete gravity dams. This choice has been motivated by two factors. Firstly, the early impetus for my research on dams was provided by an earthquake experience familiar to most of you—the earthquake that occurred in 1967 close to Koyna Dam near Poona, India. As you know, the dam was overstressed by the earthquake motions and was damaged to an alarming degree. This experience with the earthquake performance of Koyna Dam, which was designed by the best available procedure of the time, indicated that concrete gravity dams are not immune to earthquake damage as had commonly been presumed. This realization led to much interest in developing improved procedures for analysis, design and safety evaluation of concrete gravity dams. Secondly, India is one of the few countries where new concrete dams are still being built, so this should be a topic of interest to researchers and engineers here. Because arch dams are rare in India, this presentation is restricted to concrete gravity dams.

This presentation summarizes our work on the earthquake response analysis of concrete gravity 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 will be identified, the factors that should be considered in dynamic analysis will be discussed, and procedures for simplified response spectrum analysis and refined response history analysis will be summarized. The application of these linear analysis procedures to seismic design and safety evaluation of dams will be discussed, followed by the limitations of the presently available nonlinear analysis procedures in predicting the extent of cracking and damage that a dam may experience during intense ground shaking.

* 7th ISET Annual Lecture.

**Professor of Civil Engineering, University of California, Berkeley, U.S.A.

EVALUATION OF TRADITIONAL ANALYSIS AND DESIGN PROCEDURES[5]

Traditional Analysis and Design Procedures

Although new design criteria are now available,[1,2] traditionally, concrete gravity dams have been designed and analyzed by very simple procedures.[3,4] 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.

Earthquake Performance of Koyna Dam

Koyna Dam is one of a few concrete dams that has experienced a destructive earthquake.[6] 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 (Fig. 1), are approximately two to three times the tensile strength (350 psi) of the

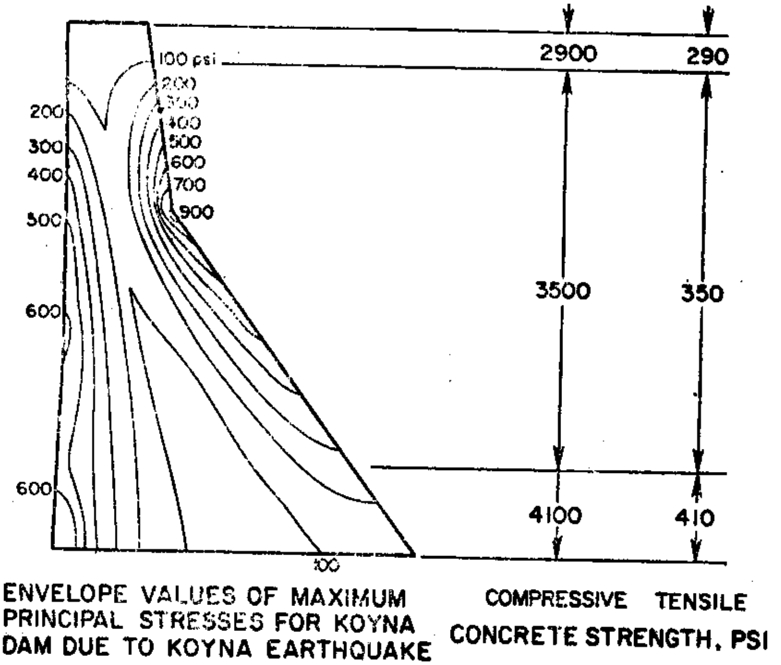


Fig. 1—Comparison of stresses in Koyna Dam, predicted by linear analysis, with tensile strength of concrete. (Adapted from Refs. 6 and 8)

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 Fig. 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.[6]

Limitations of Traditional Design Procedures

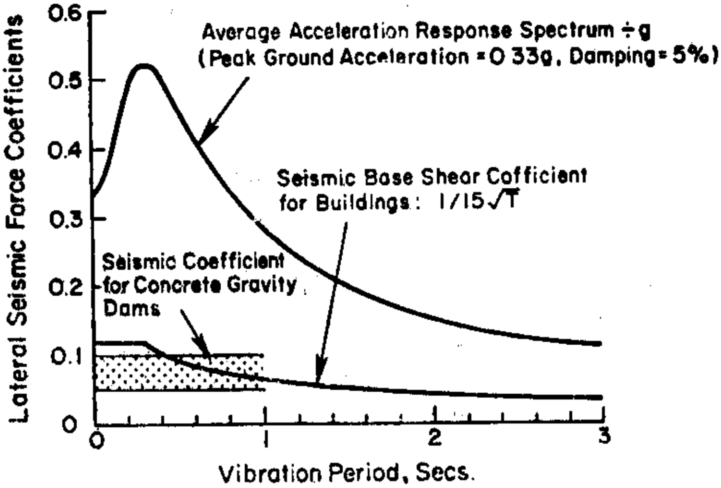
It is apparent from the preceding analysis that stresses in gravity dams due to standard design loads have little resemblance to dynamic response of such dams to earthquake ground motion. In the case of Koyna Dam, the earthquake forces included in the design loads were based on a seismic coefficient 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 (Fig. 2a), which is about the largest possible vibration period for a concrete gravity dam. It is of interest to note in Fig. 2a that the seismic base shear coefficient values for dams are similar to those specified for buildings.[7] However, building code design provisions[7] are based on the premise that buildings should be able to :

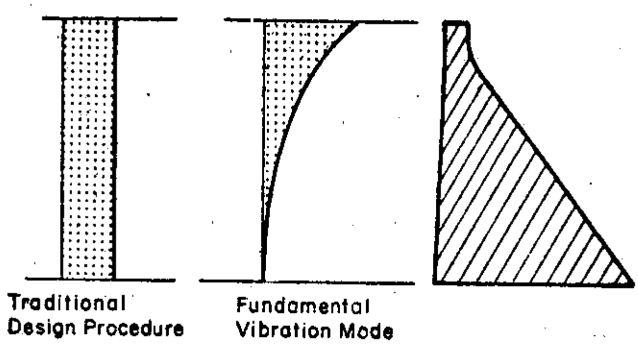
> "1. Resist minor earthquakes without damage; 2. Resist moderate earthquakes without structural damage, but with some nonstructural damage; 3. Resist major earthquakes ...... without collapse but with some structural ..... damage."

Whereas these may be appropriate design objectives, major dams should be designed more conservatively and this is reflected in the afore-mentioned 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 (Fig. 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 Fig. 2b. In contrast,



(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. (From Ref. 5)

traditional analysis and design procedures ignore the dynamic 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 (Fig. 1) and Dworshak Dam (in the United States). 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 (Fig. 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 [8] This added mass has little, if any, adverse effect on the stresses predicted by traditional analysis because of the relatively small values for the seismic coefficient which are to increased near the crest (Fig. 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 Fig. 3.

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.[3,4] One of these[4] 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, tha additional water pressure at the base of the dam is slightly over 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.

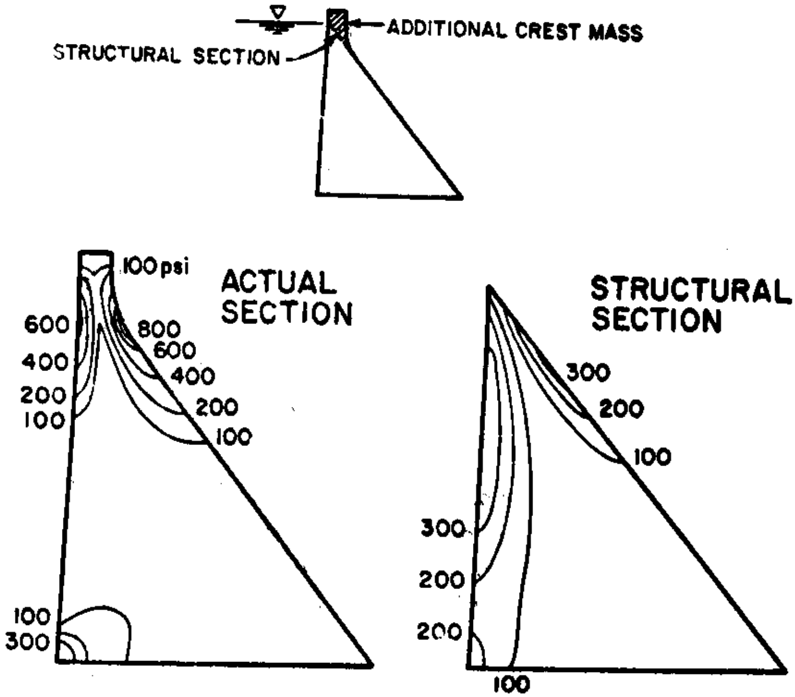
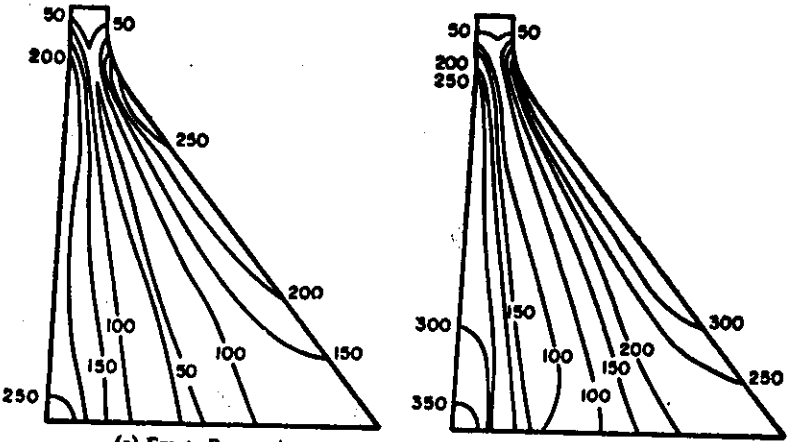


Fig. 3 – Increased stresses due to additional, nonstructural mass at the dam crest. Critical stresses in Pine Flat Dam due to Koyna earthquake are shown, (From Ref. 8)

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 Fig. 4 wherein the "envelope" values of the earthquake—induced stresses in Pine Flat Dam 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.

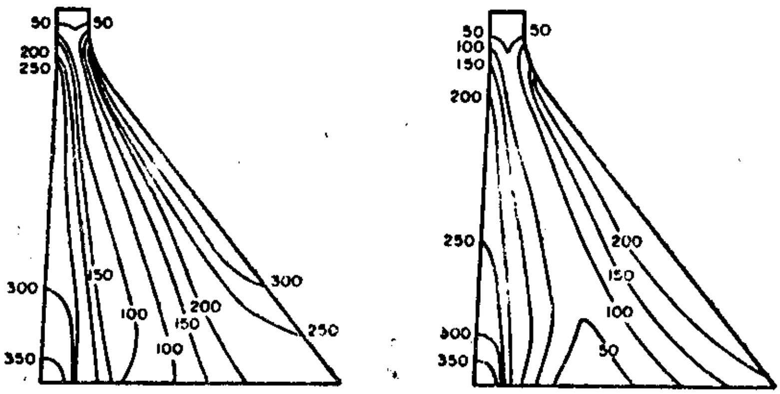


(a) Empty Reservoir

(b) Full Reservoir
Rigid Reservoir Bottom, $\alpha=1$

Fig. 4—Envelope values of maximum principal stresses (in psi) in Pine Flat Dam on rigid foundation rock due to S69E component, only, of Taft ground motion. Initial static stresses are excluded. (From Ref. 16)

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 Fig. 5.



(a) Rigid Foundation Rock

(b) Flexible Foundation Rock

Fig. 5—Envelope values of maximum principal stresses (in psi) in Pine Flat Dam with full reservoir and nonabsorptive (rigid) reservoir bottom ($\alpha=1$). (From Ref. 16)

EVALUATION OF THE STANDARD FINITE-ELEMENT METHOD[1]

It is apparent from the preceding section that traditional seismic coefficient 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 U.S. Bureau of Reclamation publication[2] 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 regid. 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 depth, 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.[10]

Although studies conducted as early as 1968 and 1970 concluded that water compressibility effects are significant in the response of concrete gravity dams,[10,11] there continues to be much interest in research[12] and in practical applications[13] 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[11,14] 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 incomprssible 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 $\alpha = 0.9$ 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

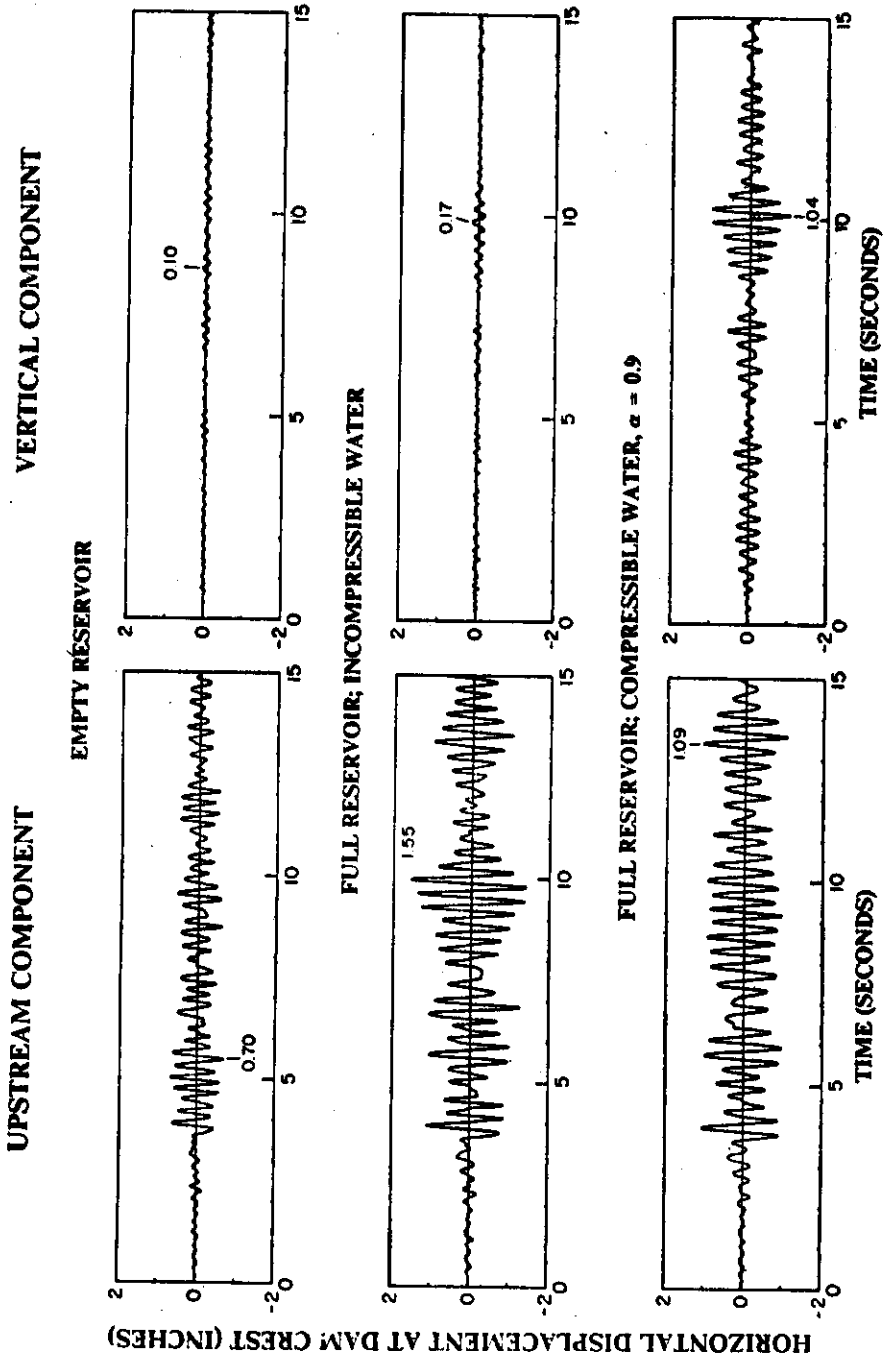


Fig. 6—Displacement response of Pine Flat Dam (E_s=4 million psi) due to upstream and vertical components, separately, of Taft ground motion.

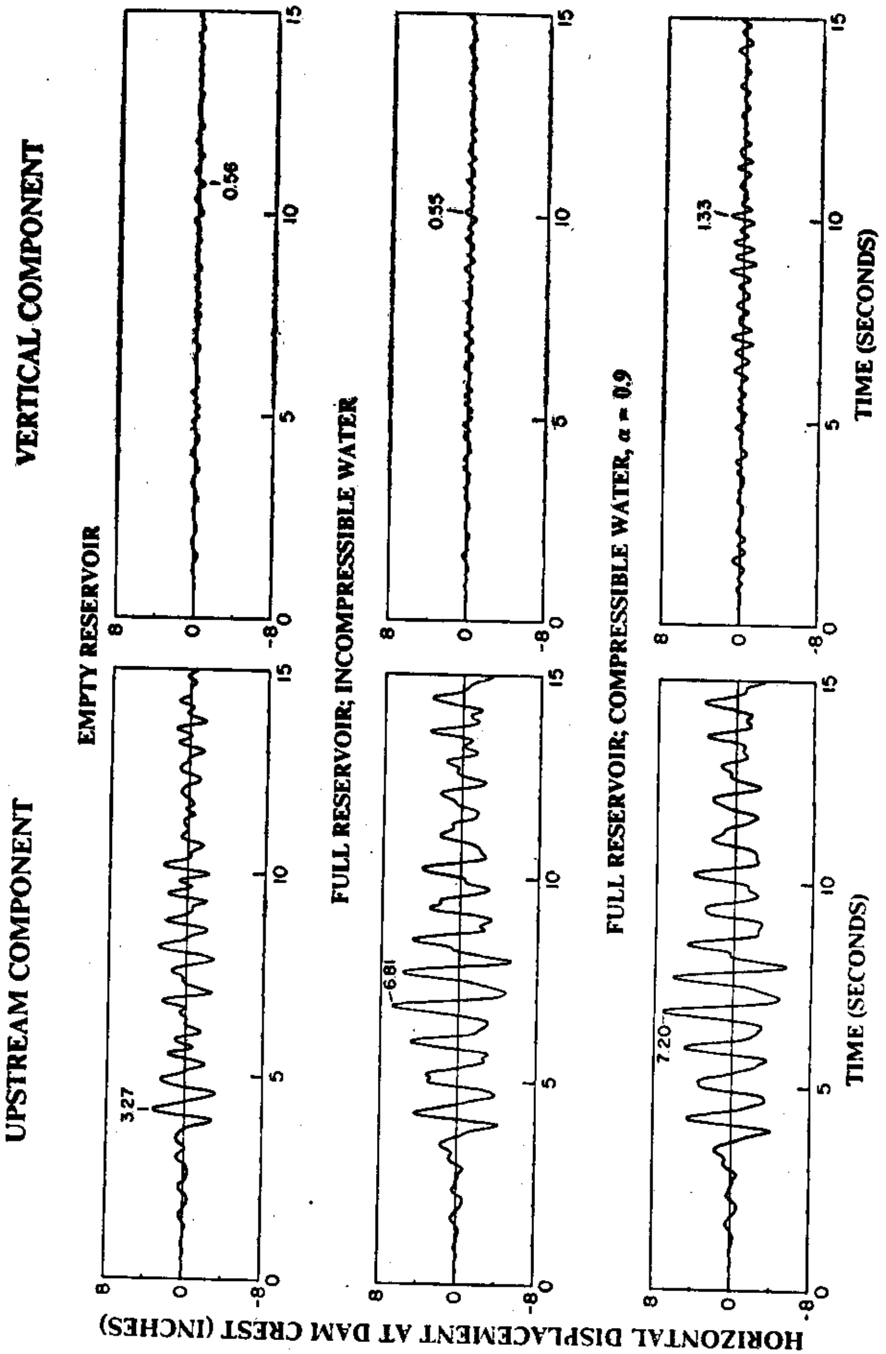


Fig. 7—Displacement response of Pine Flat Dam (E_s = 0 65 million psi) due to upstream and vertical components, separately, of Taft ground motion.

of the reflected hydrodynamic pressure wave 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 [14,15] and partly from the change in the response spectrum ordinates corresponding to the modified resonant periods and damping The lengthening of the fundamentali period due to dam-waterinteraction 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 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.

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:[16] (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 gravita dams idealized as two-dimensional systems [17,18] or treated as three-dimensional system [19] 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 treated in an overly simplified manner in the three-dimensional analysis.

At small vibration amplitudes a concrete gravity dam will behave as a soild even though the construction joints between the monoliths may slip[20]. 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.[6] 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 (Fig. 8). All response results presented in the preceding sections were obtained from this computer program. The selected

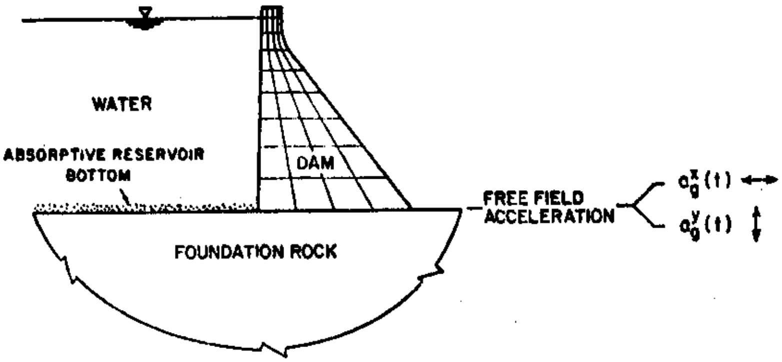


Fig. 8—Dam-water-foundation rock system. (From Ref. 17)

monolith in generalized plane stress or dam cross-section in plane strain is idealized as a two-dimensional finite-element system. In obtaining the response results presented in the preceding sections, the tallest nonoverflow monolith of Pine Flat Dam was indealized as a finite-element system consisting of 136 quadrilateral elements with eight elements across the width and seventeen over the height of the monolith, and 162 nodal points.[17] The finite-element idealization makes it possible to model a·b·trary 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 gemetry of the dam to permit a continuum solution for hydrodynanmic 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 in order 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 conditions at the reservoir bottom that partially absorbs incident hydrodynamic pressure waves.[17]

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[17] to evaluate the earthquake response of concrete gravity dams, idealized as described earlier, including all the aforementioned factors which are significant in dam response. EAGD-84* (Earthquake Analysis of Gravity Dams-1984), the computer program that implements the analytical procedure, is described in Ref. 18, where in 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.

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 Fig. 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.

SIMPLIFIED ANALYSIS PROCEDURE

While the substructure analysis procedure[17] and the EAGD-84 computer program[18] that implements the procedure are appropriate for analyzing the safety of existing dams against future earthquakes in the

* Available from NISEE (National Information Service in Earthquake Engiring), 404A Davis Hall, University of California, Berkeley CA 94720.

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 was computed directly from the earthquake design spectrum, without a response history analysis[5]. 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,[21] 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 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 compressibi ity 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 presentation of the new version of the simplified procedure in the following sections is taken from Ref. 21. The standard data necessary to implement this procedure are presented as both figures and tables in Ref. 22 available from the University of California, Berkeley; for brevity only a small portion of the data are illustra.ed here.

Selection of System Parameters

The simplifed analysis procedure requires only a few parameters to describe the dam-water foundation rock system : E_s, ξ_1, H_s, η_f, H, and α. The selection of an appropriate value for each parameter is discussed next.

The Young's modulus of elasticity E_s for the dam concrete should be based on the design strength of the concrete or suitable test data, if available. The value of E_s may be modified to recognize the strain rates representative of those the concrete may experience during earthquake motions of the dam [5] In using the standard data of Ref. 22 to conservatively include dam-water interaction effects in the computation of earthquake forces, the E_s value should be rounded down to the nearest value for which data are available : E_s = 1.0, 2 0, 2 5, 3.0, 3.5, 4 0, 4 5, or 5 0 million pounds per square inch. Forced vibration tests on dams indicate that the viscous damping ratio ξ_1 for concrete dams is in the range of 1 to 3 percent.

However, for the large motions and high stresses expected in a dam during intense earthquakes, $\xi_1 = 5$ percent is recommended. The height H_s of the dam is measured from the base to the crest.

The Young's modulus of elasticity E_f and constant hysteretic damping coefficient η_f of the foundation rock should be determined from a site investigation and appropriate test. To be conservative, the value of η_f should be rounded down to the nearest value for which data are available : $\eta_f = 0.01, 0.10, 0.25,$ or 0.50, and the value of E_f/E_s should be rounded up to the nearest value for which data are available. In the absence of information on damping properties of the foundation rock, a value of $\eta_f = 0.10$ is recommended.

The depth H of the impounded water is measured from the free suface to the reservoir bottom. It is not necessary for the reservoir bottom and dam base to be at the same elevation. The standard values for unit weight of water and velocity of pressure waves in water are $w = 62.4$ pcf and $C = 4720$ fps, respectively.

It may be impractical to determine reliably the wave reflection coefficient α because the reservoir bottom materials may consist of highly variable layers of exposed bedrock, alluvium, silt, and other sediments, and appropriate site investigation techniques have not been developed. However, to be conservative, the estimated value of α should be rounded up to the nearest value for which the figures and tables are presented : $\alpha = 1.0$, 0.90, 0.75, 0.50, 0.25, 0.00 For proposed new dams or recent dams where sediment deposits are meager, $\alpha = 0.90$ or 1.0 is recommended and, lacking data, $\alpha = 0.75$ or 0.90 is recommended for older dams where sediment deposits are substantial. In each case, the larger α value will generally give conservative results, which is appropriate at the preliminary design stage.

Design Earthquake Spectrum

The horizontal earthquake ground motion is specified by a pseudo-acceleration response spectrum in the simplified analysis procedure. This should be a smooth response spectrum—with out the irregularities inherent in response spectra of individual ground motions— — representative of the intensity and frequency characteristics of the design earthquakes which should be established after a thorough seismological and geological investigation; see Ref. 5 for more detail.

Computational Steps

The computation of earthquake response of the dam is organized in four parts :

Part 1 : The earthquake forces and stresses due to the fundamental vibration mode can be determined approximately for purposes of preliminary design by the following computational steps:

1. Compute T_1, the fundamental vibration period of the dam, in seconds, on rigid foundation rock with an empty reservoir from

$$T_1 = 1.4 \frac{H_s}{\sqrt{E_s}} \tag{1}$$

in which H_s = height of the dam in feet, and E_s = design value for Young's modulus of elasticity of concrete, in pounds per square inch.

2. Compute $\tilde{T}_r$, the fundamental vibration period of the dam, in seconds, including the influence of impounded water from

$$\tilde{T}_r = R_r T_1 \tag{2}$$

in which T_1 was computed in Step 1; R_r = period ratio determined from Table 2 of Ref. 22, which contains a completed set of data such as Fig. 9 presented here, for the design values of E_s, the wave reflection coefficient α, and the depth ratio H/H_s, where H is the depth of the impounded water, in feet. If $H/H_s < 0.5$, computation of R_r may be avoided by using $R_r \approx 1$.

3. Compute the period ratio R_w from

$$R_w = T_1^r / \tilde{T}_r \tag{3}$$

in which $\tilde{T}_r$ was computed in Step 2, and $T^r_1 = 4H/C$, where $C = 4720$ feet per second.

4. Compute $\tilde{T}_1$, the fundamental vibration period of the dam in seconds, including the influence of dam-foundation rock interaction and of impounded water, from

$$\tilde{T}_1 = R_r R_f T_1 \tag{4}$$

in which R_r was determined in Step 2; R_f = period ratio determined from Fig. 10 or Table 3 of Ref. 22, for the design value of E_f/E_s; and

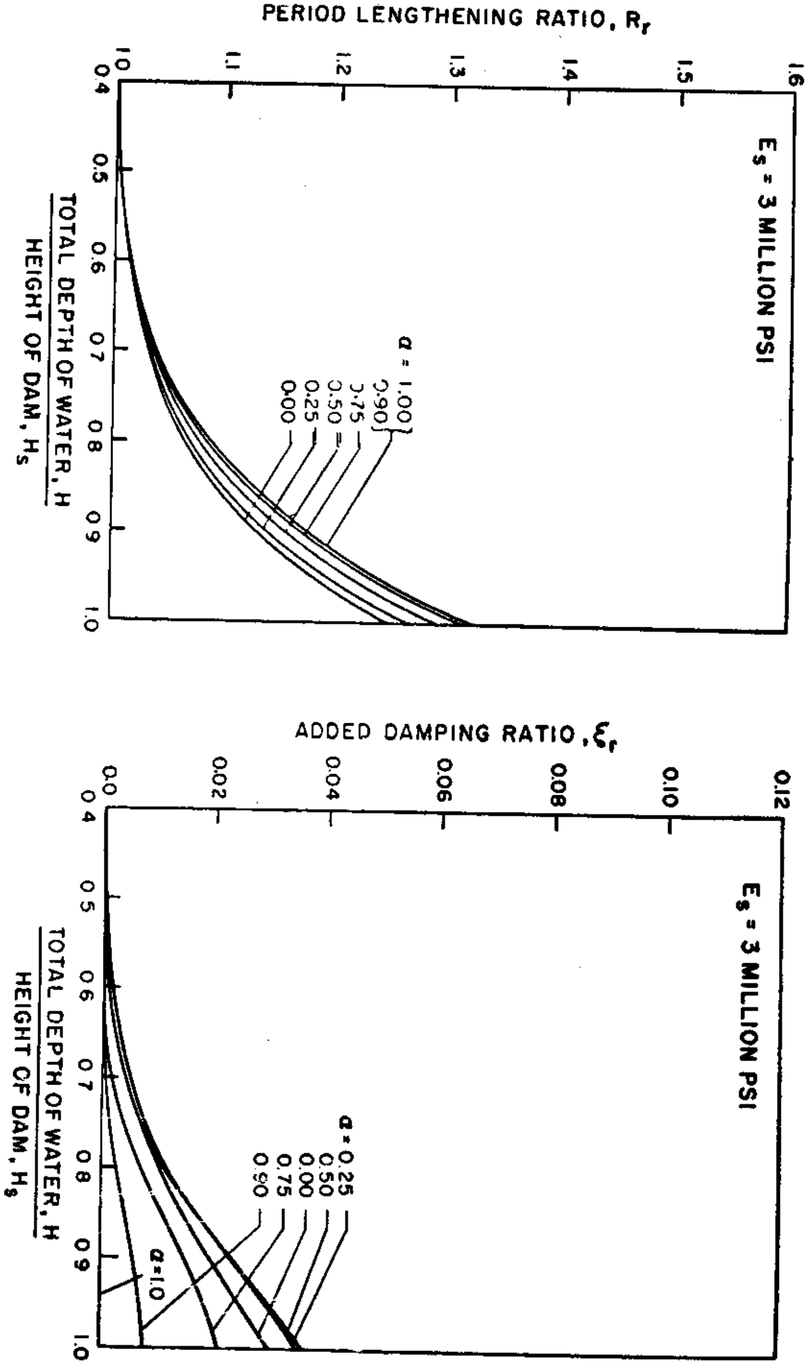


Fig. 9. Standard values for R_r, the period lengthening ratio, and ξ_r, the added damping ratio, due to hydrodynamic effects; E_s=3.0 million psi. (From Ref. 21)

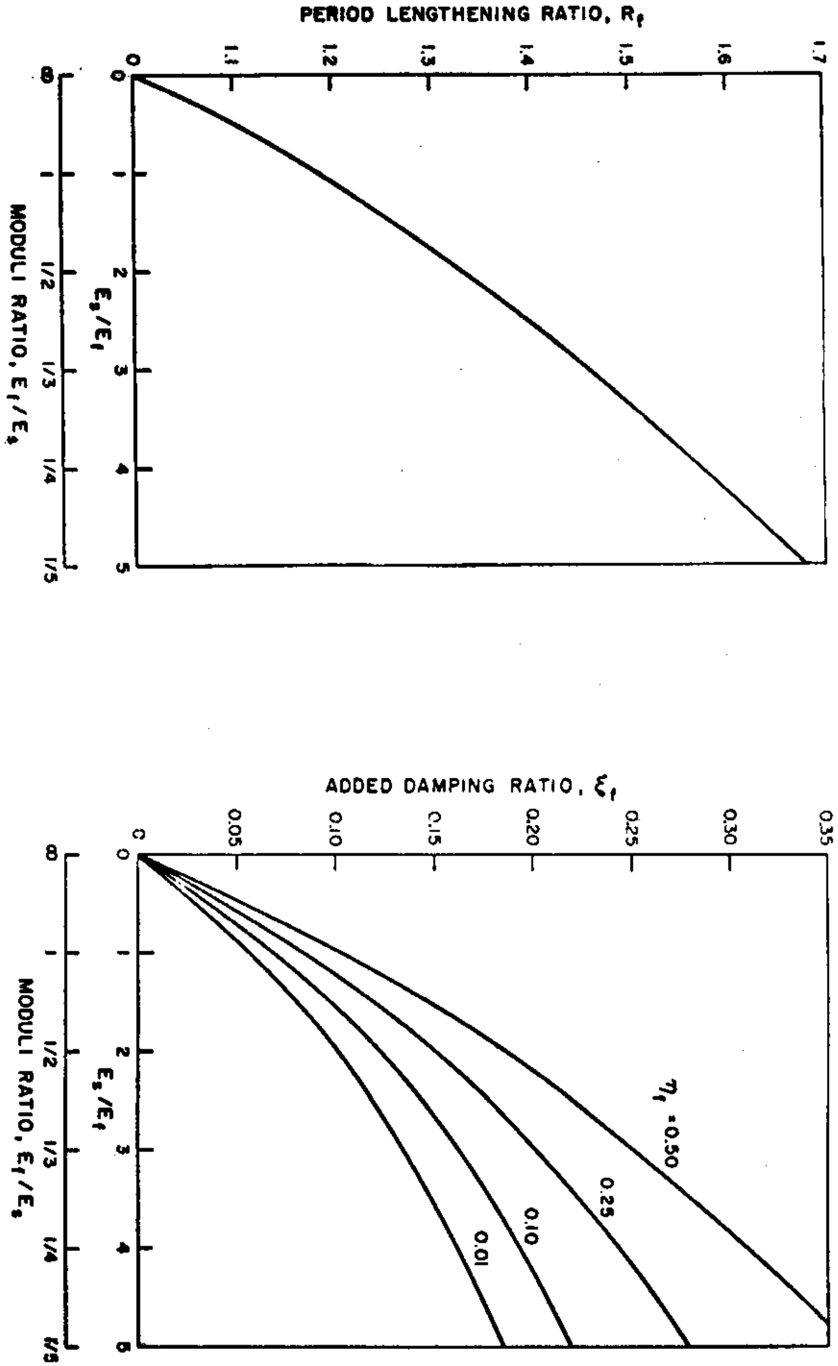


Fig. 10. Standard values for R_t, the period lengthening ratio, and ξ_t, the added damping ratio, due to dam-foundation rock interaction. (From Ref. 21)

E_f is the Young's modulus of the foundation rock in pounds per square inch. If $E_f/E_s > 4$, use $R_f \approx 1$.

5. Compute the damping ratio $\tilde{\xi}_1$ of the dam from

$$\tilde{\xi}_1 = \frac{1}{R_r} \frac{1}{(R_f)^3} \xi_1 + \xi_r + \xi_f \qquad (5)$$

using the period ratios R_r and R_f determined in Steps 2 and 4, respectively; ξ_1=viscous damping ratio for the dam on rigid foundation rock with empty reservoir; ξ_r = added damping ratio due to dam-water interaction and reservoir bottom absorption, obtained from Table 2 of Ref. 22, which contains a complete set of data such as Fig. 9 presented here, for the selected values of E_s, α, and H/H_s; and ξ_f = added damping ratio due to dam-foundation rock interaction, obtained from Fig. 10 or Table 3 of Ref. 22 for the selected values of E_f/E_s and η_f. If $H/H_s < 0.5$, use $\xi_r = 0$; if $E_f/E_s > 4$, use $\xi_f = 0$; and if the computed value of $\tilde{\xi}_1 < \xi_1$, use $\tilde{\xi}_1 = \xi_1$.

6. Determine gp $(y, \tilde{T}_r)$ from Table 4 of Ref. 22, which contains a complete set of data such as Fig. 11 presented here, corresponding to the value of R_w computed in Step 3—rounded to one of the two nearest available values, the one giving the larger p (y)—the design value of α, and for $H/H_s = 1$; the result is multiplied by $(H/H_s)^2$.

If $H/H_s < 0.5$, computation of p $(y, \tilde{T}r)$ may be avoided by using p $(y, \tilde{T}_r) \approx 0$.

7. Compute the generalized mass $\tilde{M}_1$ from

$$\tilde{M}_1 = (R_r)^2 \, M_1 \qquad (6)$$

in which R_r was computed in Step 2, and M_1 is computed from

$$M_1 = \frac{1}{g} \int_0^{H_s} w_s\,(y)\, \phi^2\,(y)\, dy \qquad (7)$$

in which w_s (y) = the weight of the dam per unit height; the fundamental vibration model shape ϕ (y) is given in Fig. 12 or Table 1 of Ref. 22; and g = 32.2 feet per squared second. Evaluation of Eq. 7 may be avoided by obtaining an approximate value from M_1 = 0.043

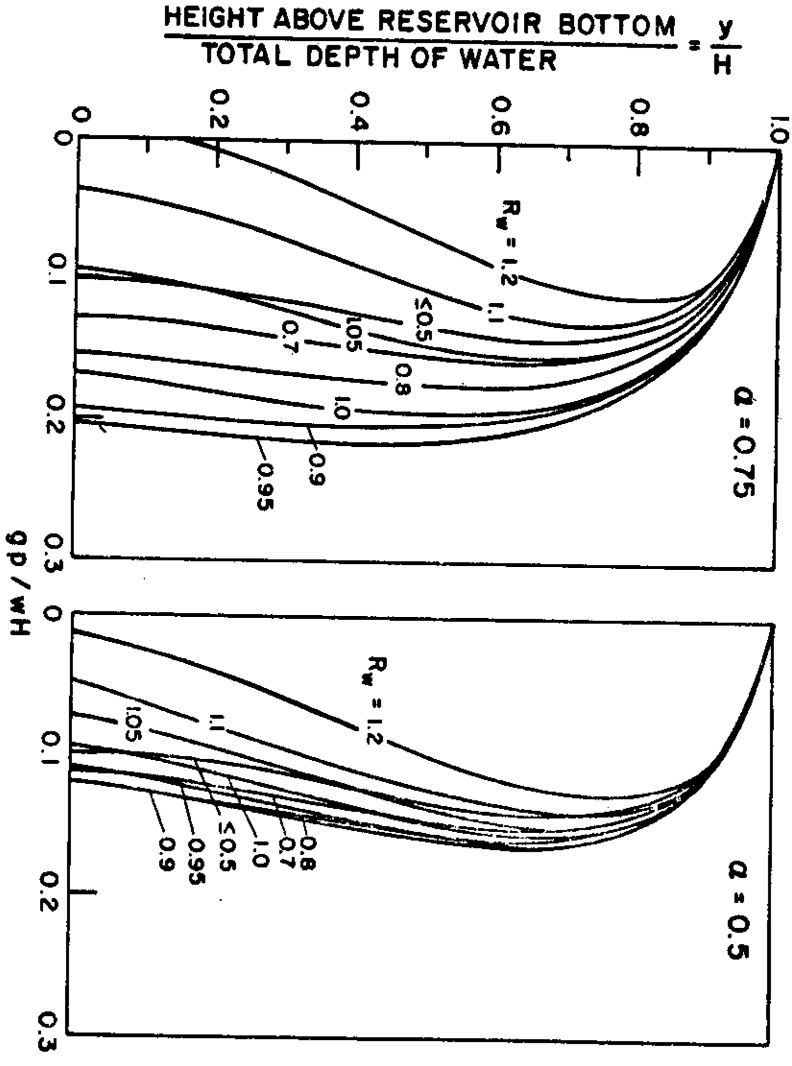
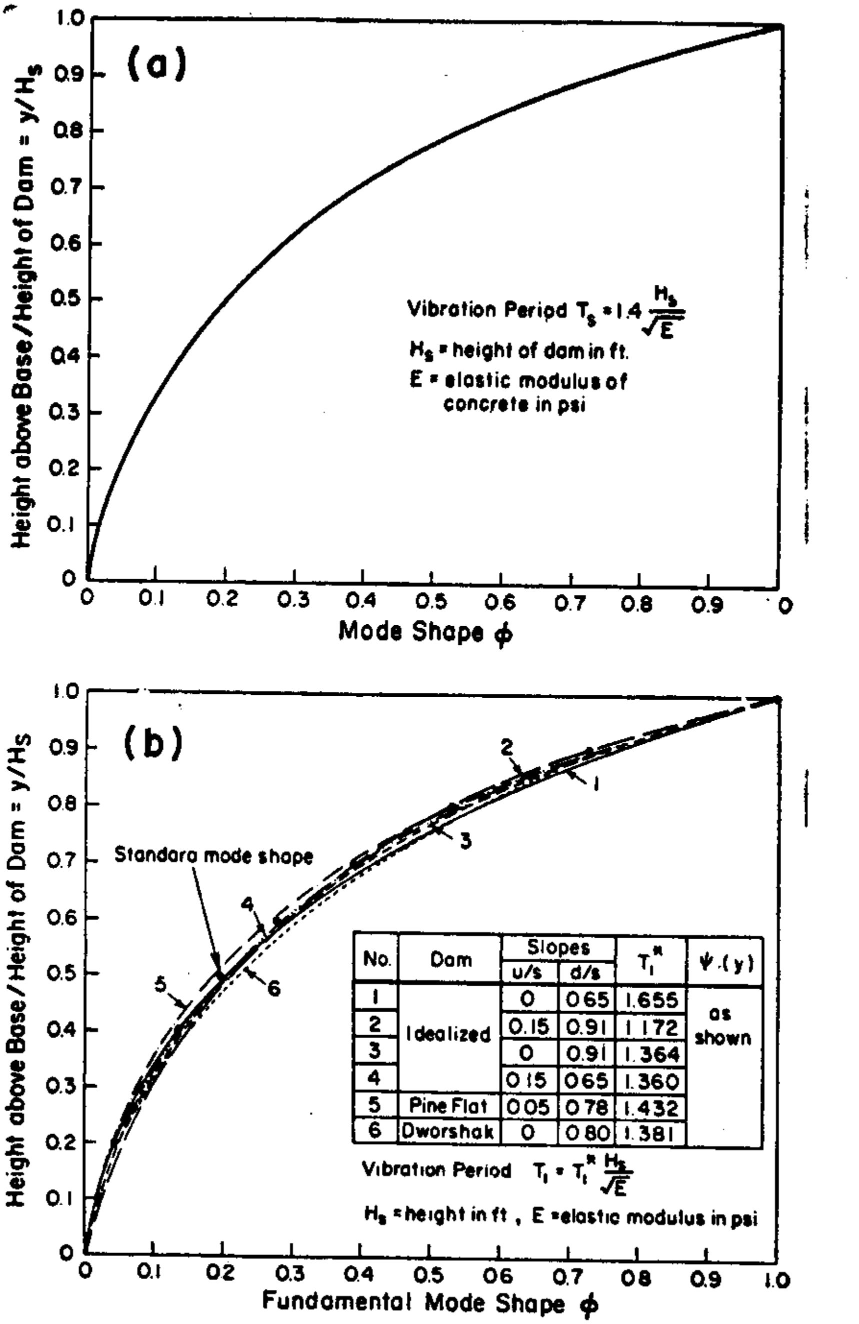


Fig. 11. Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e. $H/H_s=1$: $\alpha=0.75$ and 0.50. (From Ref. 21)



No.	Dam	Slopes u/s	Slopes d/s	T_1^*	$\psi_1(y)$
1	Idealized	0	0.65	1.655	as shown
2	Idealized	0.15	0.91	1.172	
3	Idealized	0	0.91	1.364	
4	Idealized	0.15	0.65	1.360	
5	Pine Flat	0.05	0.78	1.432	
6	Dworshak	0	0.80	1.381	

Fig. 12. Fundamental period and mode shape of vibration for concrete gravity dams. (a) Standard period and mode shape (b) Comparison of standard values with properties of six dams. (From Ref. 21).

W_s/g, where W_s is the total weight of the dam monolith.

8. Compute the generalized earthquake force coefficient $\tilde{L}_1$ from

$$\tilde{L}_1 = L_1 + \frac{1}{g} F_{st} \left[\frac{H}{H_s} \right]^2 A_p \tag{8}$$

in which L_1 is computed from

$$L_1 = \frac{1}{g} \int_0^{H_s} w_s(y)\, \phi(y)\, dy \tag{9}$$

$F_{st} = wH^2/2$; and A_p is given in Table 5 of Ref. 22 for the values of R_w and α used in Step 6. If $H/H_s < 0.5$, computation of $\tilde{L}_1$ may be avoided by using $\tilde{L}_1 \approx L_1$. Evaluation of Eq. 9 may be avoided by obtaining an approximate value from $L_1 = 0.13\ W_s/g$.

9. Compute $f_1(y)$, the equivalent lateral earthquake forces associated with the fundamental vibration mode from

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} \frac{S_a(\tilde{T}_1, \tilde{\xi}_1)}{g} \left[w_s(y)\, \phi(y) + gp(y, \tilde{T}_r) \right] \tag{10}$$

in which $S_a(\tilde{T}_1, \tilde{\xi}_1)$ = the pseudoacceleration ordinate of the earthquake design spectrum in feet per squared second at period $\tilde{T}_1$ determined in Step 4 and damping ratio $\tilde{\xi}_1$ determined in Step 5; $w_s(y)$ = weight per unit height of the dam; $\phi(y)$ = fundamental vibration mode shape of the dam from Fig 12 or Table 1 of Ref. 22; $\tilde{M}_1$ and $\tilde{L}_1$ = generalized mass and earthquake force coefficient determined in Steps 7 and 8, respectively; the hydrodynamic pressure term $gp(y, \tilde{T}_r)$ determined in Step 6; and g = 32.2 feet per squared second.

10. Determine the stresses throughout the dam by static analysis of the dam subjected to equivalent lateral forces $f_1(y)$, from Step 9, applied to the upstream face of the dam. Traditional procedures for design calculations may be used wherein the normal bending stresses σ_{y1}

across a horizontal section are computed by elementary formulas for stresses in beams. The maximum principal stresses at the upstream and downstream faces can be computed from the normal bending stresses σ_{y1} by an appropriate transformation (see Ref. 2, Figs. 4-2 and 4-3, pp. 41-42):

$$\sigma_1 = \sigma_{y1} \sec^2\theta + p_1 \tan^2\theta \tag{11}$$

where θ is the angle of the face with respect to the vertical. If no tail water is included in the analysis, the hydrodynamic pressure $p_1 = 0$ for the downstream face. At the upstream face the hydrodynamic pressure p_1 is given by the second term of Eq 10:

$$p_1(y) = \frac{\tilde{L}_1}{\tilde{M}} S_a(\tilde{T}_1, \tilde{\xi}_1)\, p(y, \tilde{T}_r) \tag{12}$$

which was computed in Step 9.

Part II The earthquake forces and stresses due to the higher vibration modes can be determined approximately for purposes of preliminary design by the following computational steps :

11. Compute, $f_{sc}(y)$, the lateral forces associated with the higher vibration modes, from

$$f_{sc}(y) = \frac{1}{g}\left\{w_s(y)\left[1 - \frac{L_1}{M_1}\phi(y)\right] + \left[gp_o(y) - \frac{B_1}{M_1} w_s(y)\,\phi(y)\right]\right\} a_g \tag{13}$$

in which M_1 and L_1 were determined in Steps 7 and 8, respectively; $gp_o(y)$ is determined from Fig. 13 or Table 6 of Ref. 22; B_1 is computed from

$$B_1 = 0.052\ \frac{F_{st}}{g}\left[\frac{H}{H_s}\right]^2 \tag{14}$$

and a_g is the maximum ground acceleration, in feet per squared second of the design earthquake. If $H/H_s < 0.5$, computation of $p_o(y)$ may be avoided by using $p_o(y) \approx 0$ and hence $B_1 \approx 0$.

12. Determine the stresses throughout the dam by static analysis of the dam subjected to the equivalent lateral forces $f_{sc}(y)$, from Step 11, applied to the upstream face of the dam. The normal bending stresses $\sigma_{y,sc}$ may be determined by the procedures mentioned in Step 10. The maximum principal stresses at the upstream and downstream forces

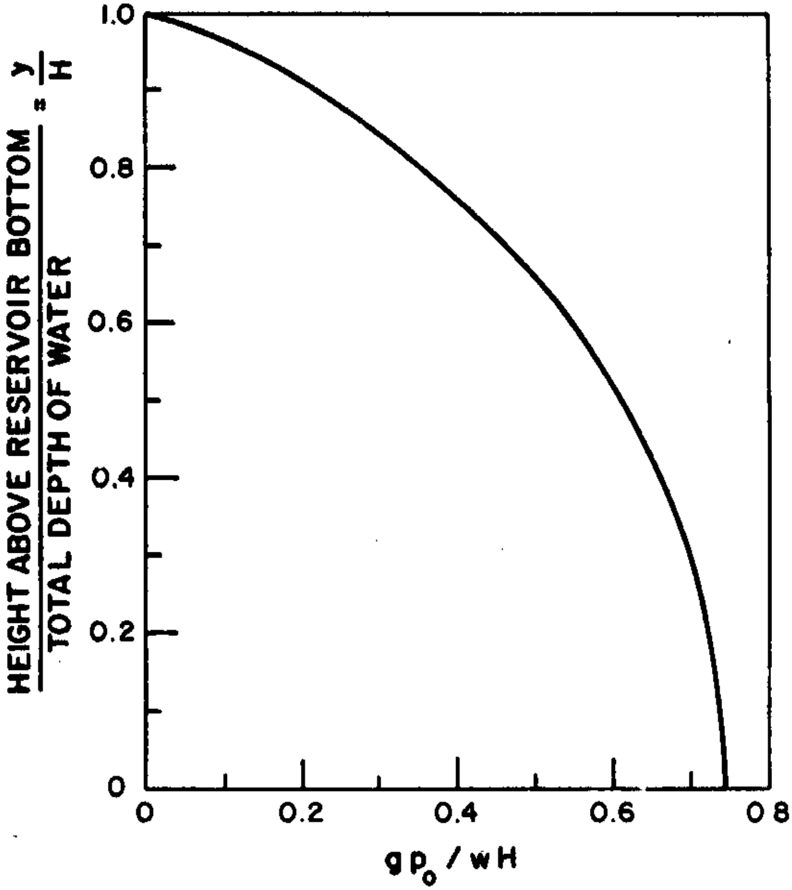


Fig. 13. Standard values for the hydrodynamic pressure function P_0 ($\hat{y}$) (From Ref. 21)

can be computed from the normal bending stresses $\sigma_{y,sc}$ by a transformation similar to Eq. 11 :

$$\sigma_{sc} = \sigma_{y,sc} \sec^2\theta + p_{sc} \tan^2\theta. \tag{15}$$

If no tail water is considered in the analysis, the hydrodynamic pressure $p_{sc} = 0$ for the downstream face. At the upstream face the hydrodynamic pressure p_{sc} is given by the second term of Eq. 13 :

$$p_{sc} (y) = \left[gp_o \ (y) - \frac{B_1}{M_1} w_s (y) \ \phi \ (y) \right] \frac{\bar{a}_g}{g} \tag{16}$$

which was computed in Step 11.

Part III : The initial stresses in the dam due to various loads, prior to the earthquake, including the self-weight of the dam, hydrostatic pressure, creep, construction sequence, and thermal effects are determined by the following computational step :

13. Compute first the normal stresses $\sigma_{y,st}$ across horizontal sections and then the maximum stresses from a transformation similar to Eqs. 11 and 13 :

$$\sigma_{st} = \sigma_{y,st} \sec^2\theta + p_{st} \tan^2\theta \qquad (17)$$

The hydrostatic pressure $p_{st} = w\,(H-y)$ on the upstream face and $p_{st} = 0$ on the downstream face if tail water is excluded.

Part IV : The total stresses in the dam are determined by the following computational steps:

14. Compute the dynamic response from the square-root-of-the-sum-of-squares (SRSS) combination rule

$$r_d = \sqrt{(r_1)^2 + (r_{sc})^2} \qquad (1)$$

in which r_1 and r_{sc} are values of the response quantity determined in Steps 10 and 12 associated with the fundamental and higher vibration modes, respectively.

15. Compute the total value of any response quantity by

$$r_{max} = r_{st} \pm \sqrt{(r_1)^2 + (r_{sc})^2} \qquad (19)$$

in which r_{st} is its initial value prior to the earthquake due to various loads, including the self-weight of the dam, hydrostatic pressure; and creep, construction sequence, and thermal effects.

The SRSS combination rule (Eq. 18) is applicable to the computation of any response quantity that is proportional to the generalized model coordinate responses. Thus, this combination rule is generally inappropriate to determine the principal stresses. However, the second term in Eqs. 11, 15, and 17 would be negligible for most gravity dams because the upstream face is usually almost vertical ($\theta = 0$) and the effects of tail water at the downstream face are small. Thus, the principal stresses in Eqs. 11. 15, and 17 become proportional to the corresponding normal bending stresses (and hence to the modal coordinates) with the same proportionality constant $\sec^2\theta$. In this situation the combination rules of Eqs 18 and 19 also apply to the principal stresses.

Use of Metric Units

Because the standard values for most quantities required in the simplified analysis procedure are presented in nondimensional form, implementation of the procedure in metric units is straightforward. The expressions and data requiring conversion to metric units are noted here :

1. The fundamental vibration period T_1 of the dam on rigid foundation rock with empty reservoir (Step 1), in seconds, is given by :

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_s}} \tag{20}$$

where H_s is the height of the dam in meters; and E_s is the Young's modulus of elasticity of the dam concrete in mega-Pascals.

2. The period ratio R_r and added damping ratio ξ_r due to dam-water interaction presented in Fig. 9 and Table 2 of Ref. 22 is for specified values of E_s in psi which should be converted to mega-Pascals as follows : 1 million psi=7 thousand mega-Pascals.

3. Where required in the calculations, the unit weight of water w=9.81 kilo-Newtons per cubic meter; the acceleration due to gravity g=9.81 meters per squared second; and velocity of pressure waves in water C=1440 meters per second.

EVALUATION OF SIMPLIFIED ANALYSIS PROCEDURE

Various approximations were introduced in developing the simplified analysis procedure and these were individually checked to ensure that they would lead to acceptable results.[22] In order to provide an overall evaluation of the simplified analysis procedure, it is used to determine the earthquake-induced stresses in Pine Flat Dam; and the results are compared with those obtained from a refined response history analysis-rigorously including dam-water-foundation rock interaction and reservoir bottom absorption effects-in which the dam is idealized as a finite element system. This material is taken from Ref. 21.

System and Ground Motion

The system properties for the simplified analysis are taken to be the same as those for the complete response history analysis : the tallest, nonoverflow monolith of the dam is shown in Fig. 14; height of the dam, H_s = 400 ft; modulus of elasticity of concrete, $E_s = 3.25 \times 10^6$ psi; unit weight of concrete = 155 pcf; damping ratio ξ = 5%; modulus of elasticity of foundation rock $E_f = 3.25 \times 10^6$ psi; constant hysteretic damping

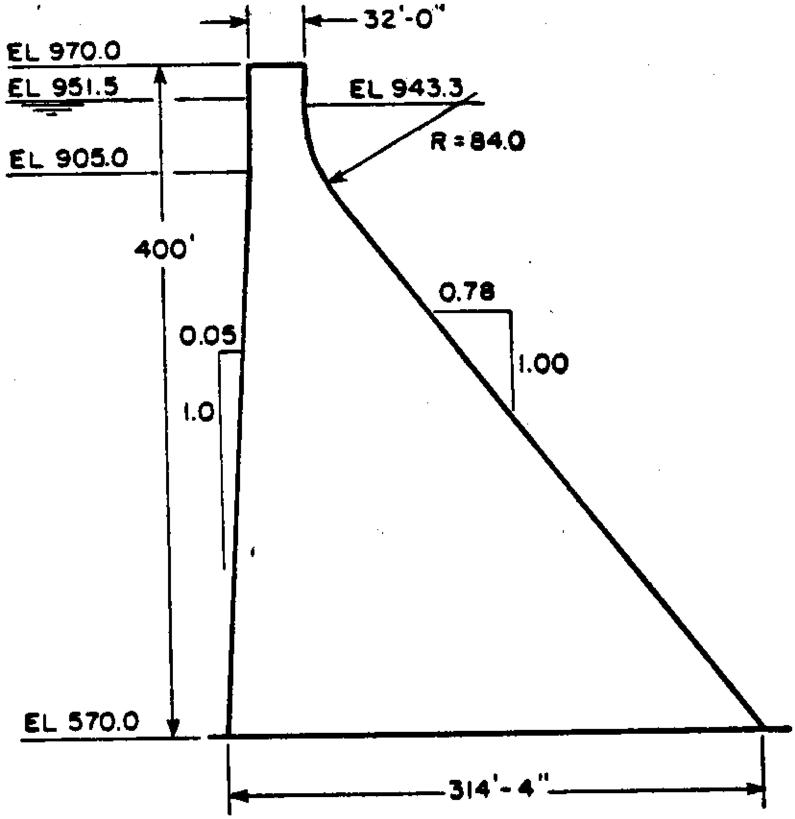


Fig. 14 Tallest, nonoverflow monolith of Pine Flat Dam. (From Ref 22)

coefficient of foundation rock, $\eta_f = 0.10$; depth of water, $H = 381$ ft; and, at the reservoir bottom, the wave reflection coefficient $\alpha = 0.5$.

The dam is analyzed for the Taft ground motion (S69E component) for which the response spectrum is shown in Fig. 15. Such an irregular spectrum of an individual ground motion is inappropriate in conjunction with the simplified procedure, wherein a smooth design spectrum is recommended, but is used here to provide direct comparison with the results obtained from the refined analysis procedure.

Computation of Earthquake Forces

The dam is analyized by the simplified analysis procedure for the four cases listed in Table 1. Implementation of the step-by step analysis procedure described in the preceding section is summarized next with additional details available in Ref. 22; all computations are performed for a unit width of the monolith :

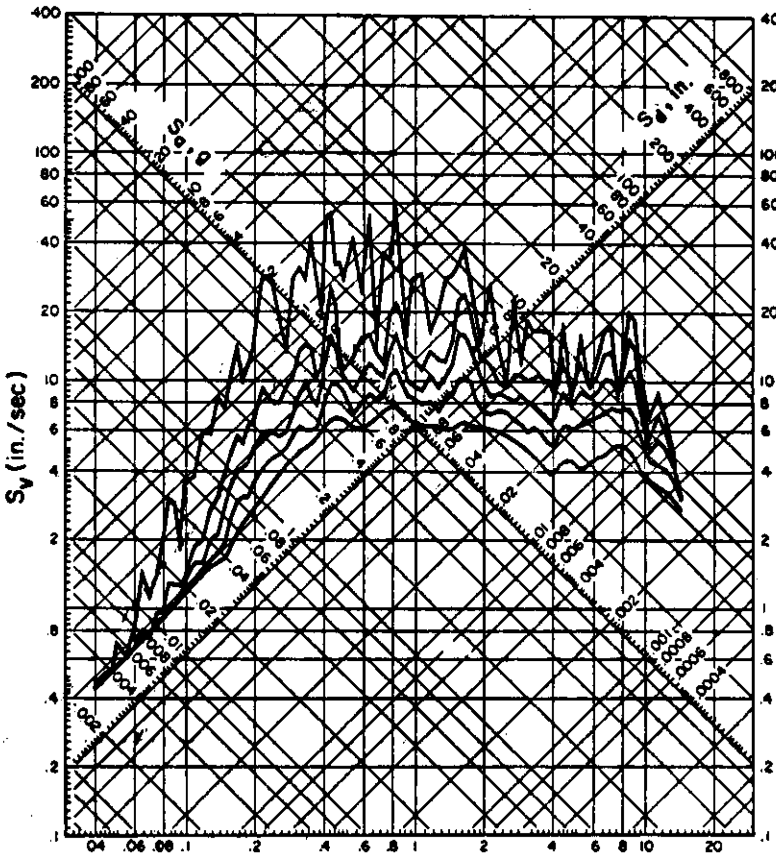


Fig. 15 Response spectrum for the S69E component of Taft ground motion; damping ratios=0, 2, 5, 10 and 20 percent. (From Ref. 22)

1. For $E_s = 3.25 \times 10^6$ psi and $H_s = 400$ ft, flom Eq. 1, $T_1 = (1.4)\ (400)/\sqrt{3.25 \times 10^6} = 0.311$ sec.
2. For $E_s = 3.0 \times 10^6$ psi (rounded down from 3.25×10^6 psi), $\alpha = 0.50$ and $H/H_s = 381/400 = 0.95$, Table 2 of Ref. 22 $R = 1.213$, so $\tilde{T}_r = (1.213)\ (0.311) = 0.377$ sec.
3. From Eq. 3, $T_1^r = (4)\ (381)/4720 = 0.323$ sec and $R_w = 323/0\ 377 = 0.86$.
4. For $E_f/E_s = 1$, Table 3 of Ref. 22 gives $R_f = 1.187$, so $\tilde{T}_1 = (1.187)\ (0.311) = 0.369$ sec for Case 3, and $\tilde{T}_1 = (1\ 187)\ (0.377) = 0.448$ sec for Case 4.

Table 1— Pine Flat Dam Analysis Cases

Simplified Procedure Parameters, and Fundamental Mode Properties from Simplified and Refined Analysis Procedure [From Ref. 21]

Case	Foundation Rock	Water	Parameters from Simplified Procedure				Fundamental Mode Properties				
							Vibration Period in seconds, $\tilde{T}_1$		Damping Ratio $\tilde{\xi}_1$		$S_a(\tilde{T}_1, \tilde{\xi}_1)$ in g's**
			R_r	R_f	ξ_r	ξ_f	Simplified Procedure	Refined Procedure*	Simplified Procedure	Refined Procedure*	
1	rigid	empty	1.0	1.0	0	0	0.311	0.317	.050	.050	0.429
2	rigid	full	1.213	1.0	0.030	0	0.377	0.386	.071	.076	0.312
3	flexible	empty	1.0	1.187	0	0.068	0.369	0.386	.098	.126	0.281
4	flexible	full	1.213	1.187	0.030	0.068	0.448	0.482	.123	.144	0.327

*From Ref. 16

**S_a value corresponding to $\tilde{T}_1$ and $\tilde{\xi}_1$ from simplified procedure.

5. For Cases 2 and 4, ξ_r=0.030 from Table 2 of Ref. 22 for $E_s=3.0\times10^6$ psi (rounded down from 3.25×10^6 psi), α=0 50, and HH_s=0.95. For Cases 3 and 4, ξ_f=0.068 from Table 3 of Ref. 22 for E_f/E_s=1 and η_f =0.10. With ξ_1=0.05, Eq. 5 gives: $\tilde{\xi}$ = (0.05)/(1.213) + 0.030= 0.071 for Case 2; $\tilde{\xi}_1$ (0.05)/(1.187)^3+ 0.068=0.098 for Case 3; and $\tilde{\xi}_1$= (0.05)/[(1.213)(1.187)^3] +0.030+0.068=0.123 for Case 4.

6. The values of gp(y) presented in Table 2 at eleven equally spaced levels were obtained from Table 4 of Ref. 22 for R_w=0.90 (by rounding R_w=0.86 from Step 3) and α=0.50, and multiplied by (0.0624) (381)^2 (0.95)=21.5 k/ft.

7. Evaluating Eq. 7 in discrete form gives M_1=(1/g) (500 kip). From Eq. 6, $\tilde{M}_1$=(1.213)^2 (1/g) (500)=(736 kip)/g.

8. Equation 9 in discrete form gives L_1=(1390 kip)/g. From Table 5 of Ref. 22, A_p=0.274 for R_w=0.90 and α=0.50. Equation 8 then gives $\tilde{L}_1$ = 1390/g+[(0.0624)(381)^2/2g] (0.95)^2 (0274) = (2510 kip)/g. Consequently, for Cases 1 and 3, L_1/M_1 = 1390/500 = 2.78, and for Cases 2 and 4, $\tilde{L}_1/\tilde{M}_1$ = 2510/736=3.41.

9. For each of the four cases, Eq. 10 was evaluated at eleven equally spaced intervals along the height of the dam, including the top and bottom, by substituting values for the quantities computed in the preceding steps, computing the weight of the dam per unit height $w_s(y)$ from the monolith dimensions (Fig. 14) and the unit weight of concrete, by substituting ϕ (y) from Fig. 12 or Table 1 of Ref. 22 and the S_a $(\tilde{T}_1, \tilde{\xi}_1)$ from Fig. 15 corresponding to the $\tilde{T}_1$ and $\tilde{\xi}_1$ obtained in Steps 4 and 5 (Table 1). The resulting equivalent lateral forces $f_1(y)$ are presented in Table 2 for each case.

10. The static stress analysis of the dam subjected to the equivalent lateral forces $f_1(y)$, from Step 9, applied to the upstream face of the dam is described in the next subsection, leading to response value r_1 at a particular location in the dam.

11. For each of the four cases, Eq. 11 was evaluated at eleven equally spaced intervals along the height of the dam, including the top and bottom, by substituting numerical values for the quantities computed in the preceding steps; obtaining gp_0 (y) from Fig. 13 or Table 6 of Ref. 22, which is presented in Table 2; using Eq. 12 to compute B_1=

Table 2 —Equivalent Lateral Earthquake Forces on Pine Flat Dam due to S69E Component of Taft Ground Motion [From Ref. 22]

y (ft)	w_s (k/ft)	ϕ	$w_s\phi$ (k/ft)	gp (k/ft)	gp_o (k/ft)	Lateral Forces, in kips per foot							
						Case 1		Case 2		Case 3		Case 4	
						f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}
400	4 96	1.00	4.96	0	0	5.92	—1.60	5.27	—1.98	3.87	—1.60	5.52	—1.98
360	5.18	0.73	3.80	1.68	3.67	4.53	—0.97	5.83	—0.59	2.97	—0.97	6.11	—0.59
320	8.19	0.53	4.35	2.96	7.45	5.19	—0.70	7.78	0.31	3.40	—0.70	8.15	0.31
280	12.7	0.39	4.95	3.39	10.3	5 91	—0.19	8.87	1.29	3.87	—0.19	9.30	1.29
240	17.8	0.28	4.98	3.50	12.5	5 94	0.71	9.02	2.58	3.89	0.71	9.46	2.58
200	23.0	0.20	4.60	3.43	14.1	5.49	1.84	8.54	4.03	3.60	1.84	8.95	4.03
160	28.1	0.13	3.65	3.24	15.5	4.35	3.23	7.34	5.74	2.85	3 23	7.69	5.74
120	33.3	0.084	2.80	3.09	16.4	3.34	4 59	6 27	7.33	2.19	4.59	6.57	7.33
80	38.4	0.047	1.80	2.85	17.1	2.15	6.01	4.95	8.96	1.41	6.01	5.19	8.96
40	43.6	0.021	0.92	2.68	17.5	1.10	7.39	3.83	10.5	0 72	7.39	4.01	10.5
0	48.7	0	0	2.51	17.6	0	8.77	2.67	11.9	0	8.77	2.80	11.9

0.052 [(0.0624) $(381)^2/2g$] $(0.95)^2$ = (212.5 kip)/g, leading to B_1/M_1 = 212.5/500 = 0.425; and substituting a_g = 0.18 g. The resulting equivalent lateral forces $f_{sc}(y)$ are presented in Table 2 for each case.

12. The static stress analysis of the dam subjected to the equivalent lateral forces f_{sc} (y), from Step 11, applied to the upstream face of the dam is described in the next subsection, leading to response value r_{sc} at a location in the dam.

13. The static stress analysis of the dam subjected to the initial forces prior to the earthquake leads to response value r_{st} at a particular location in the dam.

14. Compute the maximum total value of any response quantity by combining r_1 from Step 10, r_{sc} from Step 12, and r_{st} from Step 13, according to Eq. 19; this is described further in the next subsection.

Computation of Stresses

The equivalent lateral earthquake forces f_1 (y) and f_{sc} (y) respresenting the maximum effects of the fundamental and higher vibration modes, respectively, were computed in Steps 9 and 11. Dividing the dam into ten blocks of equal height,each of these sets of distributed forces is replaced by statically equivalent concentrated forces at the centroid of the blocks. Considering the dam monolith to be a cantilever beam, the bending stresses at the upstream and downstream faces of the monolith are computed at the bottom of each block using elementary formulas for stresses in beams. The normal bending stresses at the monolith faces are then transformed to principal stresses by Eqs. 11 and 15. In this simple stress analysis, the foundation rock is implicitly assumed to be rigid.

Because the upstream face of Pine Flat Dam is nearly vertical and the effects of tail water at the downstream face are negligible, as mentioned earlier and shown in Appendix C of Ref. 22. the principal stresses σ_1, and σ_{sc} at any location on the dam faces due to the forces f_1 (y) and f_{sc} (y), respectively, may be combined using the SRSS combination rule, Eq. 18. The combined values σ_d, along with the fundamental mode values σ_1 for the maximum principal stresses are presented in Table 3 and Figs. 16 and 17 for the four analysis cases. These stresses occur at the upstream face when the earthquake forces act in the downstream direction, and at the downstream face when the earthquake forces act in the upstream direction.

Table 3— Maximum Principal Stresses* (in psi) in Pine Flat Dam due to S69E Component of Taft Ground Motion [From Ref. 21]

Case	Foundation Rock	Water	Upstream Face			Downstream Face		
			Fundamental mode only	SRSS	Exact**	Fundamental Mode only	SRSS	Exact**
1	rigid	empty	241	247	223	333	338	272
2	rigid	full	263	266	261	411	413	277
3	flexible	empty	157	167	172	218	226	181
4	flexible	full	276	278	218	431	433	229

*Initial stresses are excluded.

**From Refined Procedure, Ref. 16.

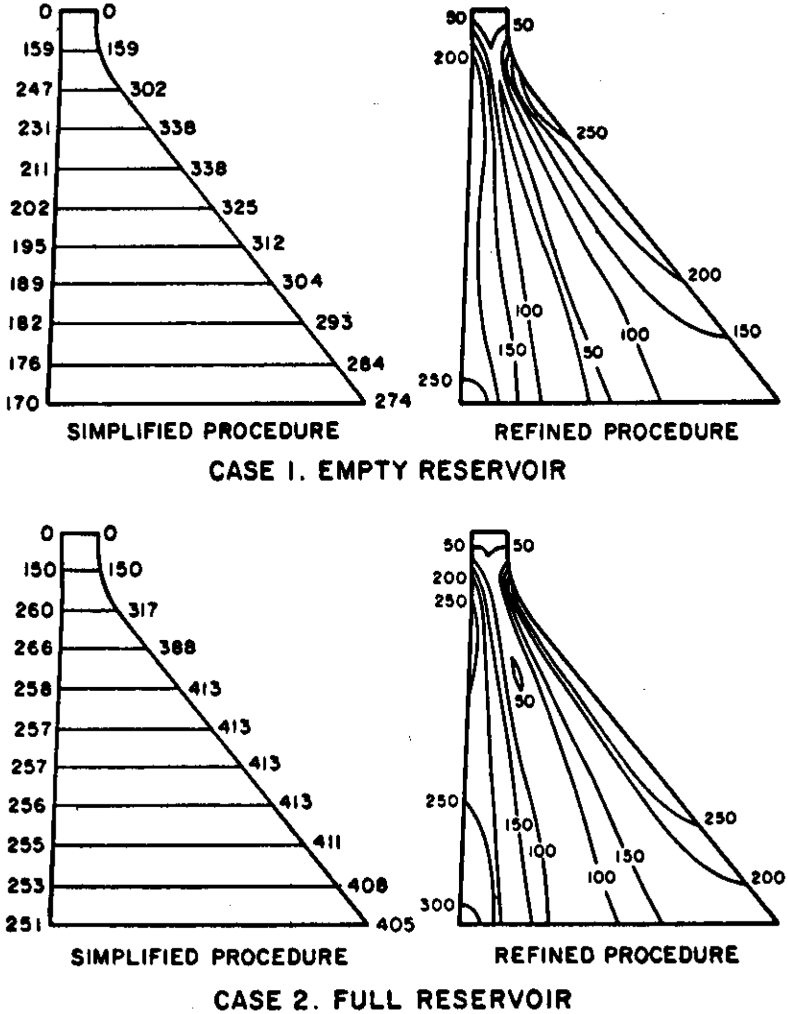
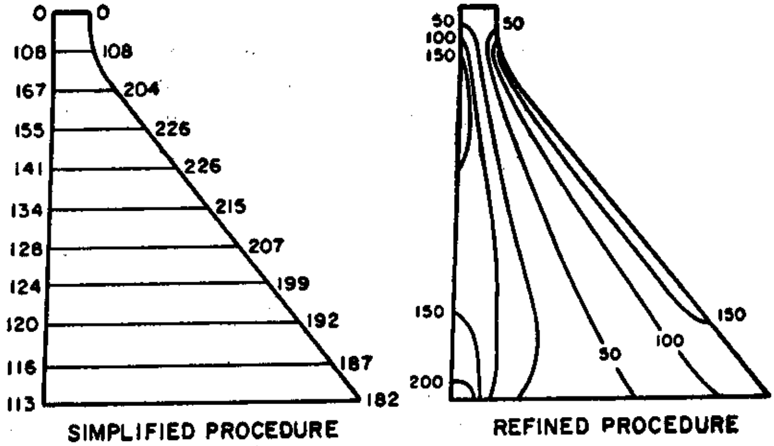


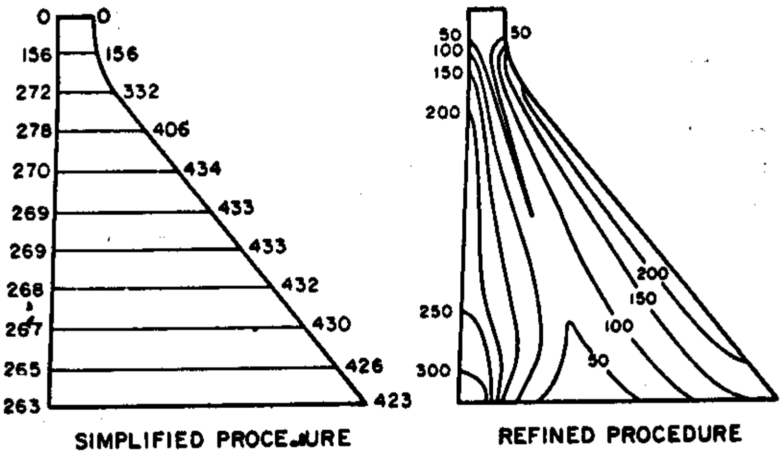
Fig 16 Maximum principal stresses (in psi) in Pine Flat Dam on rigid foundation rock due to S69E component of Taft ground motion; Cases 1 and 2 Initial static stresses are excluded. (From Ref. 21)

Comparison with Refined Analysis Procedure

The Pine Flat dam monolith was analyzed by the computer program EAGD-84 in which the response history of the dam due to Taft ground motion is computed. The dam is idealized as a finite-element system and the effects of dam-water-foundation rock interaction and of reservoir



CASE 3. EMPTY RESERVOIR



CASE 4. FULL RESERVOIR

Fig. 17. Maximum principal stresses (in psi) in Pine Flat Dam on flexible foundation rock due to S69E component of Taft ground motion; Cases 3 and 4. Initial static stresses are excluded (From Ref. 21)

bottom absorption are considered rigorously. The results from Ref. 15 at some intermediate steps of the analysis, in particular the resonant period and damping ratio of the fundamental response, are presented in Table 1, and the envelope values of the earthquake-induced maximum principal stresses are presented in Table 3 and in Figs. 16 and 17.

It is apparent from Table 1 that the simplified procedure leads to excellent estimates of the resonant vibration period and the damping ratio for the fundamental mode. Because the response of concrete gravity dams is dominated by the fundamental mode, this comparison provides a confirmation that the simplified procedure is able to represent the important effects of dam-water interaction, reservoir bottom absorption, and dam-foundation rock interaction.

As shown in Table 3, the stresses obtained by the simplified procedure considering only the fundamental vibration mode response are about the same those obtained by including higher mode responses in the SRSS combination rule. For the system parameters and excitation considered in this example, the higher mode responses add only 2 to 10 psi to the stresses, indicating that these can be neglected in this case and many other cases. However, in some cases, depending on the vibration periods and mode shapes of the dam and on the shape of the earthquake response spectrum, the higher mode responses may be more significant and should be included.

Considering only the fundamental mode response or obtaining the SRSS combination of fundamental and higher mode responses, the simplified procedure provides estimates of the maximum stress on the upstream face that are sufficiently accurate-in comparison with the "axact" results-to be useful in the preliminary design phase. The accuracy of these stresses depends, in part, on how well the resonant vibration period and damping ratio for the fundamental mode are estimated in the simplified procedure; e.g. in case 4 the maximum stress at the upstream face is overestimated by the simplified procedure primarily because it underestimates the damping ratio by about 2% (Table 1). While the simplified procedure provides excellent estimates of the maximum stress on the upstream face, at the same time it overestimates by 25 to 50% the maximum stress on the downstream face. This large discrepancy is due primarily to the limitations of elementary beam theory in predicting stresses near sloped faces. Similarly, the beam theory is incapable of reproducing the stress concentration in the heel area af dams predicted by the refined analysis (Figs. 16 and 17), and the stresses in that area are therefore underestimated.

From the comparison presented here for the earthquake-induced stresses, and in Ref. 22 for the total (static plus dynamic) stresses, it is apparent that the simplified procedure gives conservative, but reasonable, estimates of maximum stresses in dams, except in the heel area. The quality of the approximation is satisfactory for the preliminary phase in the design of new dams and in the safety evaluation of existing dams, considering the complicated effects of dam-water-foundation rock interaction and reservoir bottom absorption, the number of approximations necessary to develop the simplified analysis procedure, and noting that the results generally err on the conservative side.

SEISMIC DESIGN AND SAFETY EVALUATION

Earthquake-Resistant Design of New Dams[5]

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.

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[24] 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 spectum[25] with appropriate intensity is recommended, for it does not contain the irregularities of individual response spectra.

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. Except for the earthquake forces, all design loads should be defined as in traditional procedures.

For purposes of preliminary design, the earthquaka forces should be determined from the simplified analysis procedure presented in a preceding section 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 strength of concrete in compression and tension, respectively. 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 overturning and sliding stability criteria that have been used in standard design procedures in the past have little meaning in the context of oscillatory response of dams due to earthquakes; these should not be used until realistic criteria are 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.[26] 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.[26]

Because the tensile strength of concrete depends on the rate of loading, 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 Fig. 18 which presents four plots of tensile strength as a function of compressive strength, to be used depending on need. The lowest two plots, $f_t = 1.7 f_c^{2/3}$ and $f_t = 2.3 f_c^{2/3}$, 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, $f_t = 2.6 f_c^{2/3}$ and $f_t = 3.4 f_c^{2/3}$, 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.[27] 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 Lower Crystal Springs Dam-a curved concrete

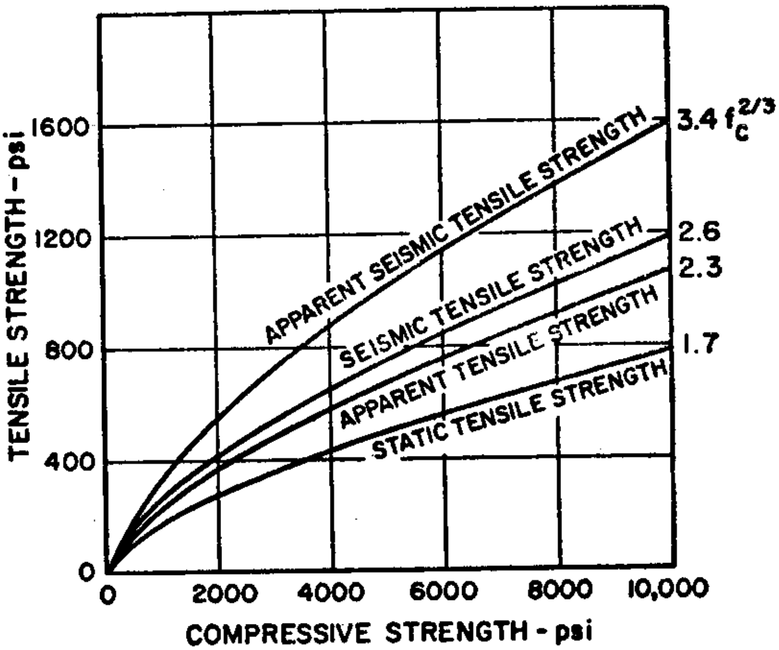


Fig. 18. Design Chart for Tensile Strength of Concrete. (From Ref. 26)

gravity dam located approximately 1,000 ft from the San Andreas fault during the great San Francisco earthquake of 1906 [28]

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,[18] the response of a preliminary design of the gravity 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 lecture, 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.[6,8]

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 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 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 discussed later in this lecture, 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.

Design Examples :— Many of the aforementioned design concepts which were initially proposed in 1978,[5] have been utilized in recent years to consider earthquake effects in the design of gravity dams. Two examples are mentioned for the reader's interest. The initial (1978) version of the simplified analysis procedure[5] as well as the EADHI computer program (a predecessor of the computer program EAGD-84 and the associated refined analysis procedure mentioned earlier in this lecture) were both utilized by the U S. Army Corps of Engineers in the design of the 170-ft high Richard B. Russell Dam on the Savannah River in Georgia;[29] construction of the dam was completed in 1982. The simplified analysis procedure presented in this chapter was utilized in the design of Pamo Dam, a 284 ft high roller-compacted-concrete dam under construction near San Diego, California.

Seismic Safety Evaluation of Existing Dams[6]

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 analyses procedures mentioned earlier should be employed. If such elastic analyses for the most intense ground shaking indicate tensile srresses much larger than the tensile strength of concrete, cracking of concrete can be expected. However, 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 lecture; a few of these are mentioned next.

The EADHI computer program, a predecessor of the EAGD-84 computer program, was utilized in the seismic safety evaluation of the Thermalito Diversion Dam, an investigation conducted in 1977 by the author for the California Division of Safety of Dams. This computer program was not considered to be an appropriate approach for stress analysis of the Thermalito Power Plant Headworks Structure. This 82-ft high structure had major openings in the upper part, and the lower part is dominated by large openings for penstocks. As a result, it was designed as a reinforced concrete structure in contrast to many gravity dams which are plain, unreinforced mass concrete structures. Consequently, it was not reasonable to analyze a two-dimensional slice of the structure. On the other hand, three-dimensional analysis which properly includes hydrodynamic effects was beyond the state-of-the-art at the time (1978). As a result, the following approach was employed. Firstly, the lateral forces were computed to represent the earthquake effects. These were computed by the initial (1978) version of the simplified procedure,[5] based on the overall dynamic properties of the structure, recognizing the openings and their complexities, and including the hydrodynamic effects. Secondly, the capacity of the existing structure to carry these lateral earthquake forces and other appropriate loads was evaluated. The reinforced concrete portions, as well as the plain mass concrete portions of the structure, were checked for these forces.

The Corps of Engineers is presently evaluating the seismic safety of Folsom Dam, which includes a 340 ft high concrete gravity structure near Sacramento, California. The simplified analysis procedure,[5,21] as well as the EAGD-84 computer program,[18] were employed in this investigation.

During the past few years Converse Consultants, San Francisco have utilized the simplified analysis procedure for the seismic safety evaluation of dams; e g. the 43-ft high Scots Flat Dam near Grass Valley, California. Because of the low stresses predicted by the simplified analysis, no further analysis by the refined response history analysis procedures was necessary.

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 linearl yelastic. Ignored in such analyses, which have been presented in this chapter, 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.[30] 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 Fig. 19 where the extent of cracking is seen to vary considerably with the tensile failure strain assumed for the concrete, a quantity that is not known accurately.

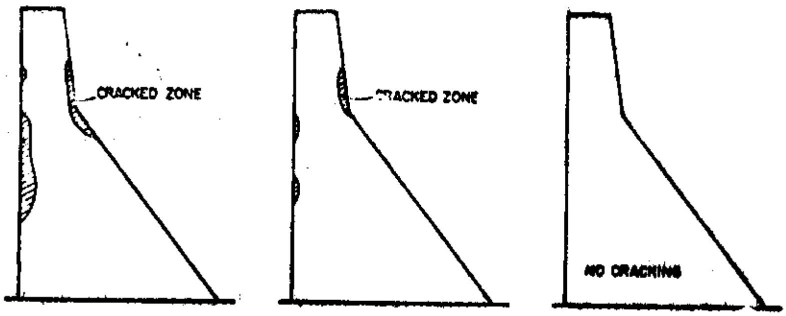


Fig. 19. Cracking in Koyna Dam predicted by nonlinear analyses for three values of of tensile failure strain. (From Ref. 30)

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.

Linear analyses of dam response to intense earthquake motions may 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 [31] Therefore, it may not be necessary to consider this nonlinear effect in practical applications.

Applications to Engineering Practice

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 twenty years, some of which have been summarized in this lecture. Static force methods involving seismic coefficients have given way to dynamic analysis procedures. As demonstrated earlier, 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 designs 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 h-ve 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 reservir 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 twenty years, 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. Several areas of needed research are identified in a recent report of the U.S. National Research Council.

Closing Comment

In closing, I wish to thank you for your attention and to record my gratitude to Indian Society of Earthquake Technology for inviting me to present the Annual Lecture for 1986.

ACKNOWLEDGEMENTS

The research studies that led to the material presented in this lecture were sponsored from 1969 to 1974 by the Office of the Chief of Engineers, Department of the Army, and since 1974 by the National Science Foundation. This support is gratefully acknowledged.

Grateful appreciation is also expressed to the author's students, Drs. P. Chakrabarti and G. Fenves, whose research provided the basis for much of the information presented in this lecture. In addition, some of the material is adapted from : (1) the author's paper published in the June, 1978 issue of the ASCE "Journal of the Structural Division" (Ref. 5); (2) his contribution to Chapter 8 (prepared by a panel chaired by A.S. Veletsos)

of the National Research Council Report, "Earthquake Engineeriug Research 1982" (Ref. 32); and (3) a chapter recently prepared for a handbook on dams (R. B. Jansen, editor) to be published by Van Nostrand Reinhold.

REFERENCES

1. U S. Bureau of Reclamation, "Design Criteria for Concrete Arch and Concrete Gravity Dams". Engineering Monograph No. 19. U.S. Government Printing Office, Washington, D.C., September 1974.
2. U.S. Bureau of Reclamation, "Design of Gravity Dams", U. S. Government Printing Office, Denver, Colorado, 1976.
3. U.S. Army Corps of Engineers, "Gravity Dam Design", Design Manual EM 1110-2-2200, Washington, D.C. September 1958.
4. U.S. Bureau of Reclamation, "Gravity Dams", Chapter 2 of "Design Standards No. 2 : Concrete Dams". Department of the Interior, Denver, Colorado, September 1966.
5. Chopra, A K., "Earthquake Resistant Design of Concrete Gravity Dams", Journal of the Structural Division, ASCE, Vol. 94, No. ST6, June 1978, pp. 953-971.
6. Chopra, A. K. and P. Chakrabarti, "The Koyna Earthquake and the Damage to Koyna Dam", Bulletin of the Seismological Society of America, Vol. 63, No. 2, April 1973, pp. 380-397.
7. International Conference of "Building Officials, Uniform Building Code, 1982.
8. Chopra, A.K. and P. Chakrabarti, "The Earthquake Experience at Koyna Dam and Stresses in Concrete Gravity Dams", Earthquake Engineering and Structural Dynamics, Vol. 1, No. 2, pp. 151-164.
9. Chopra, A.K., "Earthquake Analysis, Design and Safety Evaluation of Concrete Dams", to appear in the Proceedings of the Fifth Canadian Conference on Earthquake Engineering, Ottawa, July 1987.
10. Chopra, A K., "Earthquake Response of Concrete Gravity Dams", Journal of the Engineering Mechanics Division, ASCE, Vol 96, No. EM4, 1970.
11. Chopra, A. K., "Earthquake Behavior of Reservoir-Dam Systems", Journal of the Engineering Mechanics Division, ASCE, Vol. 94, No. EM6, December 1968. pp. 1475-1500.

12. Hall, J.F., "Study of the Earthquake Response of Pine Flat Dam", Earthquake Engineering and Structural Dynamics, Vol. 14, No. 2, March-April 1986, pp. 281-295.

13. Tarbox, G.S., K. Dreher and L. Carpenter, "Seismic Analysis of Concrete Dams", Transactions, 13th International Congress on Large Dams, New Delhi, October-November 1979, pp. 963-994.

14. Fenves, G. and A.K. Chopra, "Effects of Reservoir Bottom Absorption on Earthquake Response of Concrete Gravity Dams," Earthquake Engineering and Structural Dynamics, Vol. 11, No. 6, November-December 1983, pp. 803-829.

15. Fenves, G. and A.K. Chopra, "Effects of Reservoir Bottom Absorption and Dam-Water-Foundation Rock Interaction on Frequency Response Functions for Concrete Gravity Dams," Earthquake Engineering and Structural Dynamics, Vol. 13, No. 1, January-February 1985, pp. 13-31.

16. Fenves, G. and A.K. Chopra, "Reservoir Bottom Absorption Effects in Earthquake Response of Concrete Gravity Dams," Journal of Structural Engineering, ASCE, Vol. 111, No. 3, March 1985, pp. 545-562.

17. Fenves, G. and A.K. Chopra, "Earthquake Analysis of Concrete Gravity Dams Including Reservoir Bottom Absorption and Dam-Water-Foundation Rock Interaction," Earthquake Engineering and Structural Dynamics, Vol. 12, No. 5, September-October 1984, pp. 663-680.

18. Fenves, G. and A K. Chopra, "EAGD-84: A Computer Program for Earthquake Analysis of Concrete Gravity Dams," Report No. UCB/EERC-84/11, Earthquake Engineering Research Center, University of California, Berkeley, California, August, 1984.

19. Fok, K.L., J.F. Hall and A K. Chopra, "EACD-3D : A Computer Program for Three-Dimensional Earthquake Analysis of Concrete Dams" Report No. UCB/EERC-86/09, Earthquake Engineering Research Centre, University of California, Berkeley, California, July 1986, 144 pp.

20. Rea, D, C.Y. Liaw and A.K. Chopra, "Mathematical Models for the Dynamic Analysis of Concrete Gravity Dams," Earthquake Engineering and Structural Dynamics, Vol. 3, No. 3, January-March 1975, pp. 249-258.

21. Fenves, G. and A. K. Chopra, "Simplified Earthquake Analysis of Concrete Gravity Dams," Journal of Strucrural Engineering, ASCE, to appear in 1987.

22. Fenves, G. and A. K. Chopra, "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams," Report No. UCB/EERC-85/10, Earthquake Engineering Research Centre, University of California, Berkeley, California, June 1986, 142 pp.

23. Fenves, G. and A,K. Chopra, "Simplified Analysis of Concrete Gravity Dams : Combined Hydrodynamic and Foundation Interacion Effects'" Journal of Engineering Mechanics, ASCE, Vol. 111. No. 6 June 1985, pp 736-756.

24 Jennings, P.C., G.W. Housner and N.C. Tsai, "Simulated Earthquake Motions for Design Purposes," Proceedings of the Fourth World Conference on Earthquake Engineering, Santiago, Chile, Vol. 1, pp. 145-160.

25. Newmark, N.M., J.A. Blume and K.K. Kapur, "Seismic Design Spectra for Nuclear Power Plants," Journal of the Power Division, ASCE, Vol. 99, No. P02, November 1973, pp. 287-303.

26. Raphael, J.M., "Tensile Strength of Concrete," Title No. 81-17, ACI Journal, March-April 1984, pp. 158-165.

27. Swanson, A.A. and R.P. Sharma, "Effects of the 1971 San Fernando Earthquake on Pacoima Arch Dam," Transactions, 13th International Congress on Large Dams, New Delhi, India, October-November 1979, pp. 797-824.

28. Wulff, J.G. and R.C. Van Orden,"Evaluation of the Earthquake Stability of Lower Crystal Springs Dam," Transaction, 13th International Congress on Large Dams, New Delhi, India, October-November 1979, pp. 747-774.

29. Norman, C.D. and H.E. Stone, "Earthquake Analysis of the Gravity Dam Monoliths of the Richard B. Russell Dam, Technical Report SL-79-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, September 1979.

30. Pal, N., "Seismic Cracking of Concrete Gravity Dams," Journal of the Structural Division, ASCE, Vol. 102, No. ST9, September 1976, pp. 1827-1844.

31. Zienkiewicz, O. C., D. K. Paul and E. Hinton, "Cavitation in Fluid-Structure Respone (with Particular Reference to Dams under Earthquake Loading)," Earthquake Engineering and Structural Dynamics, Vol. 11, No. 4, July-August 1983, pp. 463-481

32. Committee on Earthquake Engineering Research, "Earthquake Engineering Research-1982," National Research Council, National Academy Press, Washington, D.C., 1982, 266 pp.