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SIMPLIFIED ANALYSIS FOR EARTHQUAKE RESISTANT DESIGN OF CONCRETE GRAVITY DAMS

by

GREGORY FENVES
ANIL K. CHOPRA

A Report on Research Conducted Under
Grants CEE-8120308 and CEE-8401439
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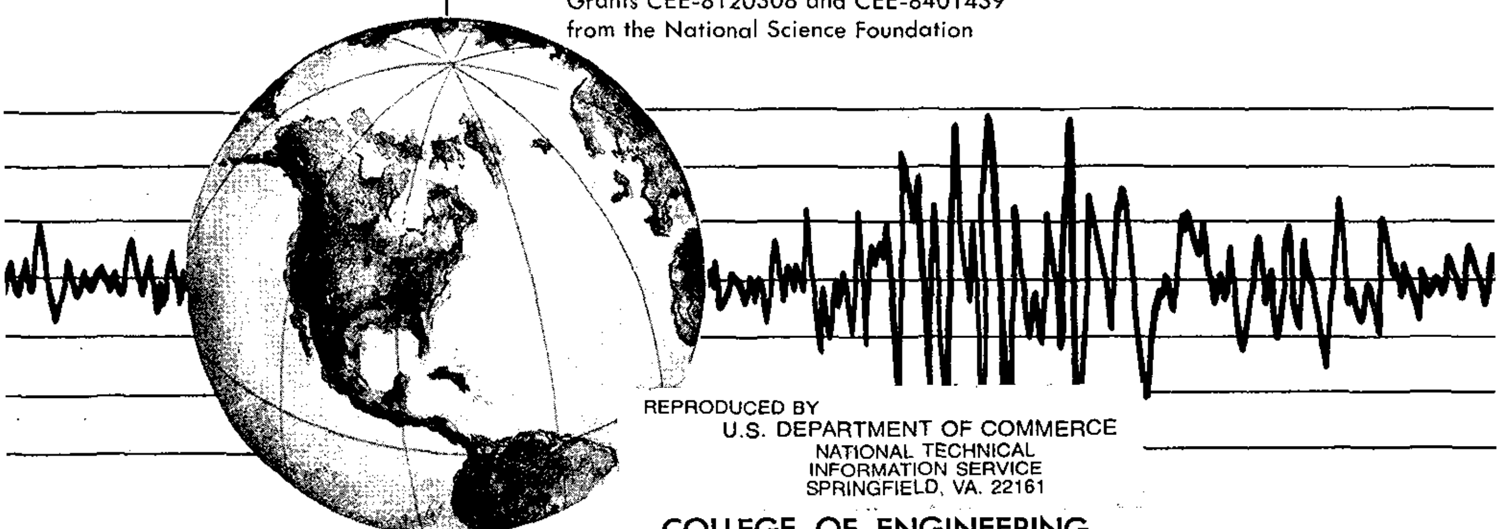

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A two-stage procedure was proposed in 1978 for the analysis phase of elastic design and safety evaluation of concrete gravity dams: (1) a simplified analysis procedure in which the response due only to the fundamental vibration mode is estimated directly from the earthquake design spectrum; and (2) a refined response history analysis procedure for finite element idealizations of the dam monolith. The former was recommended for the preliminary design and safety evaluation of dams, and the latter for accurately computing the dynamic response and checking the adequacy of the preliminary evaluation. In this report, the simplified analysis procedure has been extended to include the effects of dam-foundation rock interaction and of reservoir bottom sediments, in addition to the effects of dam-water interaction and water compressibility included in the earlier procedure. Also included now in the simplified procedure is a "static correction" method to consider the response contributions of the higher vibration modes. Thus, the design procedure proposed in 1978, which is still conceptually valid, should utilize the new simplified analysis procedure.

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Both writers are grateful to Hanchen Tan, graduate student at the University of California at Berkeley, for his contributions that led to Appendix A, and completion of the standard data presented in this report.

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INTRODUCTION

The earthquake analysis of concrete gravity dams has come a long way, progressing from traditional "static" methods for computing design forces to dynamic analysis procedures. With the aid of dynamic response analysis, considering the effects of interaction between the dam and impounded water and of water compressibility, it has been demonstrated that the traditional design procedures have serious limitations because they are based on unrealistic assumptions: rigid dam and incompressible water (3). In order to improve dam design procedures, a simplified version of the general dynamic analysis procedure was developed (3).

A two-stage procedure was proposed for the analysis phase of elastic design and safety evaluation of dams: the simplified analysis procedure in which the maximum response is estimated directly from the earthquake design spectrum; and a refined response history analysis procedure for finite element idealizations of the dam monolith. The former is recommended for the preliminary phase of design and safety evaluation of dams and the latter for accurately computing the dynamic response and checking the adequacy of the preliminary evaluation (3). Both procedures have been utilized in practical applications. At the time (1978) the design procedure was presented, both of these analysis procedures included the effects of dam-water interaction and compressibility of water.

More recently, the response history analysis procedure for two-dimensional finite element idealizations of gravity dam monoliths has been extended to also consider the absorption of hydrodynamic pressure waves into the alluvium and sediments deposited at the bottom of reservoirs, and the interaction between the dam and underlying flexible foundation rock. With the aid of response analysis utilizing EAGD-84 (9), the computer program that implements the extended procedure, these effects have been demonstrated to be significant (6, 7). Thus, this procedure and computer program supersede the earlier procedure for the final design and safety evaluation of dams.

The objective of this report is to extend the simplified analysis procedure (3) to include the effects of dam-foundation rock interaction and reservoir bottom materials so that these important effects can be considered also in the preliminary design phase. The simplified procedure involves computation of the lateral earthquake forces associated with the fundamental vibration mode of the dam. Utilizing the analytical development underlying the procedure (4, 5), this paper is concerned with implementation of the procedure. Recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely, standard data for the vibration properties of dams and the quantities that depend on them are presented to minimize the computations. Also included now in the simplified procedure is a "static correction" method to consider the response contributions of the higher vibration modes, and a rule for combining the modal responses. The use of the simplified procedure is illustrated by examples and is shown to be sufficiently accurate for the preliminary phase of design and safety evaluation of dams.

SIMPLIFIED ANALYSIS OF FUNDAMENTAL MODE RESPONSE

The dynamic response of short-vibration-period structures, such as concrete gravity dams, to earthquake ground motion is primarily due to the fundamental mode of vibration. Thus, this is the most important vibration mode that need be considered in a simplified analysis procedure. The dam response to the vertical ground motion is less significant relative to the horizontal ground motion (7), and is therefore not included in the simplified procedure.

Even the analysis of the fundamental mode response of a dam is very complicated because dam water-foundation rock interaction introduces frequency-dependent, complex-valued hydrodynamic and foundation terms in the governing equations. However, frequency-independent values for these terms can be defined and an equivalent single-degree-of-freedom (SDF) system developed to represent approximately the fundamental m ode response of concrete gravity dams (5).

For computing the response of the dam to intense earthquake ground motion it is appropriate to consider the two-dimensional vibration of individual monoliths. Each monolith is assumed to be supported on a viscoelastic half plane and impounding a reservoir of water, possibly with alluvium and sediments at the bottom (Fig. 1). Although the equivalent SDF system representation is valid for dams of any cross-section, the upstream face of the dam was assumed to be vertical (4,5) only for the purpose of evaluating the hydrodynamic terms in the governing equations. The standard data presented in this report is also based on this assumption, which is reasonable for actual concrete gravity dams because the upstream force is vertical or almost vertical for most of the height, and the hydrodynamic pressure 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 dynamic effects of the tail water are neglected because it is usually too shallow to influence dam response. A complete description of the dam-water-foundation rock system is presented in Refs. 4 and 8.

Equivalent Lateral Forces

Considering only the fundamental mode of vibration of the dam, the maximum effects of the horizontal earthquake ground motion can be represented by equivalent lateral forces acting on the dam (5):

$$f_1(x,y) = \frac{\tilde{L}_1}{\tilde{M}_1}\,\frac{S_a(\tilde{T}_1,\tilde{\xi}_1)}{g}\,[\,w_s(x,y)\,\phi_1^x(x,y) + g\bar{p}_1(y,\tilde{T}_r)\,\delta(x)] \tag{1}$$

In Eq. 1, the x-coordinate is along the breadth of the dam monolith, the y-coordinate is measured from the base of the dam along its height, $w_s(x,y) = gm_s(x,y)$ is the unit weight of the dam,

$$\tilde{M}_1 = M_1 + \mathrm{Re}\left[\int_0^H \bar{p}_1\,(y,\tilde{T}_r)\,\phi_1^x\,(0,y)\,dy\right] \tag{2a}$$

$$M_1 = \int\int m_s\,(x,y)\left\{[\phi_1^x\,(x,y)\,]^2 + [\,\phi_1^y\,(x,y)\,]^2\right\}dxdy \tag{2b}$$

$$\tilde{L}_1 = L_1 + \int_0^H \bar{p}_1\,(y,\tilde{T}_r)\,dy \tag{3a}$$

$$L_1 = \int\int m_s\,(x,y)\,\phi_1^x\,(x,y)\,dxdy \tag{3b}$$

M_1 is the generalized mass and L_1 the generalized earthquake force coefficient, with the integrations in Eqs. 2b and 3b extending over the cross-sectional area of the dam monolith; $\phi_1^x(x,y)$ and $\phi_1^y(x,y)$ are, respectively, the horizontal and vertical components of displacement in the fundamental vibration mode shape of the dam supported on rigid foundation rock with empty reservoir; $\bar{p}_1(y,\tilde{T}_r)$ is the complex-valued function representing the hydrodynamic pressure on the upstream face due to harmonic acceleration of period $\tilde{T}_r$ (defined later) in the fundamental vibration mode; H is the depth of the impounded water; $\delta(x)$ is the Dirac delta function, which indicates that the hydrodynamic pressure acts at the upstream face of the dam; g is the acceleration due to gravity; and $S_a(\tilde{T}_1,\tilde{\xi}_1)$ is the psuedo-acceleration ordinate of the earthquake design spectrum evaluated at the vibration period $\tilde{T}_1$ and damping ratio $\tilde{\xi}_1$ of the equivalent SDF system respresenting the dam-water-foundation rock system.

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on rigid foundation rock with impounded water is (4):

$$\tilde{T}_r = R_r\,T_1 \tag{4a}$$

in which T_1 is the fundamental vibration period of the dam on rigid foundation rock with empty reservoir. Because of the frequency-dependent, added hydrodynamic mass arising from dam-water interaction, the factor $R_r > 1$. It depends on the properties of the dam, the depth of the water, and the absorptiveness of the reservoir bottom materials. The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on flexible foundation rock with empty reservoir is (4):

$$\tilde{T}_f = R_f\,T_1 \tag{4b}$$

Because of the frequency-dependent, added foundation-rock flexibility arising from dam-foundation rock interaction, the factor $R_f > 1$. It depends on the properties of the dam and foundation rock.

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on flexible foundation rock with impounded water is approximately given by (5):

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

The damping ratio of this equivalent SDF system is (5):

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

in which ξ_1 is the damping ratio of the dam on rigid foundation rock with empty reservoir; ξ_r represents the added damping due to dam-water interaction and reservoir bottom absorption; and ξ_f represents the added radiation and material damping due to dam-foundation rock interaction. Considering that R_r and $R_f > 1$, Eq. 5 shows that dam-water interaction and dam-foundation rock interaction reduce the effectiveness of structural damping. However, usually this reduction is more than compensated by the added damping due to reservoir bottom absorption and due to dam-foundation rock interaction, which leads to an increase in the overall damping of the dam.

It is important to note that the effects of dam-water interaction and dam-foundation rock interaction on the parameters of the equivalent SDF system -- natural vibration period, damping ratio, generalized mass and earthquake force coefficient -- are computed independently of each other and applied sequentially to give values for the parameters that include the simultaneous interaction effects. The ability to separate the interaction effects in the computation of the natural vibration period and generalized mass is a consequence of the fact that dam-foundation rock interaction has little influence on the added hydrodynamic mass, and dam-water interaction does not substantially alter the effects of foundation rock flexibility. Such separation of the interaction effects is less accurate in the computation of the overall damping ratio and earthquake force coefficient by the simplified expressions of Eqs. 5 and 3a, but the results are acceptable for the preliminary phase of design and safety evaluation of dams. The separate consideration of dam-water interaction effects and dam-foundation rock interaction effects is an important feature of the simplified analysis procedure in that it greatly simplifies the evaluation of the fundamental vibration mode response of dams on flexible foundation rock with impounded water.

The quantities R_r, R_f, ξ_r, ξ_f, $\bar{p}_1(y,\tilde{T}_r)$, $\tilde{L}_1$, and $\tilde{M}_1$, which are required to evaluate the equivalent lateral forces, Eq. 1, contain all the modifications of the vibration properties of the equivalent SDF system and of the generalized earthquake force coefficient necessary to account for the effects of dam-water interaction, reservoir bottom absorption, and dam-foundation rock interaction. Even after the considerable simplification necessary to arrive at Eq. 1, its evaluation is still too complicated for practical applications because the afore-mentioned quantities are complicated functions of the hydrodynamic and foundation-rock flexibility terms (5). Fortunately, as will be seen in a later section, the computation of lateral forces can be considerably simplified by recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely.

One-Dimensional Approximation of Lateral Forces

The lateral forces $f_1(x,y)$ due to the fundamental vibration mode, Eq. 1, are distributed over the cross-section of the dam monolith. Because the variation of the fundamental vibration mode shape $\phi_1^x(x,y)$ across the dam breadth is small, i.e. $\phi_1^x(x,y) \approx \phi_1^x(0,y)$, it would be reasonable in a simplified analysis to integrate $f_1(x,y)$ over the breadth of the monolith to obtain equivalent lateral forces $f_1(y)$ per unit height of the dam:

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} \frac{S_a(\tilde{T}_1,\tilde{\xi}_1)}{g} \left[w_s(y)\, \phi(y) + g\bar{p}_1(y,\tilde{T}_r) \right] \tag{6}$$

in which $\phi(y) = \phi_1^x(0,y)$ is the horizontal component of displacement at the upstream face in the fundamental vibration mode shape of the dam; $w_s(y)$ is the weight of the dam per unit height; and all other quantities are as defined before except that the generalized mass and earthquake force coefficient, Eqs. 2b and 3b, are now represented by one-dimensional integrals:

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

$$L_1 = \frac{1}{g} \int_0^{H_s} w_s(y)\, \phi(y)\, dy \tag{7b}$$

in which H_s is the height of the dam. Eq. 6 is an extension of Eq. 9 in Ref. 3 to include the effects of dam-foundation rock interaction and reservoir bottom materials on the lateral forces.

Approximation of Hydrodynamic Pressure

If the hydrodynamic wave absorption in the alluvium and sediments at the bottom of a reservoir is considered, the hydrodynamic pressure function $\bar{p}_1(y,\tilde{T}_r)$ is complex-valued at the period $\tilde{T}_r$. The distributions of the real- and imaginary-valued components of lateral forces $f_1(y)$ at the upstream face of a typical concrete dam monolith with nearly full reservoir ($H/H_s = 0.95$) are shown in Fig. 2. The hydrodynamic pressure and hence lateral forces are real-valued for nonabsorptive reservoir bottom materials, i.e. $\alpha = 1$ (3). Even when reservoir bottom absorption is considered, the imaginary-valued component of lateral forces is small relative to the real-valued component, increasing near the base of the dam, where, because of the large stiffness of the dam, it will have little influence on the stresses in the dam. Consequently, the imaginary-valued component of $\bar{p}_1(y,\tilde{T}_r)$ may be neglected in the evaluation of the lateral forces $f_1(y)$, and Eq. 6 becomes:

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} \frac{S_a(\tilde{T}_1,\tilde{\xi}_1)}{g} \left[w_s(y)\,\phi(y) + gp(y,\tilde{T}_r) \right] \tag{8}$$

where $p(y,\tilde{T}_r) \equiv \mathrm{Re}[\bar{p}_1(y,\tilde{T}_r)]$. Although the imaginary-valued component of $\bar{p}_1(y,\tilde{T}_r)$ has been dropped in Eq. 8, its more important effect, the contribution to added hydrodynamic damping ξ_r in Eq. 5, is still considered.

The generalized mass $\tilde{M}_1$ of the equivalent SDF system, Eq. 2a, only depends on the real-valued component of hydrodynamic pressure:

$$\tilde{M}_1 = M_1 + \int_0^H p(y,\tilde{T}_r)\,\phi(y)\,dy \tag{9a}$$

where M_1 is defined in Eq. 7a. However, the generalized earthquake force coefficient $\tilde{L}_1$, Eq. 3a, contains both real-valued and imaginary-valued components of the hydrodynamic pressure. Again, dropping the imaginary-valued component gives:

$$\tilde{L}_1 = L_1 + \int_0^H p(y,\tilde{T}_r)\,dy \tag{9b}$$

where L_1 is defined in Eq. 7b.

STANDARD PROPERTIES FOR FUNDAMENTAL MODE RESPONSE

Direct evaluation of Eq. 8 would require complicated computation of several quantities: $p(y,\tilde{T}_r)$ from an infinite series expression; the period lengthening ratios R_r and R_f due to dam-water and dam-foundation rock interactions by iterative solution of equations involving frequency-dependent terms; damping ratios ξ_f and ξ_r from expressions involving complicated foundation-rock flexibility and hydrodynamic terms; the integrals in Eq. 9; and the fundamental vibration period and mode shape of the dam (4, 5). The required computations would be excessive for purposes of preliminary design of dams. Recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely, standard values for the vibration properties of dams and all quantities in Eq. 8 that depend on them are developed in this section. Tables and curves of the standard values are presented.

Vibration Properties of the Dam

Computed by the finite element method, the fundamental vibration period, in seconds, of a "standard" cross-section for nonoverflow monoliths of concrete gravity dams on rigid foundation rock with empty reservoir is (3):

$$T_1 = 1.4 \frac{H_s}{\sqrt{E_s}} \tag{10}$$

in which H_s is the height of the dam, in feet; and E_s is the Young's modulus of elasticity of concrete, in pounds per square inch. The fundamental vibration mode shape $\phi(y)$ of the standard cross-section is shown in Fig. 3a, which is compared in Fig. 3b with the mode shape for four idealized cross-sections and two actual dams. Because the fundamental vibration periods and mode shapes for these cross-sections are similar, it is appropriate to use the standard vibration period and mode shape presented in Fig. 3a for the preliminary design and safety evaluation of concrete gravity dams. The ordinates of the standard vibration mode shape are presented in Table 1.

Modification of Period and Damping: Dam-Water Interaction

Dam-water interaction and reservoir bottom absorption modify the natural vibration period (Eq. 4a) and the damping ratio (Eq. 5) of the equivalent SDF system representing the fundamental vibration mode response of the dam. For the standard dam cross-section, the period lengthening ratio R_r and added damping ξ_r depend on several parameters, the more significant of which are: Young's modulus E_s of the dam concrete, ratio H/H_s of water depth to dam height, and wave reflection coefficient α. This coefficient, α, is the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom (4, 8, 10, 11); $\alpha = 1$ indicates that pressure waves are completely reflected, and smaller values of α indicate increasingly absorptive materials.

The results of many analyses of the "standard" dam cross-section, using the procedures developed in Ref. 4 and modified in Appendix A for dams with larger elastic modulus E_s, are summarized in Figs. 4 and 5 and Table 2. The period lengthening ratio R_r and added damping ξ_r are presented as a function of H/H_s for E_s = 5.0, 4.5, 4.0, 3.5, 3.0, 2.5, 2.0, and 1.0 million psi; and α = 1.00, 0.90, 0.75, 0.50, 0.25, and 0. Whereas the dependence of R_r and ξ_r on E_s, H/H_s and α, and the underlying mechanics of dam-water interaction and reservoir bottom absorption are discussed elsewhere in detail (4, 5), it is useful to note that R_r increases and ξ_r generally, but not always, increases with increasing water depth, absorptiveness of reservoir bottom materials, and concrete modulus; also see Appendix A. The effects of dam-water interaction and reservoir bottom absorption may be neglected, and the dam analyzed as if there is no impounded water, if the reservoir depth is not large, $H/H_s < 0.5$; in particular $R_r \approx 1$ and $\xi_r \approx 0$.

Modification of Period and Damping: Dam-Foundation Rock Interaction

Dam-foundation rock interaction modifies the natural vibration period (Eq. 4b) and added damping ratio (Eq. 5) of the equivalent SDF system representing the fundamental vibration mode response of the dam. For the "standard" dam cross-section, the period lengthening ratio R_f and the added damping ξ_f due to dam-foundation rock interaction depend on several parameters, the more significant of which are: moduli ratio E_f/E_s, where E_s and E_f are the Young's moduli of the dam concrete and foundation rock, respectively; and the constant hysteretic damping factor η_f for the foundation rock. The period ratio R_f is, however, insensitive to η_f.

The results of many analyses of the "standard" dam cross-section, using the procedures developed in Ref. 4, are summarized in Figs. 6 and 7 and Table 3. The period lengthening ratio R_f and added damping ξ_f are presented for many values of E_f/E_s between 0.2 and 5.0, and η_f = 0.01, 0.10, 0.25, and 0.50. Whereas the dependence of R_f and ξ_f on E_f/E_s and η_f, and the underlying mechanics of dam-foundation rock interaction are discussed elsewhere in detail (4, 5), it is useful to note that the period ratio R_f is essentially independent of η_f, but increases as the moduli ratio E_f/E_s decreases, which for a fixed value of E_s implies an increasingly flexible foundation rock. The added damping ξ_f increases with decreasing E_f/E_s and increasing hysteretic damping factor η_f. The foundation rock may be considered rigid in the simplified analysis if $E_f/E_s > 4$ because then the effects of dam-foundation rock interaction are negligible.

Hydrodynamic Pressure

In order to provide a convenient means for determining $p(y,\tilde{T}_r)$ in Eqs. 8 and 9, a nondimensional form of this function $gp(\hat{y})/wH$, where $\hat{y} = y/H$, and w = the unit weight of water, has been computed from the equations presented in Ref. 4 for α = 1.00, 0.90, 0.75, 0.5, 0.25, and 0, and the necessary range of values of

$$R_w = \frac{T_1^r}{\tilde{T}_r} \tag{11}$$

in which the fundamental vibration period of the impounded water $T_1^r = 4H/C$, where C is the velocity of pressure waves in water. The results presented in Fig. 8 and Table 4 are for full reservoir, $H/H_s = 1$. The function $gp(\hat{y})/wH$ for any other value of H/H_s is approximately equal to $(H/H_s)^2$ times the function for $H/H_s = 1$ (3).

For nonabsorptive reservoir bottom materials ($\alpha = 1$), Fig. 8 shows that the function $p(\hat{y})$ increases monotonically with increasing period ratio R_w. Because dam-water interaction always lengthens the fundamental vibration period $\tilde{T}_r$ of the dam to a value greater than T_1^r, the period ratio R_w cannot exceed unity (10). For $R_w \leq 0.5$, the effects of water compressibility are negligible and $p(\hat{y})$ is essentially independent of R_w.

Figure 8 shows that $p(\hat{y})$ decreases because of the absorption of hydrodynamic pressure waves into the reservoir bottom materials ($\alpha < 1$). The period ratio R_w can exceed unity if $\alpha < 1$, but $p(\hat{y})$ decreases as R_w increases because of the increasingly stiff dam. The upper limit for R_w of 1.2 is the largest value that can result from the data presented in Fig. 4 and Table 2.

Generalized Mass and Earthquake Force Coefficient

The generalized mass $\tilde{M}_1$ (Eq. 9a) of the equivalent SDF system representing the dam, including hydrodynamic effects, can be conveniently computed from (4):

$$\tilde{M}_1 = (R_r)^2 M_1 \tag{12a}$$

in which M_1 is given by Eq. 7a and standard values are presented later. In order to provide a convenient means to compute the generalized earthquake force coefficient $\tilde{L}_1$, Eq. 9b is expressed as:

$$\tilde{L}_1 = L_1 + \frac{1}{g} F_{st} \left[\frac{H}{H_s} \right]^2 A_p \tag{12b}$$

where $F_{st} = \frac{1}{2} wH^2$ is the total hydrostatic force on the dam, and A_p is the integral of the function $2gp(\hat{y})/wH$ over the depth of the impounded water, for $H/H_s = 1$. The hydrodynamic force coefficient A_p is presented in Table 5 for a range of values for the period ratio R_w and the wave reflection coefficient α.

STATIC CORRECTION FOR HIGHER MODE RESPONSE

Because the earthquake response of short vibration period structures, such as concrete gravity dams, is primarily due to the fundamental mode of vibration, the response contributions of the higher vibration modes have, so far, been neglected in the simplified analysis procedure presented in the preceding sections. However, the height-wise mass distribution of concrete gravity dams is such that the effective mass (2) in the fundamental vibration mode is small, e.g. it is 35 percent of the total mass for the standard dam section mentioned earlier. Thus, the contributions of the higher vibration modes to the earthquake forces may not be negligible, and a simple method to consider them is presented in this section.

Dams on Rigid Foundation Rock with Empty Reservoir

Because the periods of the higher vibration modes of concrete gravity dams are very short, the corresponding ordinates of the pseudo-acceleration response spectrum for the design earthquake will be essentially equal to the zero-period ordinate or maximum ground acceleration. With little dynamic amplification, the higher vibration modes respond in essentially a static manner to earthquake ground motion, leading to the "static correction" concept (1, 12). The maximum earthquake effects associated with the higher vibration modes can then be represented by the equivalent lateral forces (Appendix B):

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

in which a_g is the maximum ground acceleration. The shape of only the fundamental vibration mode enters into Eq. 13 and the shapes of the higher modes are not required, thus simplifying the analysis considerably.

Dams on Flexible Foundation Rock with Empty Reservoir

Just as in the case of multistory buildings (14), soil-structure interaction effects may be neglected in a simplified procedure to compute the contributions of the higher vibration modes to the earthquake response of dams. Thus, Eq. 13 for the equivalent lateral forces is still valid.

To demonstrate the accuracy of Eq. 13, based on the "static correction" method to determine the response contributions of the higher vibration modes, the response of an idealized triangular dam monolith with empty reservoir to the S69E component of Taft ground motion is presented. Assuming the dam base to be rigid, which is reasonable (4), the earthquake response of the dam was computed as a function of time, considering a variable number of natural vibration modes of the dam on rigid foundation rock, in addition to the two rigid-body modes of the base allowed by foundation-rock flexibility. The height-wise distribution of maximum shear force V_{max} and maximum bending moment M_{max} for two dam heights and three values of E_f/E_s is presented in Figs. 9 and 10 from three analyses: (1) using the first eight vibration modes, which essentially gives the exact response; (2) using only the fundamental vibration mode; and (3) using the fundamental vibration mode and the "static correction" for the higher mode response contributions. It is apparent that in some cases the higher mode response contributions may be significant and that they are estimated to a useful degree of accuracy by the "static correction" procedure.

Dams on Flexible Foundation Rock with Impounded Water

Dam-water interaction introduces significant damping in the response of the higher vibration modes of concrete gravity dams (6), but it has little effect on the higher vibration periods. Because these periods for concrete gravity dams are very short and the corresponding modes are heavily damped, the "static correction" method would be appropriate to represent the higher mode responses of the dam, even with impounded water. The equivalent lateral earthquake forces associated with the higher vibration modes of dams, including the effects of the impounded water, are given by an extension of Eq. 13 (Appendix B):

$$f_{sc}(y) = \frac{1}{g}\left\{w_s(y)\left[1 - \frac{L_1}{M_1}\ \phi(y)\right] + \left[gp_o(y) - \frac{B_1}{M_1}\ w_s(y)\ \phi(y)\right]\right\}a_g \tag{14}$$

In Eq. 14, $p_o(y)$ is a real-valued, frequency-independent function describing the hydrodynamic pressure on a rigid dam undergoing unit acceleration, with water compressibility neglected, both assumptions being consistent with the "static correction" concept; and B_1 provides a measure of the portion of $p_o(y)$ that acts in the fundamental vibration mode. Standard values for $p_o(y)$ are presented in Fig. 11 and Table 6. Using the fundamental mode vibration properties of the standard dam:

$$B_1 = 0.052 \frac{F_{st}}{g} \left[\frac{H}{H_s}\right]^2 \tag{15}$$

where F_{st} is the total hydrostatic force on the dam.

RESPONSE COMBINATION

Dynamic Response

As shown in the preceding two sections, the maximum effects of earthquake ground motion in the fundamental vibration mode of the dam have been represented by equivalent lateral forces $f_1(y)$ and those due to all the higher modes by $f_{sc}(y)$. Static analysis of the dam for these two sets of forces provide the values r_1 and r_{sc} for any response quantity r, e.g., the shear force or bending moment at any horizontal section, or the shear or bending stresses at any point. Because the maximum responses r_1 and r_{sc} do not occur at the same time during the earthquake, they should be combined to obtain an estimate of the dynamic response r_d according to the well known modal combination rules: square-root-of-the-sum-of-squares (SRSS) of modal maxima leading to

$$r_d = \sqrt{(r_1)^2 + (r_{sc})^2} \tag{16}$$

or the sum-of-absolute-values (ABSUM) which always provides a conservative result:

$$r_d = |r_1| + |r_{sc}| \tag{17}$$

Because the natural frequencies of lateral vibration of a concrete dam are well separated, it is not necessary to include the correlation of modal responses in Eq. 16. Later in the report when the accuracy of the simplified analysis procedure is evaluated, the SRSS combination rule will be shown to be preferable.

The SRSS and ABSUM combination rules are applicable to the computation of any response quantity that is proportional to the generalized modal coordinate responses. Thus, these combination rules are generally inappropriate to determine the principal stresses. However, as shown in Appendix C, the principal stresses at the faces of a dam monolith may be determined by the SRSS method if the upstream face is nearly vertical and the effects of tail water at the downstream face are small.

Total Response

In order to obtain the total value of a response quantity r, the SRSS estimate of dynamic response r_d should be combined with the static effects r_{st}. The latter may be determined by standard analysis procedures to compute the initial stresses in a dam prior to the earthquake, including effects of the self weight of the dam, hydrostatic pressures, and temperature changes. In order to recognize that the direction of lateral earthquake forces is reversible, combinations of static and dynamic stresses should allow for the worst case, leading to the maximum value of total response:

$$r_{\max} = r_{st} \pm \sqrt{(r_1)^2 + (r_{sc})^2} \tag{18}$$

This combination of static and dynamic responses is appropriate if r_{st}, r_1 and r_{sc} are oriented similarly. Such is the case for the shear force or bending moment at any horizontal section, for the shear and bending stresses at any point, but generally not for principal stresses except under the restricted conditions mentioned above.

SIMPLIFIED ANALYSIS PROCEDURE

The maximum effects of an earthquake on a concrete gravity dam are represented by equivalent lateral forces in the simplified analysis procedure. The lateral forces associated with the fundamental vibration mode are computed to include the effects of dam-water interaction, water compressibility, reservoir bottom absorption, and dam-foundation rock interaction. The response contributions of the higher vibration modes are computed under the assumption that the dynamic amplification of the modes is negligible, the interaction effects between the dam, impounded water, and foundation rock are not significant, and that the effects of water compressibility can be neglected. These approximations provide a practical method for including the most important factors that affect the earthquake response of concrete gravity dams.

Selection of System Parameters

The simplified analysis procedure requires only a few parameters to describe the dam-water-foundation rock system: E_s, ξ_1, H_s, E_f, η_f, H and α.

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 (3). In using the figures and tables presented earlier to conservatively include dam-water interaction effects in the computation of earthquake forces (Eq. 8), 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, ξ_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 tests. To be conservative, the value of η_f should be rounded down to the nearest value for which data are available: η_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 η_f = 0.10 is recommended.

The depth H of the impounded water is measured from the free surface 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: α = 1.0, 0.90, 0.75, 0.50, 0.25 , 0.00. For proposed new dams or recent dams where sediment deposits are meager, α = 0.90 or 1.0 is recommended and, lacking data, α = 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 -- without 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. 3 for more detail.

Computational Steps

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

Part I: 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 Eq. 10 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 Eq. 4a in which T_1 was computed in Step 1; R_r = period ratio determined from Fig. 4 or Table 2 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 Eq. 11 in which $\tilde{T}_r$ was computed in Step 2, and $T_1^r = 4H/C$ where $C = 4720$ feet per second.

4. Compute $\tilde{T}_1$, the fundamental vibration period of the dam in seconds, including the influence of foundation-rock flexibility and of impounded water, from Eq. 4c in which R_r was determined in Step 2; R_f = period ratio determined from Fig. 6 or Table 3 for the design value of E_f/E_s ; and 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 Eq. 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 Fig. 5 or Table 2 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. 7 or Table 3 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 Fig. 8 or Table 4 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 Eq. 12a: in which R_r was computed in Step 2, and M_1 is computed from Eq. 7a, in which $w_s(y)$ = the weight of the dam per unit height; the fundamental vibration mode shape $\phi(y)$ is given in Fig. 3 or Table 1; and $g = 32.2$ feet per squared second. Evaluation of Eq. 7a may be avoided by obtaining an approximate value from $M_1 = 0.043\ W_s/g$, where W_s is the total weight of the dam monolith.

8. Compute the generalized earthquake force coefficient $\tilde{L}_1$ from Eq. 12b in which L_1 is computed from Eq. 7b; $F_{st} = wH^2/2$; and A_p is given in Table 5 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. 7b may be avoided by obtaining an approximate value from $L_1 = 0.13\ W_s/g$.

Note: Computation of Steps 7 and 8 may be avoided by using conservative values: $\tilde{L}_1/\tilde{M}_1 = 4$ for dams with impounded water, and $L_1/M_1 = 3$ for dams with empty reservoirs.

9. Compute $f_1(y)$, the equivalent lateral earthquake forces associated with the fundamental vibration mode from Eq. 8 in which $S_a(\tilde{T}_1,\tilde{\xi}_1)$ = the pseudo-acceleration 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. 3 or Table 1; $\tilde{M}_1$ and $\tilde{L}_1$ = generalized mass and earthquake force coefficient determined in Steps 7 and 8, respectively; the hydrodynamic pressure term $gp(y,\tilde{T}_r)$ was determined in Step 6; and g = 32.2 feet per squared second.

10. Determine 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, all the response quantities of interest, in particular the stresses throughout the dam. Traditional procedures for design calculations may be used wherein the bending stresses across a horizontal section are computed by elementary formulas for stresses in beams. Alternatively, the finite element method may be used for accurate static stress analysis.

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 Eq. 14 in which M_1 and L_1 were determined in Steps 7 and 8, respectively; $gp_o(y)$ is determined from Fig. 11 or Table 6; B_1 is computed from Eq. 15; 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 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, all the response quantities of interest, in

particular the bending stresses throughout the dam. The stress analysis may be carried out by the procedures mentioned in Step 10.

Part III: The total earthquake forces and stresses in the dam are determined by the following computational step:

13. Compute the total value of any response quantity by Eq. 18, 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, and r_{st} is its initial value prior to the earthquake due to various loads, including the self weight of the dam, hydrostatic pressure, and thermal effects.

Use of Metric Units

Because the standard values for most quantities required in the simplified analysis procedure are presented in non-dimensional 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:

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_s}} \tag{19}$$

   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 Figs. 4 and 5 and Table 2 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

As mentioned earlier (4, 5) and in the preceding sections, various approximations were introduced to develop the simplified analysis procedure and these were individually checked to ensure that they would lead to acceptable results. 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.

System and Ground Motion

The system properties for the simplified analysis are taken to be the same as those assumed for the complete response history analysis: the tallest, nonoverflow monolith of the dam is shown in Fig. 12; 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_1 = 5%; modulus of elasticity of foundation rock $E_f = 3.25 \times 10^6$ psi; constant hysteretic damping coefficient of foundation rock, η_f = 0.10; depth of water, H = 381 ft; and, at the reservoir bottom, the wave reflection coefficient α = 0.5.

The ground motion for which the dam is analyzed is the S69E component of the ground motion recorded at the Taft Lincoln School Tunnel during the Kern County, California, earthquake of July 21, 1952. The response spectrum for this ground motion is shown in Fig. 13. 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 analyzed by the simplified analysis procedure for the four cases listed in Table 7. Implementation of the step-by-step analysis procedure described in the preceding section is summarized next with additional details available in Appendix C; all computations are performed for a unit width of the monolith:

1. For $E_s = 3.25 \times 10^6$ psi and $H_s = 400$ ft, from Eq. 10, $T_1 = (1.4)(400)/\sqrt{3.25 \times 10^6} = 0.311$ sec.

2. For $E_s = 3.0 \times 10^6$ psi (rounded down from 3.25×10^6 psi), $\alpha = 0.50$ and $H/H_s = 381/400 = 0.95$, Fig. 5 or Table 2 gives $R = 1.213$, so $\tilde{T}_r = (1.213)(0.311) = 0.377$ sec.

3. From Eq. 11, $T_1^r = (4)(381)/4720 = 0.323$ sec. and $R_w = 0.323/0.377 = 0.86$.

4. For $E_f/E_s = 1$, Fig. 6 or Table 3 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.

5. For Cases 2 and 4, $\xi_r = 0.030$ from Fig. 5 or Table 2 for $E_s = 3.0 \times 10^6$ psi (rounded down from 3.25×10^6 psi), $\alpha = 0.50$, and $H/H_s = 0.95$. For Cases 3 and 4, $\xi_f = 0.068$ from Fig. 7 or Table 3 for $E_f/E_s = 1$ and $\eta_f = 0.10$. With $\xi_1 = 0.05$, 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 8 at eleven equally spaced levels were obtained from Fig. 8 or Table 4 for $R_w = 0.90$ (by rounding $R_w = 0.86$ from Step 3) and $\alpha = 0.50$, and multipled by $(0.0624)(381)(.95)^2 = 21.5$ k/ft.

7. Evaluating Eq. 7a in discrete form gives $M_1 = (1/g)(500 \text{ kip})$. From Eq. 12a, $\tilde{M}_1 = (1.213)^2(1/g)(500) = (736 \text{ kip})/g$.

8. Equation 7b in discrete form gives $L_1 = (1390 \text{ kip})/g$. From Table 5, $A_p = 0.274$ for $R_w = 0.90$ and $\alpha = 0.50$. Equation 12b then gives $\tilde{L}_1 = 1390/g + [(0.0624)(381)^2/2g](0.95)^2(0.274) = (2510 \text{ kip})/g$. 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. 8 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. 12) and the unit weight of concrete, by substituting $\phi(y)$ from Fig. 3a or Table 1, and the $S_a(\tilde{T}_1,\tilde{\xi}_1)$ from Fig. 13 corresponding to the $\tilde{T}_1$ and $\tilde{\xi}_1$ obtained in Steps 4 and 5 (Table 7). the resulting equivalent lateral forces $f_1(y)$ are presented in Table 8 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. 14 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 $gp_o(y)$ from Fig. 11 or Table 6, which is presented in Table 8; using Eq. 15 to compute $B_1 = 0.052[(0.0624)(381)^2/2g](0.95)^2$ = (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 8 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 particular location in the dam.

13. Compute the maximum total value of any response quantity by combining r_1 from step 10, r_{sc} from step 12, and r_{st}, the initial value prior to the earthquake, according to Eq. 18; this is described further in the next subsection.

Computation of Stresses

The equivalent lateral earthquake forces $f_1(y)$ and $f_{sc}(y)$ representing 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 centroids 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 (Ref. 13, page 42). 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 shown in Appendix C, the principal stresses σ_1 and σ_{sc} at any location in the dam due to the forces $f_1(y)$ and $f_{sc}(y)$, respectively, may be combined using the ABSUM or SRSS combination rules, Eqs. 16 and 17. The combined values σ_d based on both these rules, along with the fundamental mode values σ_1 for the maximum principal stresses, are presented in Table 9 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. Obtained by using only the SRSS combination rule, the maximum principal stresses at both faces of the monolith are presented in Figs. 14 and 15.

Comparison with Refined Analysis Procedure

The dam monolith of Fig. 12 was analyzed by the computer program EAGD-84 (9) in which the response history of the dam, idealized as a finite element system, due to the Taft ground motion is computed considering rigorously the effects of dam-water-foundation rock interaction and of reservoir bottom absorption. The results from Ref. 6 at some intermediate steps of the analysis, in particular the resonant period and damping ratio of the fundamental resonant response, are presented in Table 7, and the envelope values of the earthquake-induced maximum principal stresses are presented in Table 9 and in Figs. 14 and 15.

It is apparent from Table 7 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 analysis procedure is able to represent the important effects of dam-water interaction, reservoir bottom absorption, and dam-foundation rock interaction.

As shown in Table 9, the stresses obtained by the simplified procedure considering only the fundamental vibration mode response are about the same as 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. The ABSUM combination rule is overly conservative in representing the higher mode responses and therefore not recommended or included in the subsequent observations.

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 "exact" 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 7). 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 primarily due 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 of dams predicted by the refined analysis (Figs. 14 and 15), and the stresses in that area are therefore underestimated.

Figures 16 and 17 show the maximum principal stresses, including the static stresses. The simplified procedure gives conservative, but reasonable, maximum stresses, with the exception of the heel area of dams with full reservoir. 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 interactions 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.

CONCLUSIONS

A procedure was presented in 1978 for earthquake resistant design of new concrete gravity dams and for the seismic safety evaluation of existing dams (3). The procedure was based on two performance requirements: firstly, dams should remain essentially within the elastic range of behavior for the most intense ground shaking, expected to occur during the useful life of the structure; and secondly, some cracking, which is limited enough that it does not impair the ability of the dam to contain the impounded water and is economically repairable, may be permitted if the most intense ground shaking that the seismic environment is capable of producing were to occur.

A two-stage procedure was proposed for the analysis phase of elastic design and safety evaluation of dams: a simplified analysis procedure in which the response due only to the fundamental vibration mode is estimated directly from the earthquake design spectrum; and a refined response history analysis procedure for finite element idealizations of the dam monolith. The former was recommended for the preliminary phase of design and safety evaluation of dams and the latter for accurately computing the dynamic response and checking the adequacy of the preliminary evaluation (3).

At the time (1978) the dam design procedure was presented, both of these analysis procedures included the effects of dam-water interaction and compressibility of water, but assumed rigid, non-absorptive reservoir bottom materials and neglected the effects of dam-foundation rock interaction. Recently (1984), the refined response history analysis procedure and computer program have been extended to also include the absorptive effects of reservoir bottom materials and dam-foundation rock interaction effects (8, 9). In this paper, the simplified analysis procedure has been extended to include these important effects so that they can now also be considered in the preliminary phase of design and safety evaluation of dams. Also included now in the simplified procedure is a "static correction" method to consider the response contributions of the higher vibration modes. Thus the design procedure proposed in 1978 (3), which is conceptually still valid, should utilize the new refined and simplified analysis procedures.

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NOTATION

The following symbols are used in this report:

A_p	=	integral of $2gp(\hat{y})/wH$ over depth of the impounded water for $H/H_s = 1$ as listed in Table 5;
a_g	=	maximum ground acceleration;
B_1	=	defined in Eq. 15;
C	=	velocity of pressure waves in water;
E_f	=	Young's modulus of elasticity of foundation rock;
E_s	=	Young's modulus of elasticity of dam concrete;
F_{st}	=	$\frac{1}{2}wH^2$;
$f_1(x,y)$	=	equivalent lateral forces acting on the dam due to fundamental vibration mode as defined in Eq. 1;
$f_1(y)$	=	$\int_o^b f_1(x,y)dx$, as defined in Eq. 6;
$f_{sc}(y)$	=	equivalent lateral forces acting on dam due to higher vibration modes as defined in Eq. 14;
g	=	acceleration due to gravity;
H	=	depth of impounded water;
H_s	=	height of upstream face of dam;
L_1	=	integral defined in Eq. 7b;
$\tilde{L}_1$	=	defined in Eq. 12b;
M_1	=	integral defined in Eq. 7a;
$\tilde{M}_1$	=	defined in Eq. 12a;
$m_s(x,y)$	=	unit mass of concrete;
$p_o(y)$	=	hydrodynamic pressure on a rigid dam with water compressibility neglected;
$p(y,\tilde{T}_r)$	=	$Re[\bar{p}_1(y,\tilde{T}_r)]$;
$\bar{p}_1(y,\tilde{T}_r)$	=	complex-valued hydrodynamic pressure on the upstream face due to harmonic acceleration of dam, at period $\tilde{T}_r$, in the fundamental vibration mode;
R_f	=	period lengthening ratio due to foundation-rock flexibility effects;

R_r	=	period lengthening ratio due to hydrodynamic effects;
R_w	=	$T_1^r / \tilde{T}_r$;
r_1	=	maximum response due to the fundamental vibration mode;
r_d	=	maximum dynamic response;
r_{max}	=	maximum total response of dam;
r_{sc}	=	maximum response due to the higher vibration modes;
r_{st}	=	response due to initial static effects;
$S_a(\tilde{T}_1, \tilde{\xi}_1)$	=	ordinate of pseudo-acceleration response spectrum for the ground motion evaluated at period $\tilde{T}_1$ and damping ratio $\tilde{\xi}_1$;
T_1	=	fundamental vibration period of dam on rigid foundation rock with empty reservoir given by Eq. 10;
$\tilde{T}_1$	=	fundamental resonant period of dam on flexible foundation rock with impounded water given by Eq. 4c;
T_1^r	=	$4H/C$, fundamental vibration period of impounded water;
$\tilde{T}_f$	=	fundamental resonant period of dam on flexible foundation rock with empty reservoir given by Eq. 4b;
$\tilde{T}_r$	=	fundamental resonant period of dam on rigid foundation rock with impounded water given by Eq. 4a;
t	=	time;
W_s	=	total weight of dam;
w	=	unit weight of water;
$w_s(x,y)$	=	unit weight of dam;
$w_s(y)$	=	weight of dam per unit height;
x	=	coordinate along the breadth of the dam;
y	=	coordinate along the height of the dam;
$\hat{y}$	=	y/H;
α	=	wave reflection coefficient for reservoir bottom materials;
$\delta(x)$	=	Dirac delta function;
η_f	=	constant hysteretic damping factor for foundation rock;

ξ_1	=	damping ratio of dam on rigid foundation rock with empty reservoir;
$\tilde{\xi}_1$	=	damping ratio for dam on flexible foundation rock with impounded water;
ξ_f	=	added damping ratio due to foundation-rock flexibility effects;
ξ_r	=	added damping ratio due to hydrodynamic effects;
$\phi(y)$	=	fundamental vibration mode shape of dam at upstream face, i.e. $\phi_1^x(O,y)$; and
$\phi_1^k(x,y)$	=	fundamental vibration mode of dam on rigid foundation rock with empty reservoir, where $k = x,y$ denotes x and y components of modal displacements, respectively.

TABLES

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Table 1 -- Standard Fundamental Mode Shape
of Vibration for Concrete Gravity Dams

y/H_s	$\phi(y)$
1.0	1.000
.95	.866
.90	.735
.85	.619
.80	.530
.75	.455
.70	.389
.65	.334
.60	.284
.55	.240
.50	.200
.45	.165
.40	.135
.35	.108
.30	.084
.25	.065
.20	.047
.15	.034
.10	.021
.05	.010
0	0

Table 2(a) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects for Modulus of Elasticity of Concrete, E_s = 5 and 4.5 million psi

H/H_s	α	E_s = 5 million psi		E_s = 4.5 million psi	
		R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.454	0	1.409	0
	.90	1.462	.043	1.416	.030
	.75	1.456	.060	1.412	.051
	.50	1.355	.067	1.344	.060
	.25	1.284	.054	1.285	.050
	.0	1.261	.038	1.259	.036
.95	1.0	1.368	0	1.323	0
	.90	1.376	.044	1.330	.031
	.75	1.366	.056	1.323	.049
	.50	1.255	.060	1.256	.053
	.25	1.208	.045	1.208	.042
	0	1.192	.032	1.191	.030
.90	1.0	1.289	0	1.247	0
	.90	1.297	.041	1.253	.029
	.75	1.284	.050	1.247	.042
	.50	1.181	.050	1.185	.044
	.25	1.151	.036	1.152	.033
	0	1.139	.025	1.139	.023
.85	1.0	1.215	0	1.179	0
	.90	1.224	.033	1.185	.023
	.75	1.206	.042	1.177	.034
	.50	1.129	.039	1.131	.033
	.25	1.111	.027	1.109	.025
	0	1.100	.019	1.099	.018
.80	1.0	1.148	0	1.121	0
	.90	1.156	.024	1.126	.015
	.75	1.140	.032	1.121	.024
	.50	1.092	.028	1.092	.024
	.25	1.078	.019	1.078	.018
	0	1.071	.014	1.071	.013
.75	1.0	1.092	0	1.078	0
	.90	1.099	.014	1.080	.008
	.75	1.089	.021	1.078	.014
	.50	1.065	.018	1.064	.015
	.25	1.055	.013	1.055	.012
	0	1.049	.009	1.050	.009

Table 2(a). - Continued

H/H_s	α	E_s = 5 million psi		E_s = 4.5 million psi	
		R_r	ξ_r	R_r	ξ_r
.70	1.0	1.055	0	1.048	0
	.90	1.057	.006	1.050	.003
	.75	1.055	.011	1.050	.007
	.50	1.045	.011	1.044	.009
	.25	1.038	.009	1.037	.008
	0	1.034	.006	1.035	.006
.65	1.0	1.033	0	1.031	0
	.90	1.034	.002	1.031	.001
	.75	1.034	.005	1.031	.003
	.50	1.030	.006	1.029	.005
	.25	1.026	.005	1.027	.005
	0	1.024	.004	1.025	.004
.60	1.0	1.020	0	1.020	0
	.90	1.020	.001	1.020	.001
	.75	1.020	.002	1.020	.001
	.50	1.019	.003	1.018	.003
	.25	1.017	.003	1.018	.003
	0	1.016	.003	1.016	.002
.55	1.0	1.013	0	1.012	0
	.90	1.013	.000	1.012	.000
	.75	1.013	.001	1.012	.001
	.50	1.013	.002	1.012	.001
	.25	1.012	.002	1.012	.002
	0	1.011	.002	1.012	.001
.50	1.0	1.009	0	1.008	0
	.90	1.009	.000	1.008	.000
	.75	1.009	.000	1.008	.000
	.50	1.008	.001	1.008	.001
	.25	1.008	.001	1.008	.001
	0	1.008	.001	1.008	.001

Table 2(b) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects for Modulus of Elasticity of Concrete, E_s = 4, 3.5, and 3 million psi

H/H_s	α	E_s = 4 million psi		E_s = 3.5 million psi		E_s = 3 million psi.	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.370	0	1.341	0	1.320	0
	.90	1.374	.021	1.344	.013	1.319	.008
	.75	1.374	.040	1.341	.029	1.312	.021
	.50	1.333	.051	1.316	.042	1.289	.035
	.25	1.285	.045	1.282	.040	1.264	.036
	0	1.259	.034	1.256	.032	1.247	.030
.95	1.0	1.289	0	1.259	0	1.241	0
	.90	1.292	.020	1.263	.012	1.240	.007
	.75	1.289	.038	1.259	.027	1.233	.019
	.50	1.247	.045	1.238	.036	1.213	.030
	.25	1.208	.038	1.208	.033	1.194	.030
	0	1.191	.028	1.188	.026	1.181	.025
.90	1.0	1.214	0	1.191	0	1.176	0
	.90	1.220	.017	1.193	.010	1.176	.006
	.75	1.214	.033	1.193	.022	1.171	.015
	.50	1.179	.037	1.174	.029	1.155	.024
	.25	1.152	.030	1.152	.026	1.141	.024
	0	1.139	.022	1.136	.020	1.131	.019
.85	1.0	1.152	0	1.136	0	1.126	0
	.90	1.157	.013	1.139	.007	1.125	.004
	.75	1.155	.024	1.136	.016	1.122	.011
	.50	1.129	.028	1.124	.023	1.111	.017
	.25	1.109	.022	1.109	.020	1.101	.017
	0	1.099	.017	1.099	.016	1.093	.015
.80	1.0	1.104	0	1.095	0	1.087	0
	.90	1.106	.008	1.094	.004	1.087	.003
	.75	1.106	.016	1.090	.011	1.085	.007
	.50	1.089	.019	1.080	.016	1.079	.012
	.25	1.078	.016	1.071	.014	1.071	.012
	0	1.071	.012	1.066	.011	1.066	.011
.75	1.0	1.070	0	1.063	0	1.059	0
	.90	1.069	.004	1.063	.003	1.059	.002
	.75	1.065	.010	1.061	.006	1.058	.004
	.50	1.056	.013	1.055	.010	1.054	.007
	.25	1.050	.011	1.050	.010	1.050	.008
	0	1.046	.009	1.046	.008	1.046	.007

Table 2(b) -- Continued

H/H_s	α	E_s = 4 million psi		E_s = 3.5 million psi		E_s = 3 million psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
.70	1.0	1.044	0	1.041	0	1.039	0
	.90	1.044	.002	1.041	.001	1.039	.001
	.75	1.042	.005	1.040	.003	1.038	.002
	.50	1.038	.007	1.037	.006	1.036	.004
	.25	1.034	.007	1.034	.006	1.034	.005
	0	1.031	.006	1.031	.005	1.031	.005
.65	1.0	1.028	0	1.026	0	1.025	0
	.90	1.028	.001	1.026	.001	1.025	.000
	.75	1.027	.002	1.026	.002	1.025	.001
	.50	1.025	.004	1.024	.003	1.024	.002
	.25	1.023	.004	1.022	.004	1.022	.003
	0	1.021	.004	1.021	.003	1.021	.003
.60	1.0	1.017	0	1.016	0	1.016	0
	.90	1.017	.000	1.016	.000	1.016	.000
	.75	1.017	.001	1.016	.001	1.016	.001
	.50	1.016	.002	1.015	.002	1.015	.001
	.25	1.015	.002	1.014	.002	1.014	.002
	0	1.013	.002	1.013	.002	1.013	.002
.55	1.0	1.010	0	1.010	0	1.010	0
	.90	1.010	.000	1.010	.000	1.010	.000
	.75	1.010	.001	1.010	.000	1.010	.000
	.50	1.010	.001	1.010	.001	1.009	.001
	.25	1.009	.001	1.009	.001	1.009	.001
	0	1.009	.001	1.009	.001	1.009	.001
.50	1.0	1.006	0	1.006	0	1.006	0
	.90	1.006	.000	1.006	.000	1.006	.000
	.75	1.006	.000	1.006	.000	1.006	.000
	.50	1.006	.001	1.006	.000	1.006	.000
	.25	1.005	.001	1.005	.001	1.005	.001
	0	1.005	.001	1.005	.001	1.005	.001

Table 2(c) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects for Modulus of Elasticity of Concrete, E_s = 2.5, 2 and 1 million psi

H/H_s	α	$E_s = 2.5$ million psi		$E_s = 2$ million psi		$E_s = 1$ million psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.301	0	1.286	0	1.263	0
	.90	1.301	.005	1.285	.003	1.263	.001
	.75	1.287	.014	1.284	.009	1.262	.004
	.50	1.283	.025	1.275	.018	1.260	.008
	.25	1.264	.030	1.262	.024	1.256	.013
	0	1.247	.027	1.247	.024	1.247	.017
.95	1.0	1.224	0	1.212	0	1.193	0
	.90	1.224	.005	1.211	.003	1.193	.001
	.75	1.221	.012	1.210	.008	1.193	.003
	.50	1.209	.022	1.203	.015	1.191	.007
	.25	1.194	.025	1.192	.020	1.187	.011
	0	1.181	.022	1.181	.020	1.181	.014
.90	1.0	1.164	0	1.154	0	1.140	0
	.90	1.163	.004	1.154	.002	1.140	.001
	.75	1.161	.009	1.152	.006	1.140	.002
	.50	1.152	.017	1.148	.012	1.139	.005
	.25	1.141	.020	1.140	.016	1.136	.008
	0	1.131	.018	1.131	.016	1.131	.011
.85	1.0	1.117	0	1.110	0	1.100	0
	.90	1.116	.003	1.110	.002	1.100	.001
	.75	1.115	.007	1.109	.004	1.100	.002
	.50	1.109	.012	1.106	.009	1.100	.004
	.25	1.101	.014	1.100	.012	1.097	.006
	0	1.093	.013	1.093	.012	1.093	.008
.80	1.0	1.081	0	1.077	0	1.071	0
	.90	1.081	.002	1.077	.001	1.071	.000
	.75	1.080	.004	1.076	.003	1.071	.001
	.50	1.076	.008	1.074	.006	1.070	.003
	.25	1.071	.010	1.071	.008	1.069	.005
	0	1.066	.010	1.066	.008	1.066	.006
.75	1.0	1.055	0	1.053	0	1.049	0
	.90	1.055	.001	1.053	.001	1.049	.000
	.75	1.054	.003	1.052	.002	1.049	.001
	.50	1.053	.005	1.051	.004	1.048	.002
	.25	1.050	.007	1.049	.005	1.048	.003
	0	1.046	.007	1.046	.006	1.046	.004

Table 2(c) -- Continued

H/H_s	α	$E_s = 2.5$ million psi		$E_s = 2$ million psi		$E_s = 1$ million psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
.70	1.0	1.037	0	1.035	0	1.033	0
	.90	1.037	.001	1.035	.000	1.033	.000
	.75	1.037	.002	1.035	.001	1.033	.000
	.50	1.035	.003	1.034	.002	1.033	.001
	.25	1.033	.004	1.033	.004	1.032	.002
	0	1.031	.004	1.031	.004	1.031	.003
.65	1.0	1.024	0	1.023	0	1.022	0
	.90	1.024	.000	1.023	.000	1.022	.000
	.75	1.024	.001	1.023	.001	1.022	.000
	.50	1.023	.002	1.023	.001	1.022	.001
	.25	1.022	.003	1.022	.002	1.021	.001
	0	1.021	.003	1.021	.003	1.021	.002
.60	1.0	1.016	0	1.016	0	1.014	0
	.90	1.016	.000	1.016	.000	1.014	.000
	.75	1.016	.001	1.016	.001	1.014	.000
	.50	1.015	.001	1.015	.001	1.014	.000
	.25	1.014	.002	1.014	.002	1.014	.001
	0	1.013	.002	1.013	.002	1.013	.001
.55	1.0	1.009	0	1.009	0	1.009	0
	.90	1.009	.000	1.009	.000	1.009	.000
	.75	1.009	.000	1.009	.000	1.009	.000
	.50	1.009	.001	1.009	.000	1.009	.000
	.25	1.009	.001	1.009	.001	1.009	.000
	0	1.009	.001	1.009	.001	1.009	.001
.50	1.0	1.006	0	1.006	0	1.005	0
	.90	1.006	.000	1.006	.000	1.005	.000
	.75	1.006	.000	1.006	.000	1.005	.000
	.50	1.006	.000	1.005	.000	1.005	.000
	.25	1.005	.000	1.005	.000	1.005	.000
	0	1.005	.001	1.005	.000	1.005	.000

Table 3 -- Standard Values for R_f and ξ_f, the Period Lengthening Ratio and Added Damping Ratio, due to Dam-Foundation Rock Interaction

E_f/E_s	R_f	Added Damping Ratio, ξ_f			
		η_f=0.01	η_f=0.10	η_f=0.25	η_f=0.50
5.0	1.043	.015	.014	.019	.024
4.5	1.048	.015	.016	.021	.026
4.0	1.054	.015	.018	.023	.030
3.5	1.062	.015	.020	.027	.034
3.0	1.071	.016	.024	.031	.039
2.5	1.083	.020	.028	.037	.046
2.0	1.099	.028	.035	.046	.057
1.5	1.129	.039	.047	.060	.073
1.4	1.139	.042	.050	.063	.078
1.3	1.150	.044	.053	.068	.084
1.2	1.162	.047	.058	.073	.090
1.1	1.174	.050	.062	.079	.096
1.0	1.187	.054	.068	.086	.105
0.9	1.204	.060	.075	.094	.115
0.8	1.223	.068	.083	.104	.127
0.7	1.248	.077	.093	.116	.142
0.6	1.286	.088	.105	.131	.161
0.5	1.335	.103	.121	.151	.186
0.4	1.400	.117	.143	.178	.221
0.3	1.496	.145	.173	.217	.273
0.2	1.678	.186	.220	.279	.362

Table 4(a) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$, for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 1.00$

$\hat{y} = \frac{y}{H}$	Value of $\frac{g\,p(\hat{y})}{w\,H}$												
	$R_w \leq .5$	$R_w = .7$	$R_w = .8$	$R_w = .85$	$R_w = .90$	$R_w = .92$	$R_w = .93$	$R_w = .94$	$R_w = .95$	$R_w = .96$	$R_w = .97$	$R_w = .98$	$R_w = .99$
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0
.95	.070	.073	.076	.079	.083	.086	.088	.090	.092	.096	.102	.111	.133
.90	.112	.118	.124	.129	.138	.143	.147	.151	.157	.164	.176	.195	.238
.85	.127	.135	.144	.152	.164	.172	.178	.184	.193	.204	.221	.249	.313
.80	.133	.144	.155	.165	.182	.193	.200	.208	.220	.235	.257	.295	.379
.75	.141	.154	.168	.180	.201	.214	.223	.234	.248	.267	.294	.340	.445
.70	.145	.161	.178	.192	.216	.232	.242	.255	.272	.294	.327	.382	.506
.65	.143	.161	.180	.197	.224	.242	.254	.269	.288	.313	.351	.414	.558
.60	.139	.159	.180	.199	.230	.250	.264	.280	.301	.330	.373	.444	.605
.55	.137	.159	.183	.203	.237	.260	.274	.293	.316	.348	.395	.473	.651
.50	.135	.159	.184	.206	.244	.269	.284	.304	.329	.364	.415	.500	.694
.45	.130	.155	.182	.206	.246	.272	.289	.310	.338	.375	.430	.522	.730
.40	.124	.151	.179	.204	.247	.275	.293	.315	.345	.384	.442	.540	.762
.35	.121	.149	.179	.205	.250	.279	.298	.322	.353	.395	.456	.559	.793
.30	.118	.147	.178	.206	.252	.283	.303	.328	.360	.403	.467	.575	.820
.25	.113	.143	.175	.204	.252	.284	.304	.330	.363	.408	.475	.587	.840
.20	.109	.139	.172	.202	.252	.284	.305	.332	.366	.412	.481	.596	.856
.15	.107	.138	.172	.202	.252	.286	.307	.334	.369	.417	.487	.604	.871
.10	.106	.137	.172	.202	.253	.287	.309	.337	.372	.420	.491	.611	.881
.05	.103	.135	.169	.200	.252	.286	.308	.336	.372	.420	.492	.613	.886
0	.010	.133	.168	.198	.251	.285	.307	.335	.371	.420	.492	.613	.886

Table 4(b) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$, for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.90$

$\hat{y} = \frac{y}{H}$	Value of $\frac{g\,p(\hat{y})}{w\,H}$								
	$R_w \le 0.5$	$R_w = .7$	$R_w = .8$	$R_w = .9$	$R_w = .95$	$R_w = 1.0$	$R_w = 1.05$	$R_w = 1.1$	$R_w = 1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.070	.073	.076	.082	.088	.089	.069	.064	.062
.90	.112	.118	.124	.136	.149	.149	.110	.100	.095
.85	.127	.135	.144	.162	.181	.181	.123	.108	.101
.80	.133	.144	.155	.179	.204	.205	.127	.107	.098
.75	.141	.154	.168	.197	.228	.229	.133	.108	.097
.70	.145	.161	.177	.212	.249	.249	.135	.105	.092
.65	.143	.161	.179	.219	.261	.262	.130	.096	.081
.60	.139	.159	.179	.234	.271	.272	.124	.085	.067
.55	.137	.159	.182	.231	.283	.283	.119	.076	.057
.50	.135	.159	.183	.236	.293	.292	.114	.067	.046
.45	.130	.155	.181	.238	.299	.298	.106	.055	.032
.40	.124	.150	.178	.238	.303	.301	.097	.044	.019
.35	.121	.148	.177	.241	.309	.307	.091	.035	.009
.30	.118	.146	.177	.243	.313	.311	.086	.027	.000
.25	.113	.142	.174	.242	.315	.312	.078	.017	.000
.20	.109	.139	.171	.241	.316	.312	.071	.008	.000
.15	.107	.137	.170	.242	.318	.313	.067	.003	.000
.10	.106	.136	.170	.242	.320	.313	.064	.000	.000
.05	.103	.134	.167	.241	.318	.311	.059	.000	.000
0	.101	.133	.166	.239	.317	.309	.056	.000	.000

Table 4(c) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$, for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.75$

$\hat{y} = \frac{y}{H}$	Value of $\frac{g\,p(\hat{y})}{w\,H}$								
	$R_w \leq 0.5$	$R_w = .7$	$R_w = .8$	$R_w = .9$	$R_w = .95$	$R_w = 1.0$	$R_w = 1.05$	$R_w = 1.1$	$R_w = 1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.070	.073	.075	.079	.080	.078	.073	.068	.065
.90	.112	.118	.122	.129	.132	.128	.118	.101	.101
.85	.127	.133	.140	.151	.154	.150	.134	.121	.110
.80	.133	.143	.152	.166	.171	.163	.142	.125	.110
.75	.140	.153	.164	.181	.187	.177	.151	.130	.110
.70	.145	.159	.173	.193	.200	.188	.157	.131	.108
.65	.143	.159	.174	.197	.205	.191	.155	.126	.099
.60	.139	.157	.174	.199	.208	.192	.151	.118	.088
.55	.137	.157	.175	.203	.213	.195	.150	.113	.079
.50	.135	.156	.176	.206	.216	.196	.147	.107	.070
.45	.129	.152	.173	.205	.216	.194	.140	.097	.058
.40	.123	.147	.170	.203	.214	.191	.134	.088	.045
.35	.120	.145	.169	.204	.215	.190	.129	.080	.036
.30	.117	.143	.168	.204	.215	.188	.125	.074	.027
.25	.112	.139	.164	.201	.212	.184	.118	.065	.016
.20	.108	.135	.161	.199	.209	.180	.111	.056	.007
.15	.106	.134	.159	.198	.208	.177	.107	.051	.001
.10	.104	.133	.158	.197	.207	.175	.103	.046	.000
.05	.102	.130	.156	.194	.204	.171	.098	.040	.000
0	.100	.128	.154	.192	.201	.167	.093	.036	.000

Table 4(d) – Standard Values for the Hydrodynamic Pressure Function $p(y)$, for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.50$

$\hat{y} = \frac{y}{H}$	Value of $\frac{g\,p(\hat{y})}{w\,H}$								
	$R_w \leq 0.5$	$R_w = .7$	$R_w = .8$	$R_w = .9$	$R_w = .95$	$R_w = 1.0$	$R_w = 1.05$	$R_w = 1.1$	$R_w = 1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.071	.072	.073	.074	.074	.073	.072	.070	.068
.90	.112	.116	.118	.119	.119	.118	.116	.113	.108
.85	.125	.132	.135	.136	.135	.134	.130	.127	.120
.80	.132	.139	.143	.146	.145	.143	.138	.133	.123
.75	.139	.148	.153	.156	.155	.152	.146	.139	.127
.70	.144	.154	.160	.163	.162	.158	.151	.143	.128
.65	.141	.152	.159	.163	.161	.156	.148	.138	.122
.60	.137	.149	.157	.162	.160	.153	.143	.132	.113
.55	.135	.148	.156	.161	.158	.151	.141	.128	.107
.50	.133	.147	.155	.159	.156	.148	.137	.123	.099
.45	.127	.142	.150	.154	.151	.142	.129	.115	.088
.40	.121	.136	.145	.149	.145	.136	.122	.106	.077
.35	.117	.133	.143	.146	.142	.131	.116	.099	.069
.30	.114	.131	.140	.143	.137	.126	.110	.092	.060
.25	.109	.126	.135	.137	.131	.119	.102	.083	.050
.20	.104	.121	.130	.132	.125	.112	.094	.074	.040
.15	.102	.119	.127	.128	.121	.108	.089	.068	.033
.10	.100	.117	.125	.125	.118	.104	.083	.062	.026
.05	.098	.114	.121	.121	.113	.098	.077	.055	.018
0	.096	.111	.119	.117	.108	.093	.072	.049	.012

Table 4(e) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$, for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.25$

$\hat{y} = \frac{y}{H}$	of $\frac{g\,p(\hat{y})}{w\,H}$								
	$R_w \leq 0.5$	$R_w = .7$	$R_w = .8$	$R_w = .9$	$R_w = .95$	$R_w = 1.0$	$R_w = 1.05$	$R_w = 1.1$	$R_w = 1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.069	.070	.071	.071	.071	.071	.070	.070	.070
.90	.111	.113	.114	.114	.114	.114	.113	.113	.111
.85	.124	.127	.128	.129	.129	.128	.127	.127	.125
.80	.130	.133	.134	.135	.135	.134	.133	.132	.129
.75	.137	.141	.142	.143	.142	.141	.140	.138	.135
.70	.141	.145	.147	.147	.146	.145	.143	.141	.137
.65	.137	.142	.144	.144	.143	.142	.140	.137	.131
.60	.133	.138	.140	.139	.138	.136	.134	.131	.124
.55	.131	.136	.137	.136	.135	.133	.130	.126	.118
.50	.128	.133	.134	.133	.131	.128	.125	.121	.112
.45	.121	.126	.127	.126	.124	.120	.116	.112	.101
.40	.115	.120	.120	.118	.115	.112	.107	.102	.091
.35	.111	.116	.116	.113	.110	.106	.100	.095	.082
.30	.107	.111	.111	.107	.104	.099	.093	.087	.074
.25	.101	.105	.104	.100	.096	.091	.084	.077	.063
.20	.096	.099	.098	.093	.088	.082	.076	.068	.052
.15	.094	.096	.094	.088	.083	.076	.069	.061	.044
.10	.092	.096	.090	.083	.078	.071	.063	.054	.037
.05	.088	.088	.085	.077	.071	.064	.055	.046	.028
0	.086	.085	.081	.072	.065	.057	.048	.039	.020

Table 4(f) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$, for Full Reservoir, i.e., $H/H_s = 1; \alpha = 0.00$

$\hat{y} = \frac{y}{H}$	Value of $\frac{g\,p(\hat{y})}{w\,H}$								
	$R_w \leq 0.5$	$R_w = .7$	$R_w = .8$	$R_w = .9$	$R_w = .95$	$R_w = 1.0$	$R_w = 1.05$	$R_w = 1.1$	$R_w = 1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.069	.069	.069	.069	.069	.069	.070	.070	.070
.90	.109	.110	.110	.111	.111	.111	.112	.112	.112
.85	.122	.123	.124	.125	.125	.125	.126	.126	.126
.80	.127	.128	.128	.129	.129	.129	.130	.130	.130
.75	.133	.134	.134	.135	.135	.135	.136	.136	.136
.70	.135	.136	.137	.138	.138	.138	.139	.139	.139
.65	.132	.133	.133	.133	.133	.133	.134	.134	.134
.60	.127	.127	.127	.127	.127	.127	.127	.127	.127
.55	.123	.123	.123	.123	.123	.123	.122	.122	.121
.50	.120	.119	.118	.118	.118	.117	.116	.116	.115
.45	.113	.111	.110	.109	.109	.108	.107	.106	.105
.40	.105	.103	.102	.100	.099	.098	.097	.096	.094
.35	.101	.098	.096	.094	.092	.091	.090	.088	.085
.30	.096	.092	.090	.087	.085	.084	.082	.080	.076
.25	.090	.085	.082	.078	.076	.074	.072	.069	.065
.20	.084	.078	.074	.070	.067	.065	.062	.059	.053
.15	.080	.073	.068	.064	.061	.058	.055	.051	.045
.10	.077	.069	.064	.058	.054	.051	.048	.044	.036
.05	.073	.063	.057	.050	.046	.043	.039	.035	.026
0	.070	.058	.052	.044	.040	.036	.031	.027	.017

Table 5(a) -- Standard Values for A_p, the Hydrodynamic Force Coefficient in $\tilde{L}_1$; $\alpha = 1$

R_w	Value of A_p for $\alpha=1$
0.99	1.242
0.98	.893
0.97	.739
0.96	.647
0.95	.585
0.94	.539
0.93	.503
0.92	.474
0.90	.431
0.85	.364
0.80	.324
0.70	.279
≤ 0.50	.237

Table 5(b) -- Standard Values for A_p, the Hydrodynamic Force Coefficient in $\tilde{L}_1$; α = 0.90, 0.75, 0.50, 0.25 and 0

R_w	Value of A_p				
	α=0.90	α=0.75	α=0.5	α=0.25	α=0
1.20	.071	.111	.159	.178	.181
1.10	.110	.177	.204	.197	.186
1.05	.194	.249	.229	.205	.189
1.00	.515	.340	.252	.213	.191
0.95	.518	.378	.267	.219	.193
0.90	.417	.361	.274	.224	.195
0.80	.322	.309	.269	.229	.198
0.70	.278	.274	.256	.228	.201
≤ 0.50	.237	.236	.231	.222	.206

Table 6 -- Standard Values
for the Hydrodynamic Pressure Function $p_0(\hat{y})$

$\hat{y} = y/H$	$\frac{g\,p_0}{wH}$
1.0	0
.95	.137
.90	.224
.85	.301
.80	.362
.75	.418
.70	.465
.65	.509
.60	.546
.55	.580
.50	.610
.45	.637
.40	.659
.35	.680
.30	.696
.25	.711
.20	.722
.15	.731
.10	.737
.05	.741
0	.742

Table 7 - Pine Flat Dam Analysis Cases,
Simplified Procedure Parameters, and Fundamental Mode Properties
from Simplified and Refined Analysis Procedures

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. 7.

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

Table 8 - Equivalent Lateral Earthquake Forces on Pine Flat Dam due to S69E Component of Taft Ground Motion

y	w_s	ϕ	$w_s\phi$	gp	gp_o	Lateral Forces, in kips per foot							
						Case 1		Case 2		Case 3		Case 4	
(ft)	(k/ft)		(k/ft)	(k/ft)	(k/ft)	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

Table 9 - Maximum Principal Stresses* (in psi) in Pine Flat Dam due to S69E Component of Taft Ground Motion

Case	Foundation Rock	Water	Upstream Face				Downstream Face			
			Fundamental Mode only	ABSUM	SRSS	Exact†	Fundamental Mode only	ABSUM	SRSS	Exact†
1	rigid	empty	241	296	247	223	333	398	338	272
2	rigid	full	263	309	266	261	411	440	413	277
3	flexible	empty	157	213	167	172	218	284	226	181
4	flexible	full	276	322	278	218	431	460	433	229

*Initial static stresses are excluded.

† From Refined Procedure, Ref. 7.

FIGURES

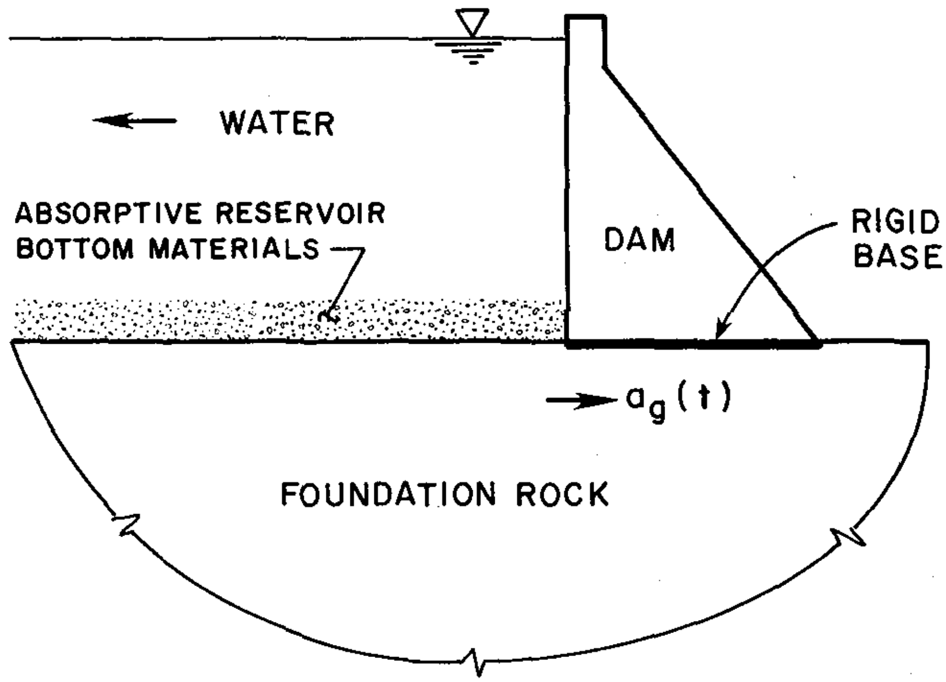


Figure 1. Dam-Water-Foundation Rock System

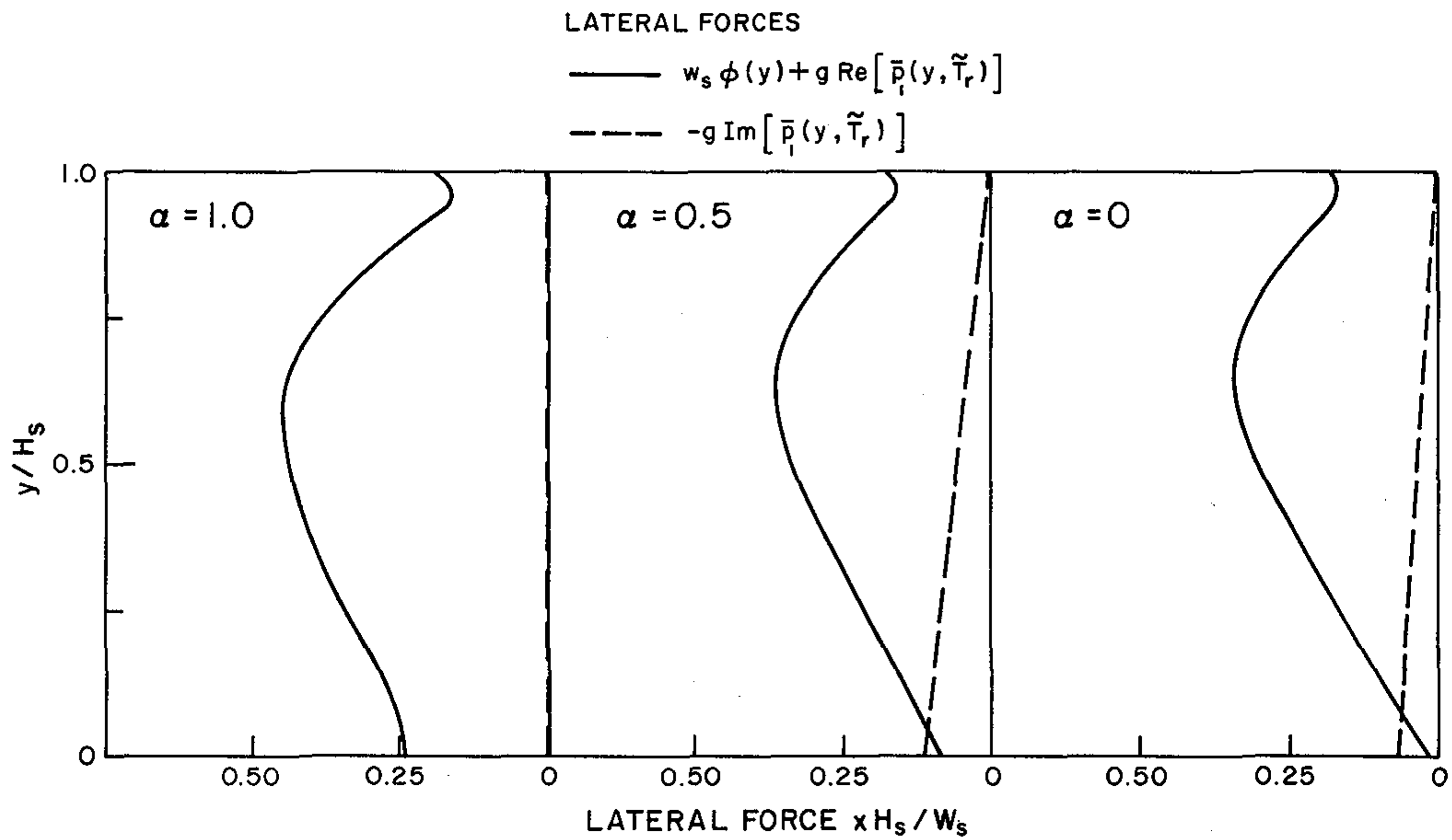


Figure 2. Distribution of Real (Re) and Imaginary (Im) Valued Components of Equivalent Lateral Earthquake Forces on Upstream Face of a Typical Dam

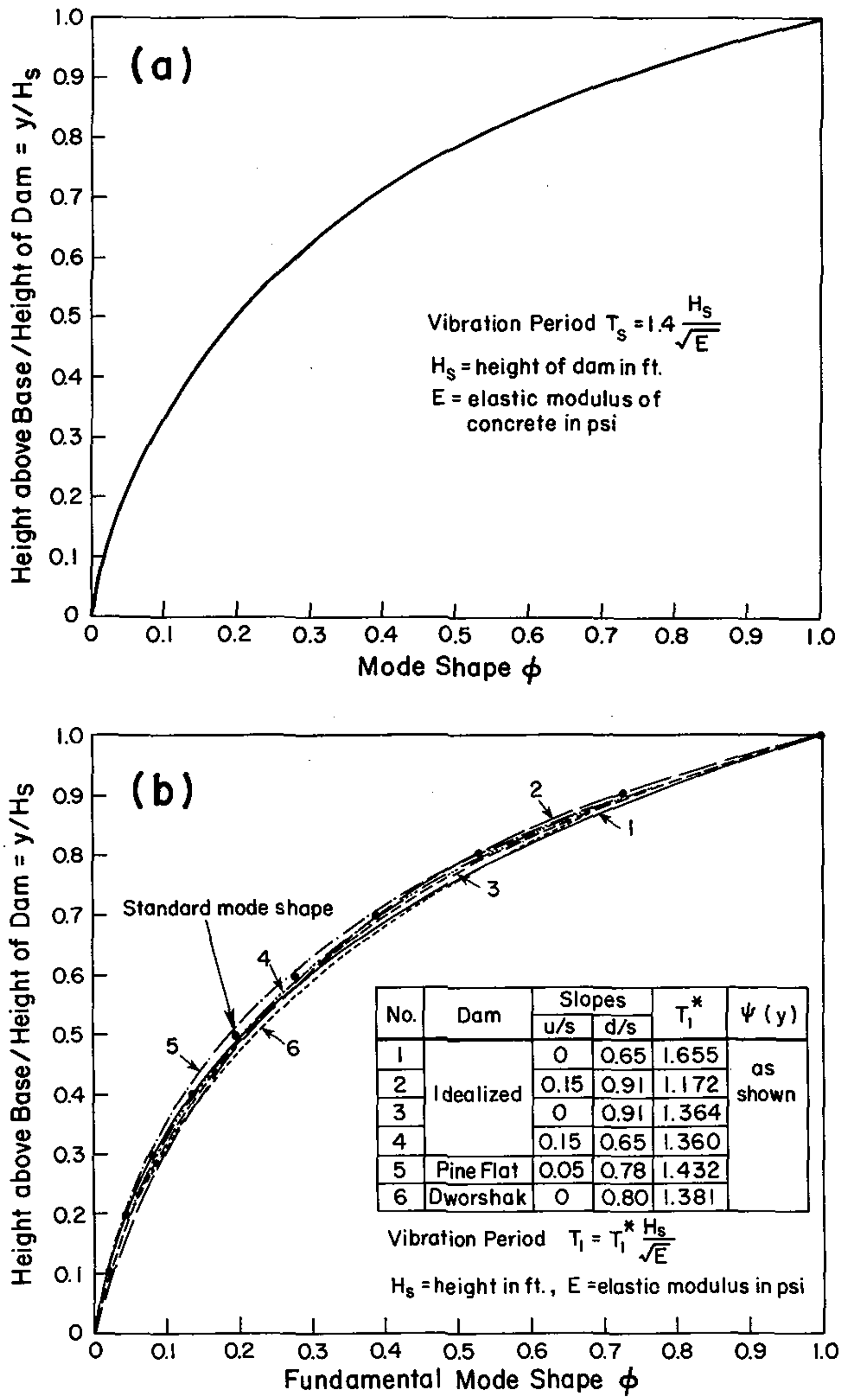


Figure 3. Fundamental Period and Mode Shape of Vibration for Concrete Gravity Dams. (a) Standard Period and Mode Shape. (b) Comparison of Standard Values with Properties of Six Dams.

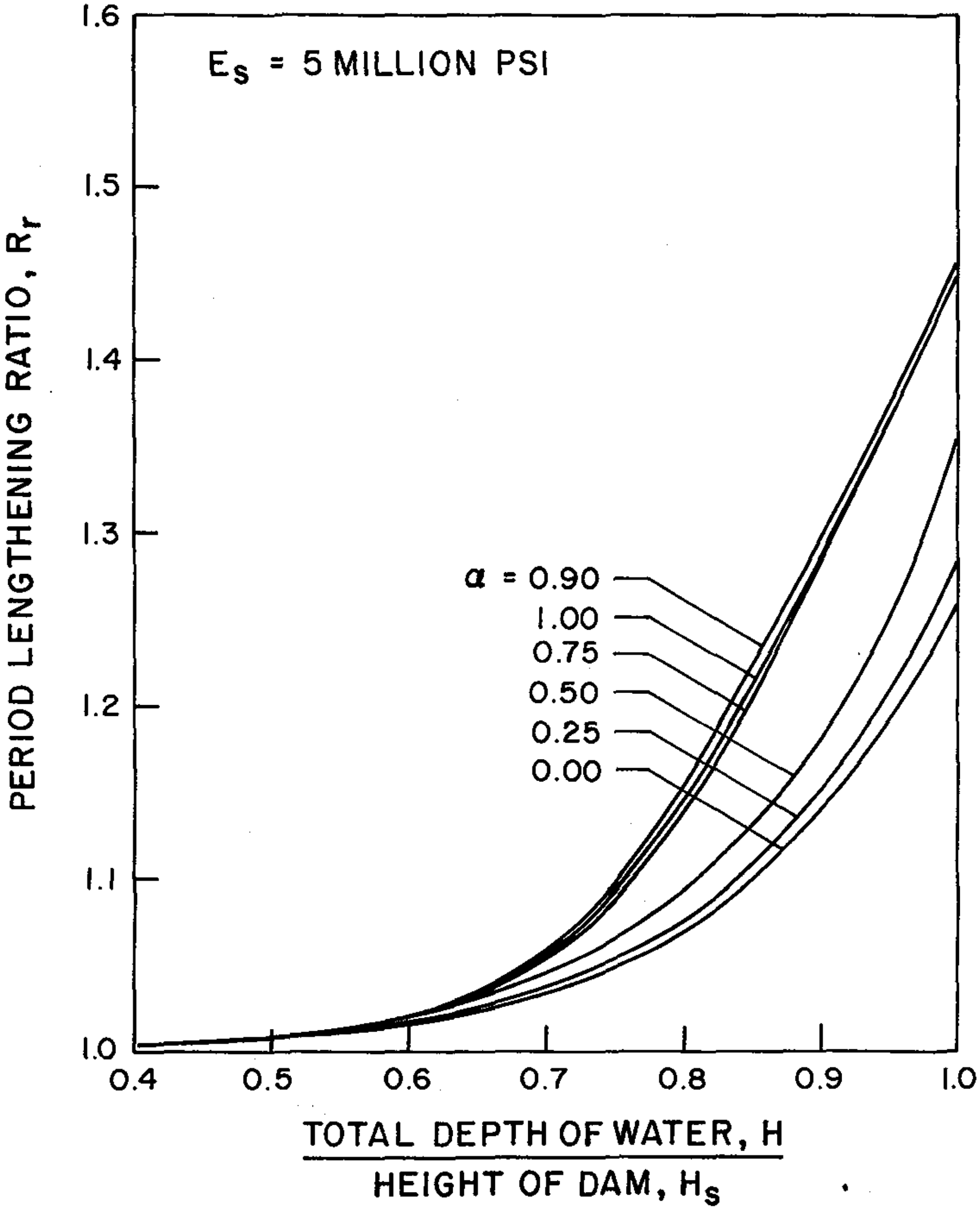


Figure 4(a). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 5 million psi.

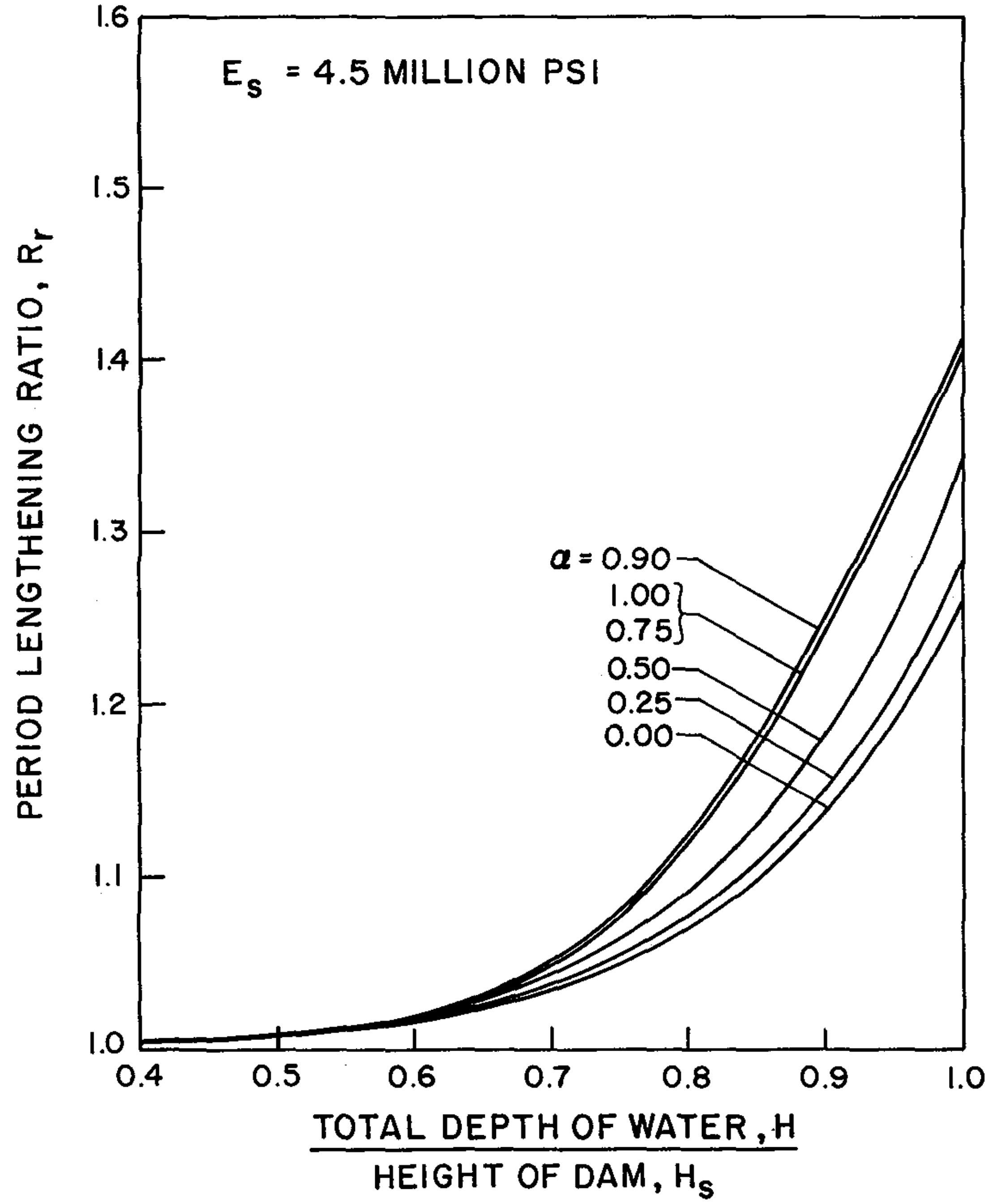


Figure 4(b). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi.

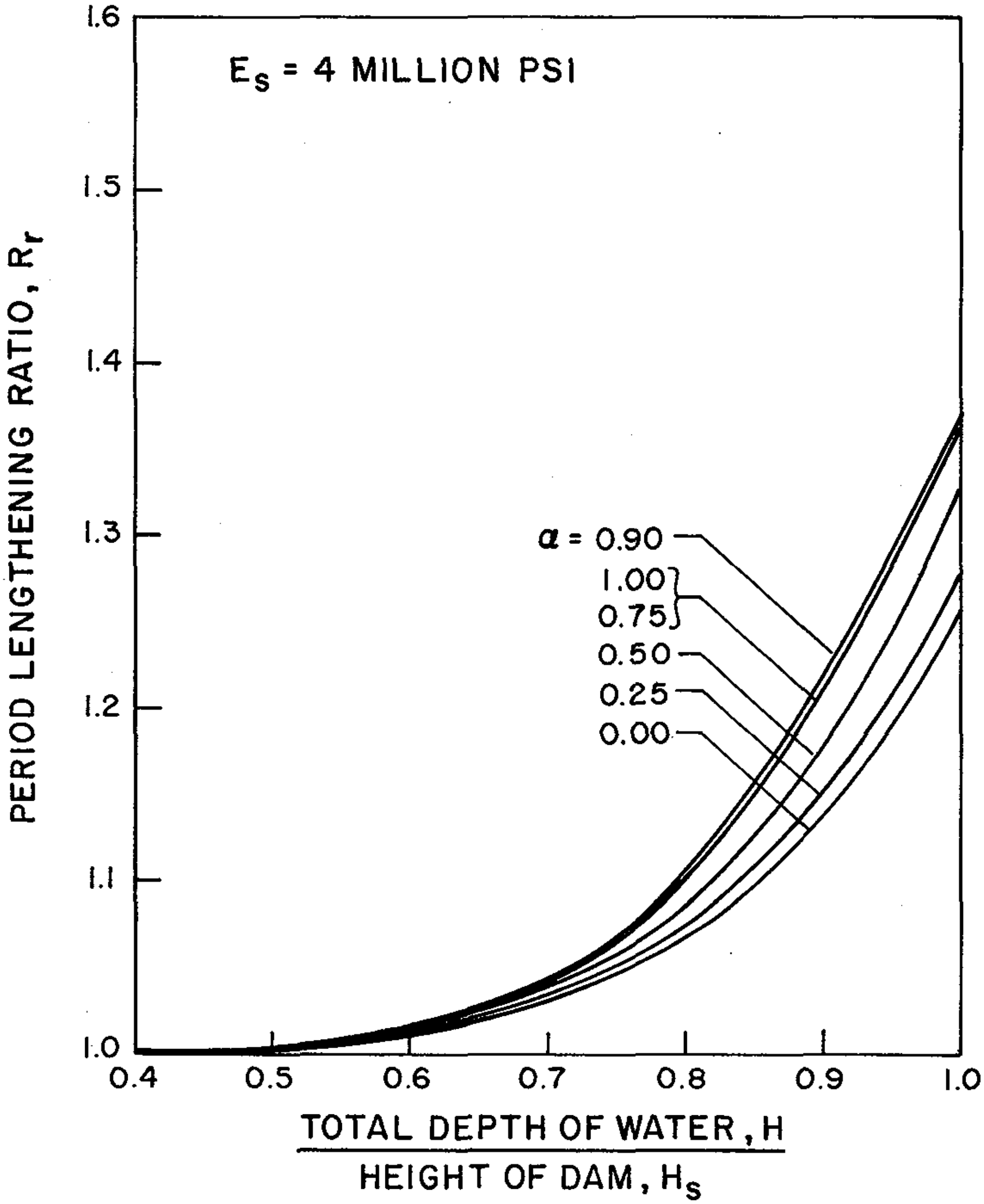


Figure 4(c). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4.0 million psi.

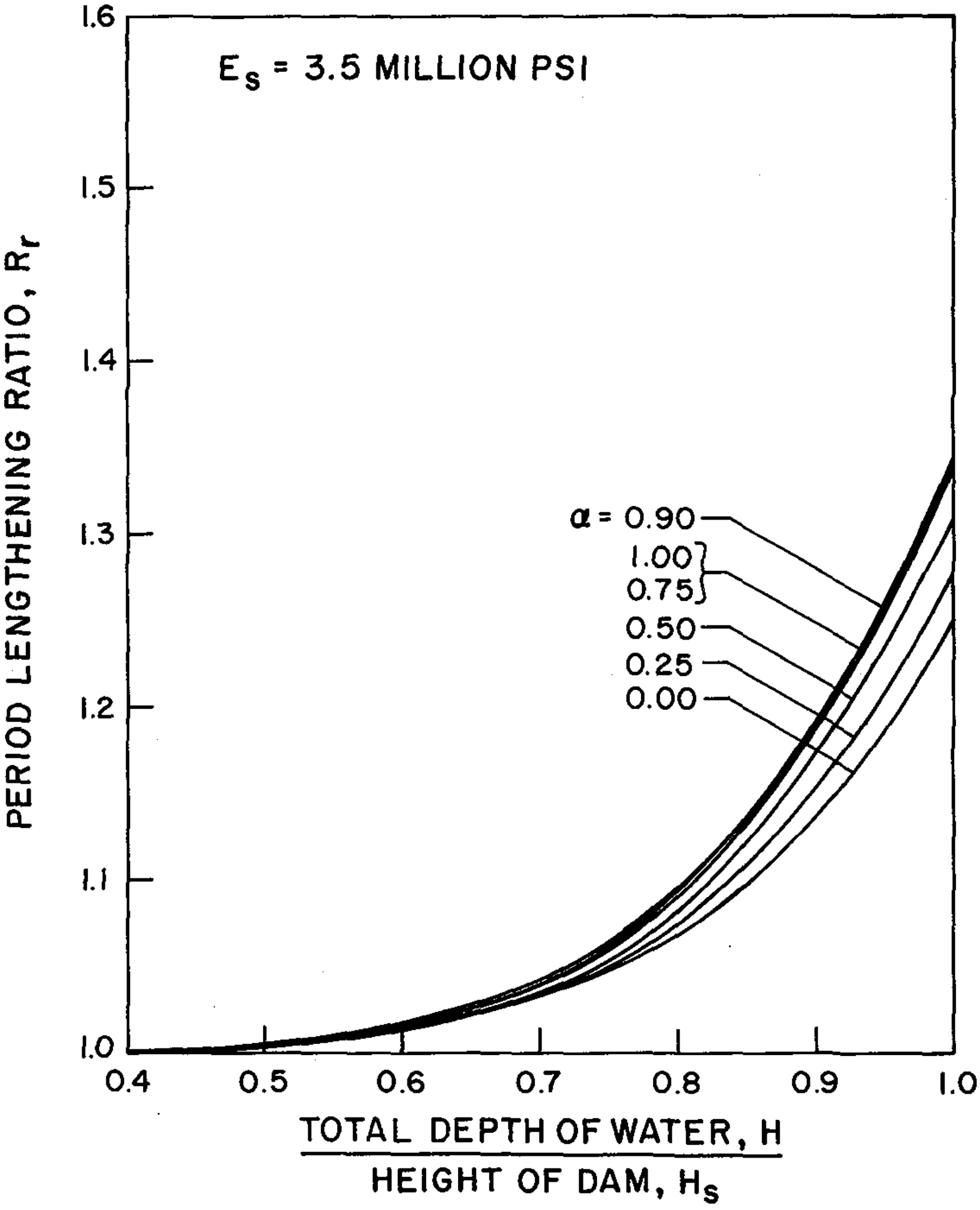


Figure 4(d). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

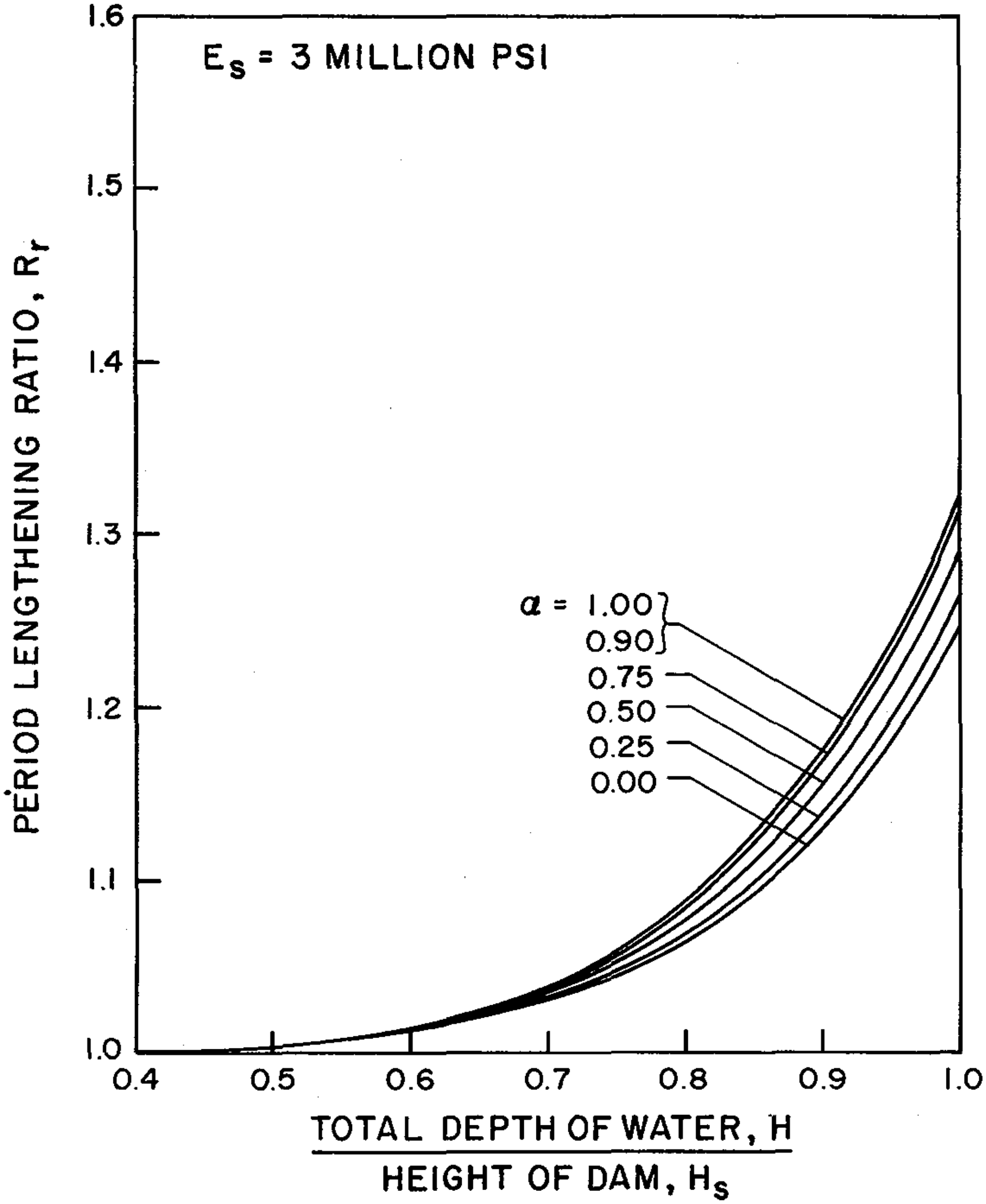


Figure 4(e). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3.0 million psi.

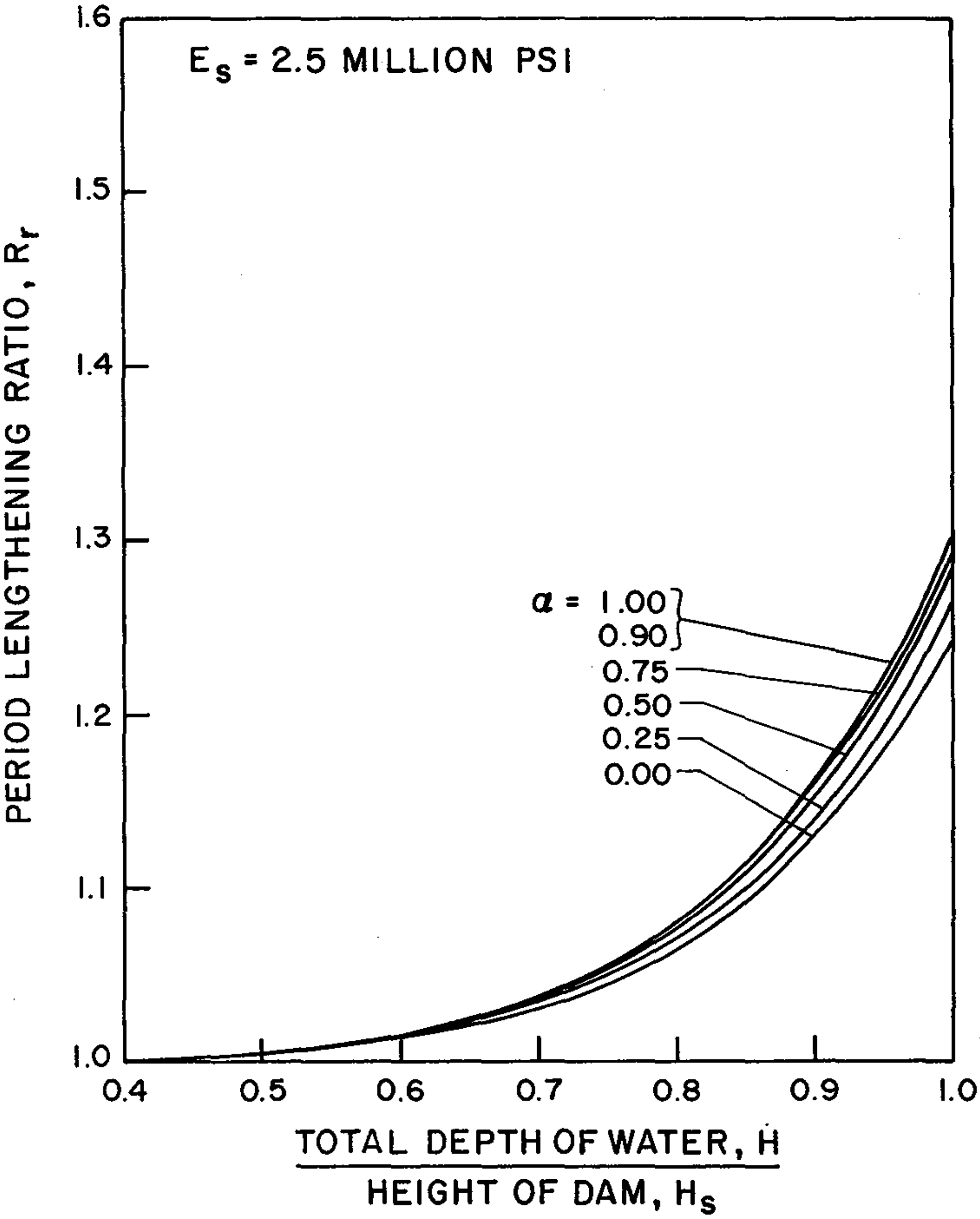


Figure 4(f). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

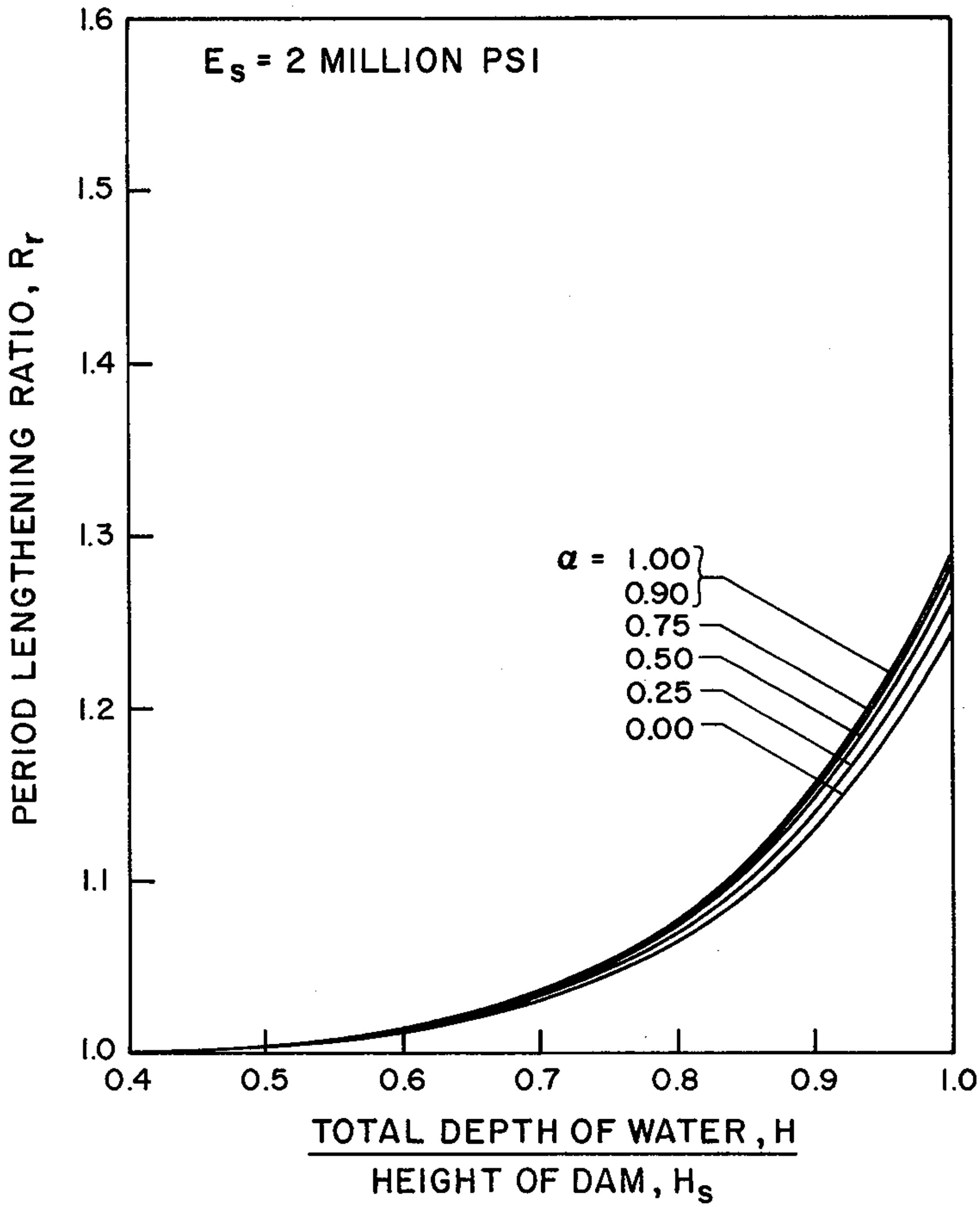


Figure 4(g). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2.0 million psi.

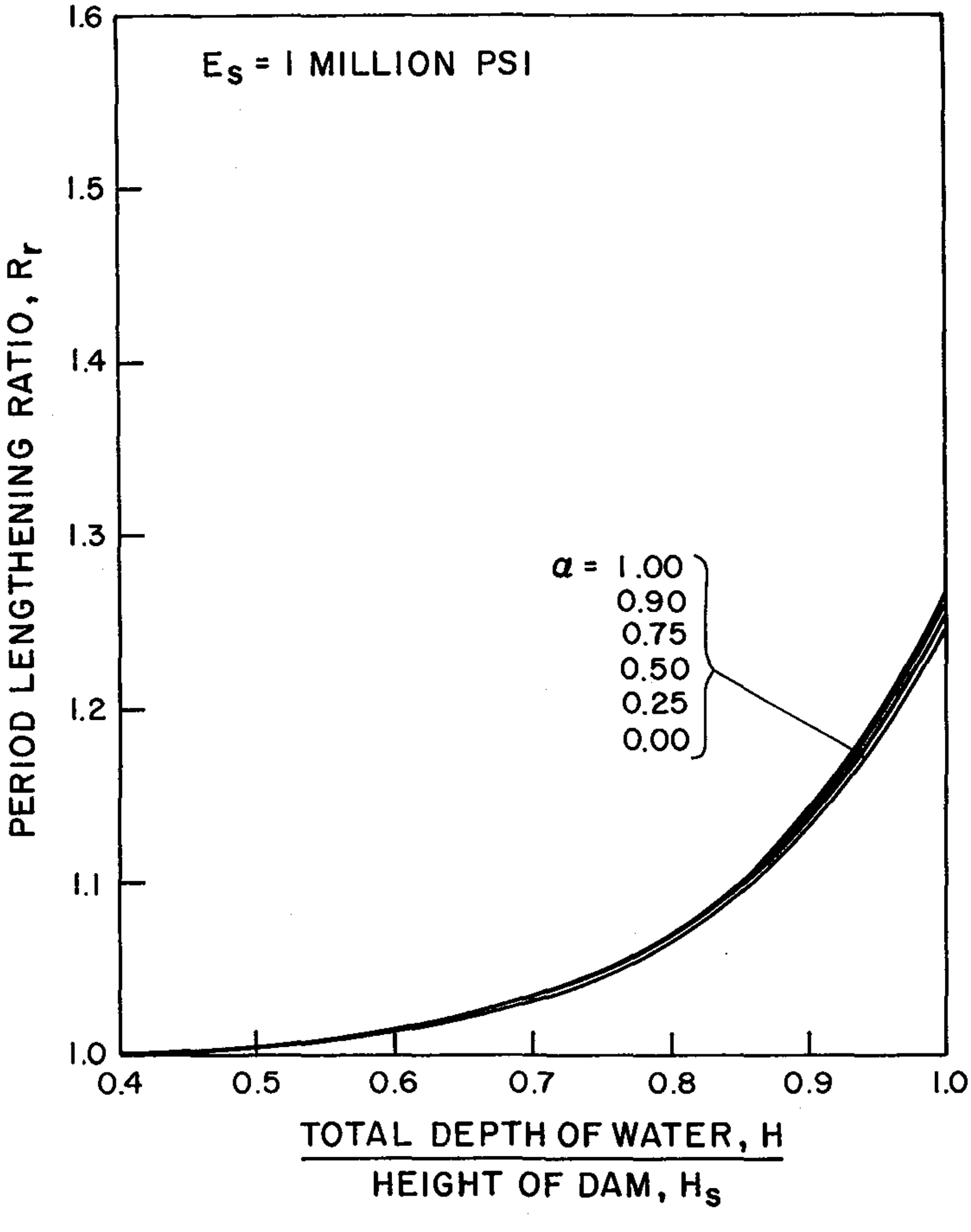


Figure 4(h). Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 1.0 million psi.

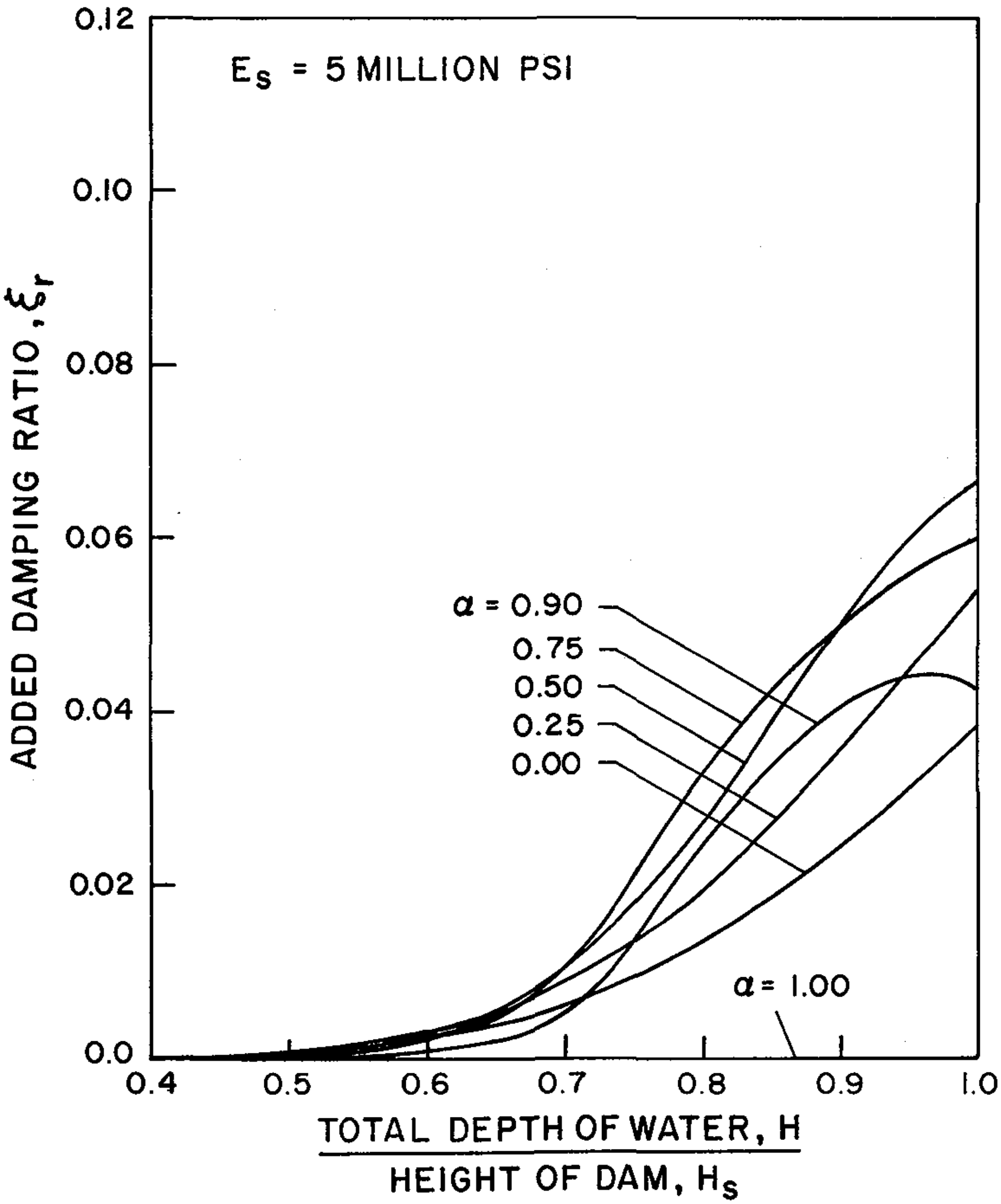


Figure 5(a). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 5 million psi

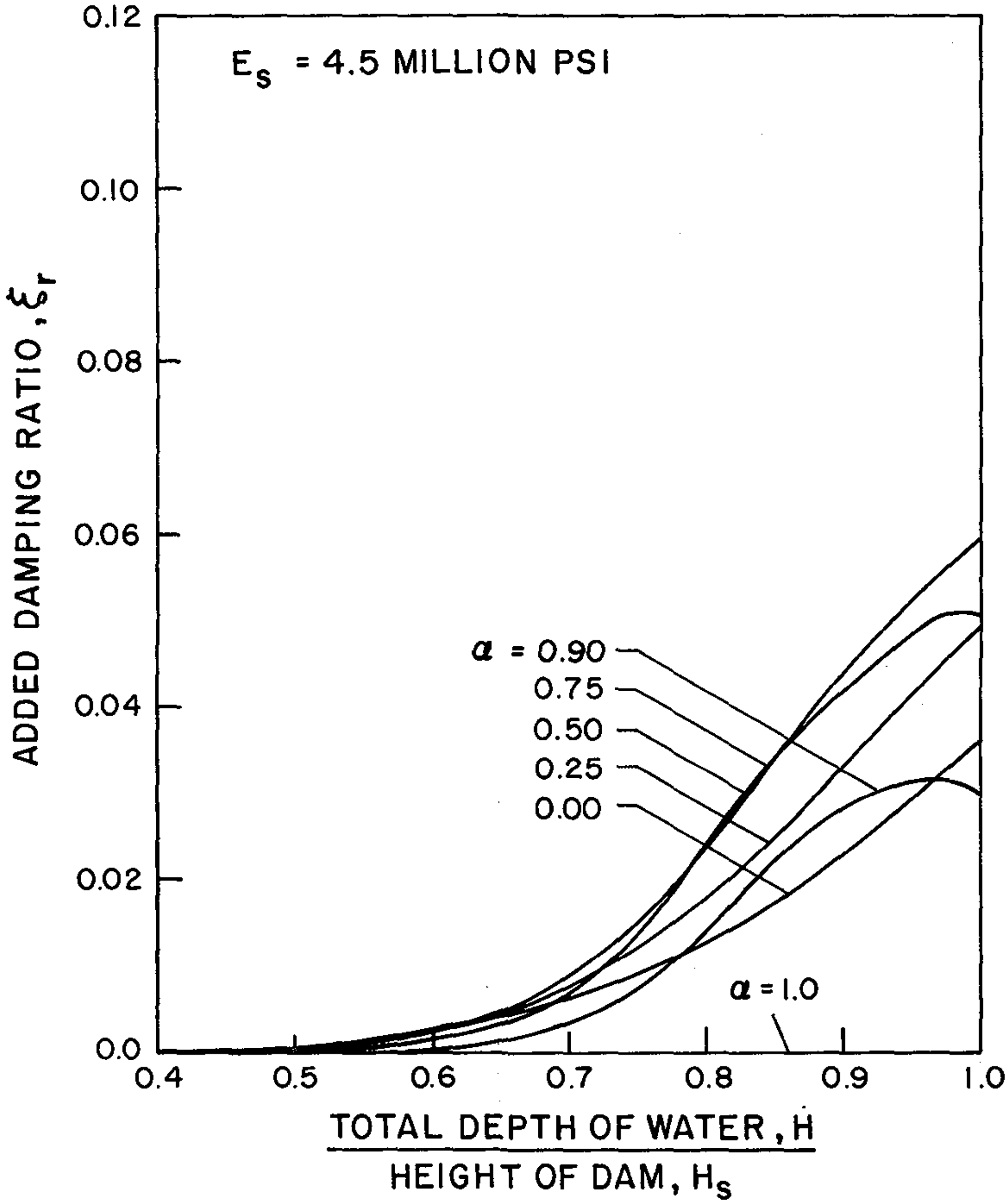


Figure 5(b). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi

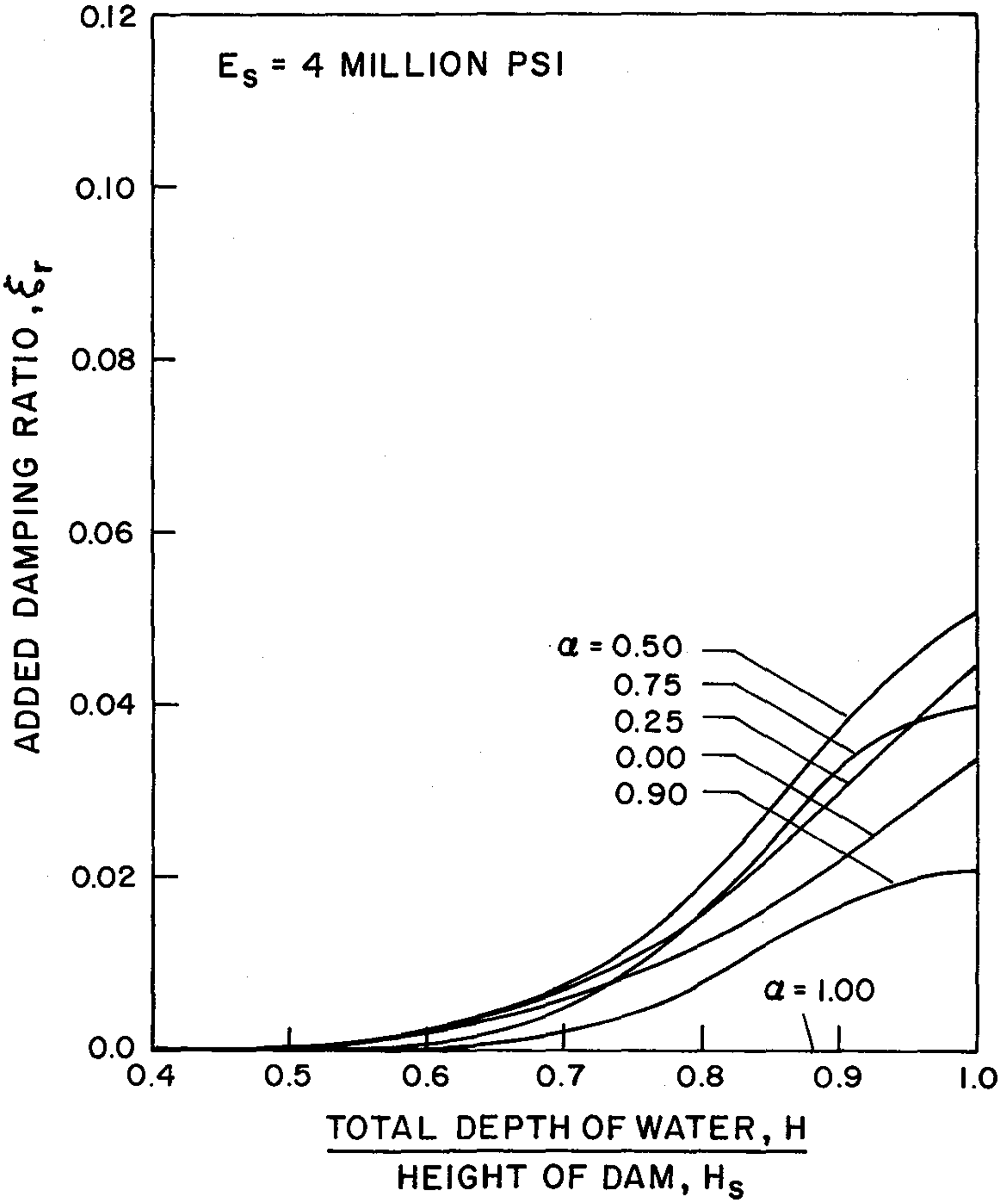


Figure 5(c). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4.0 million psi.

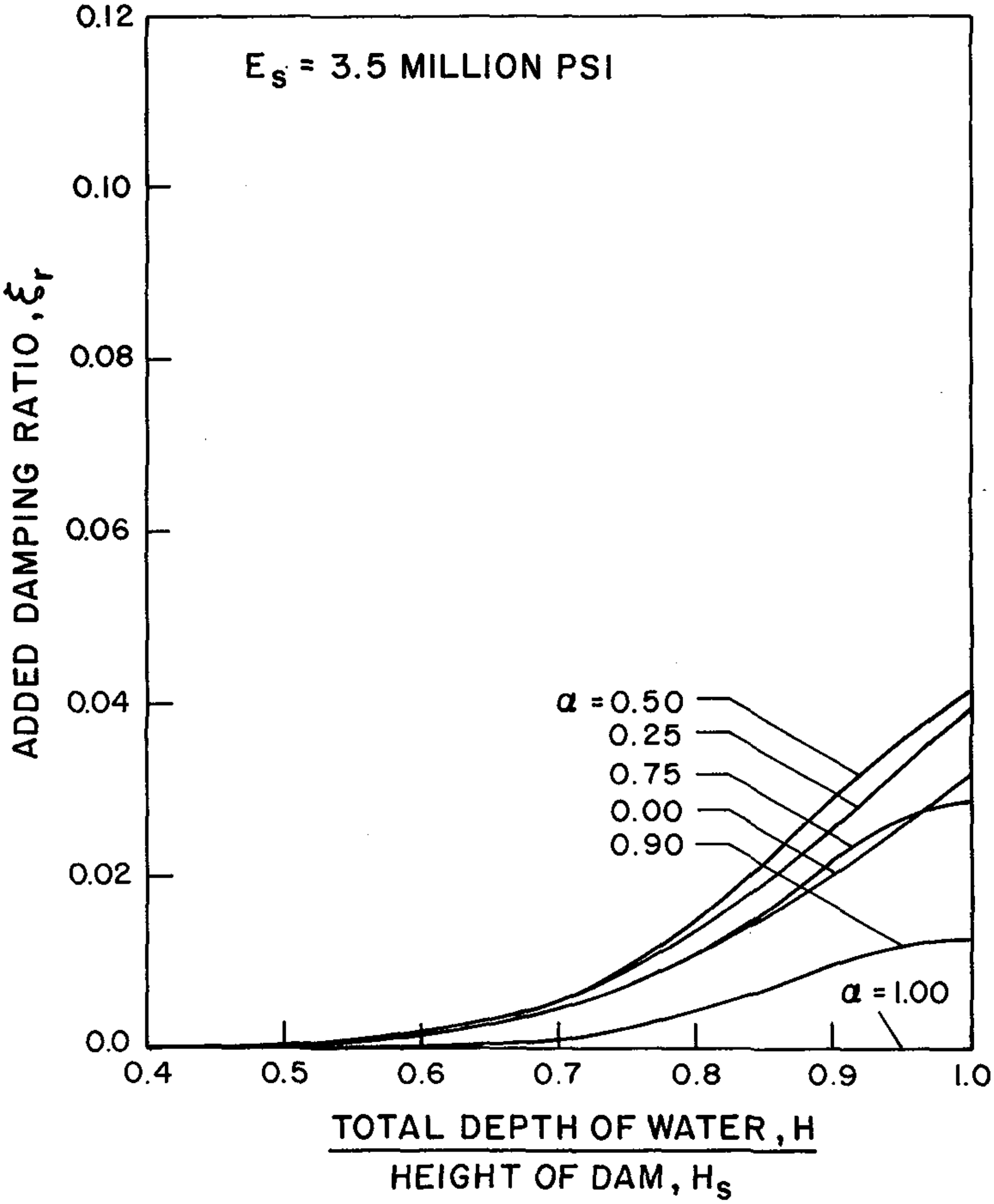


Figure 5(d). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

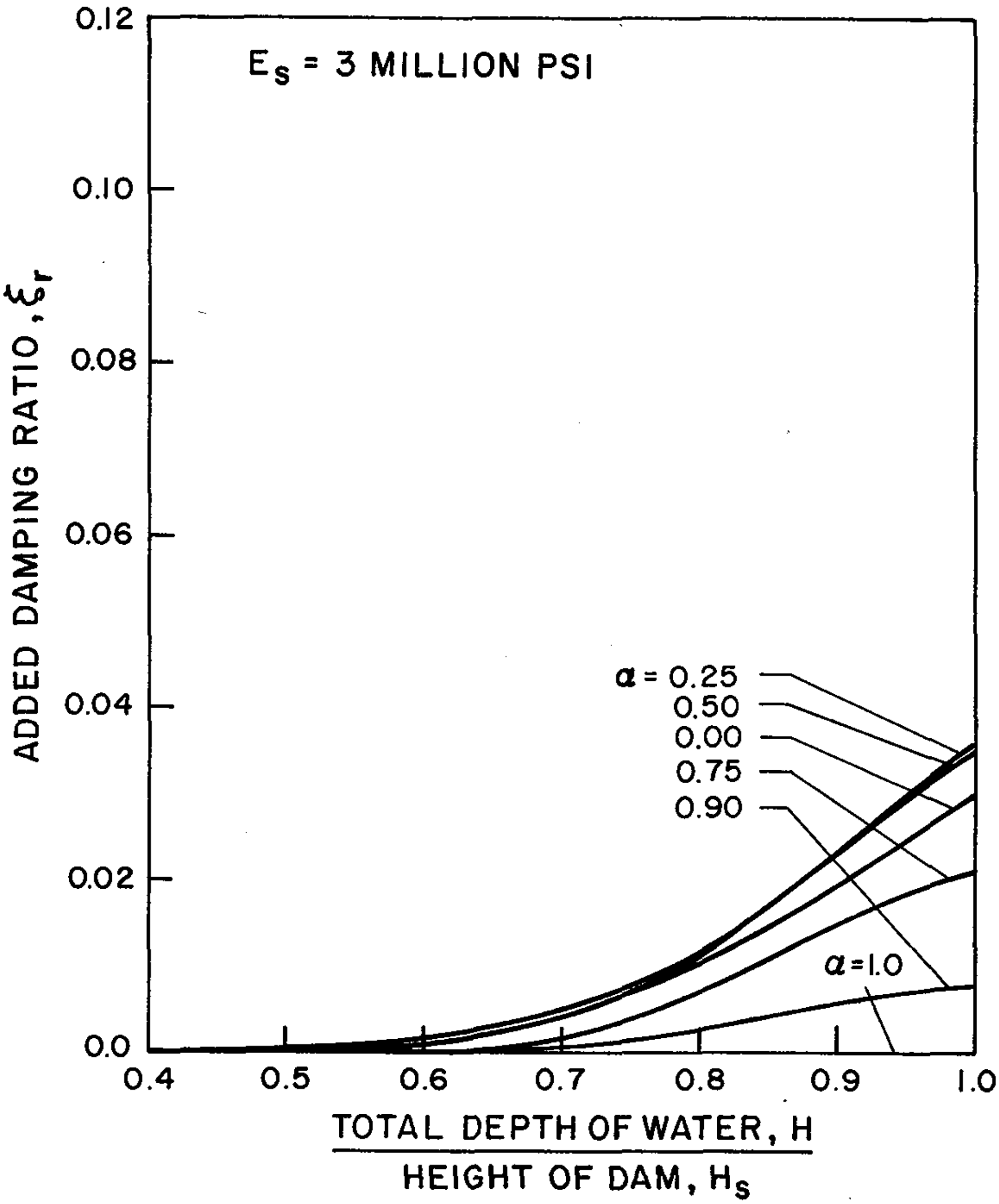


Figure 5(e). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3.0 million psi.

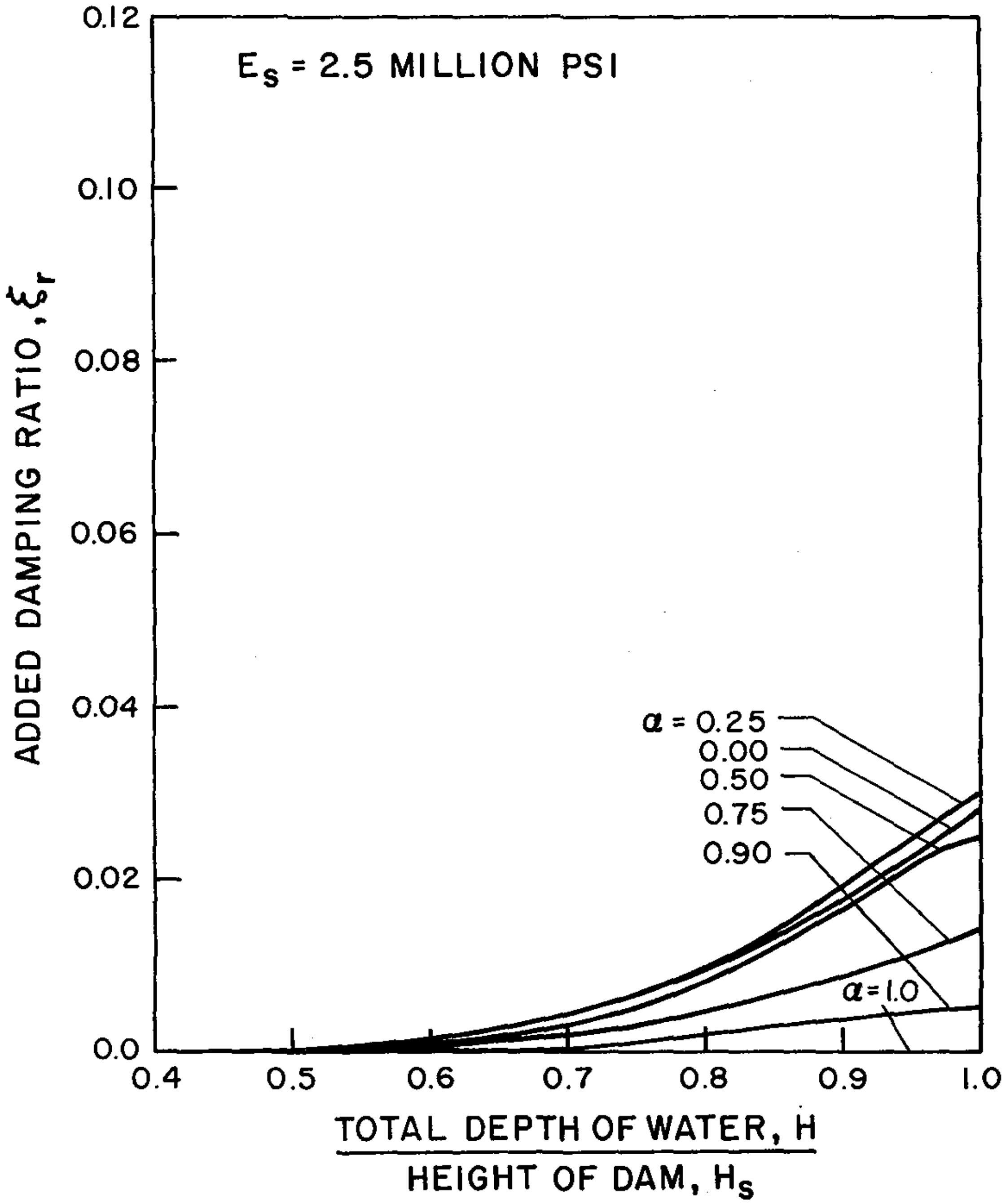


Figure 5(f). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

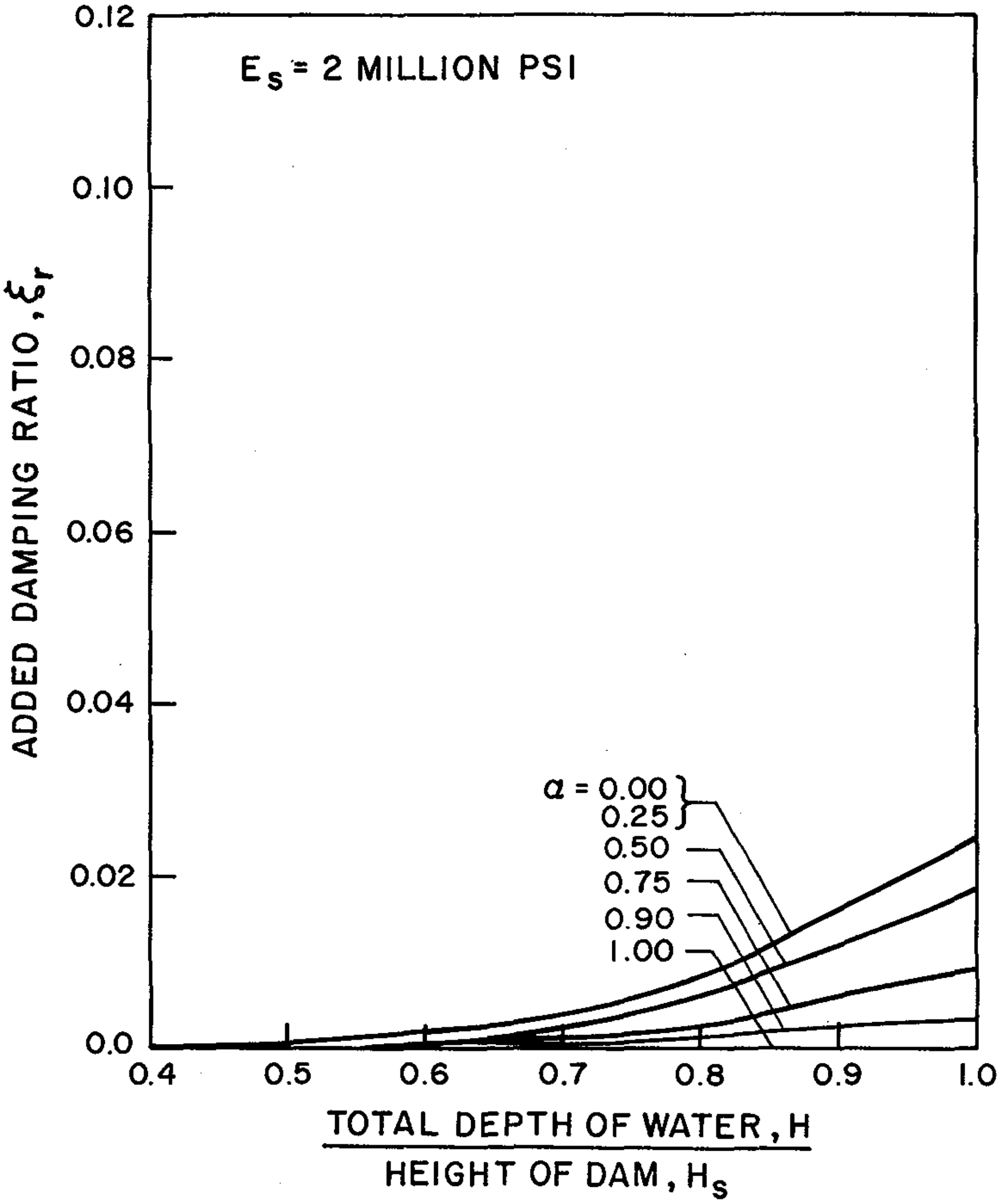


Figure 5(g). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2.0 million psi

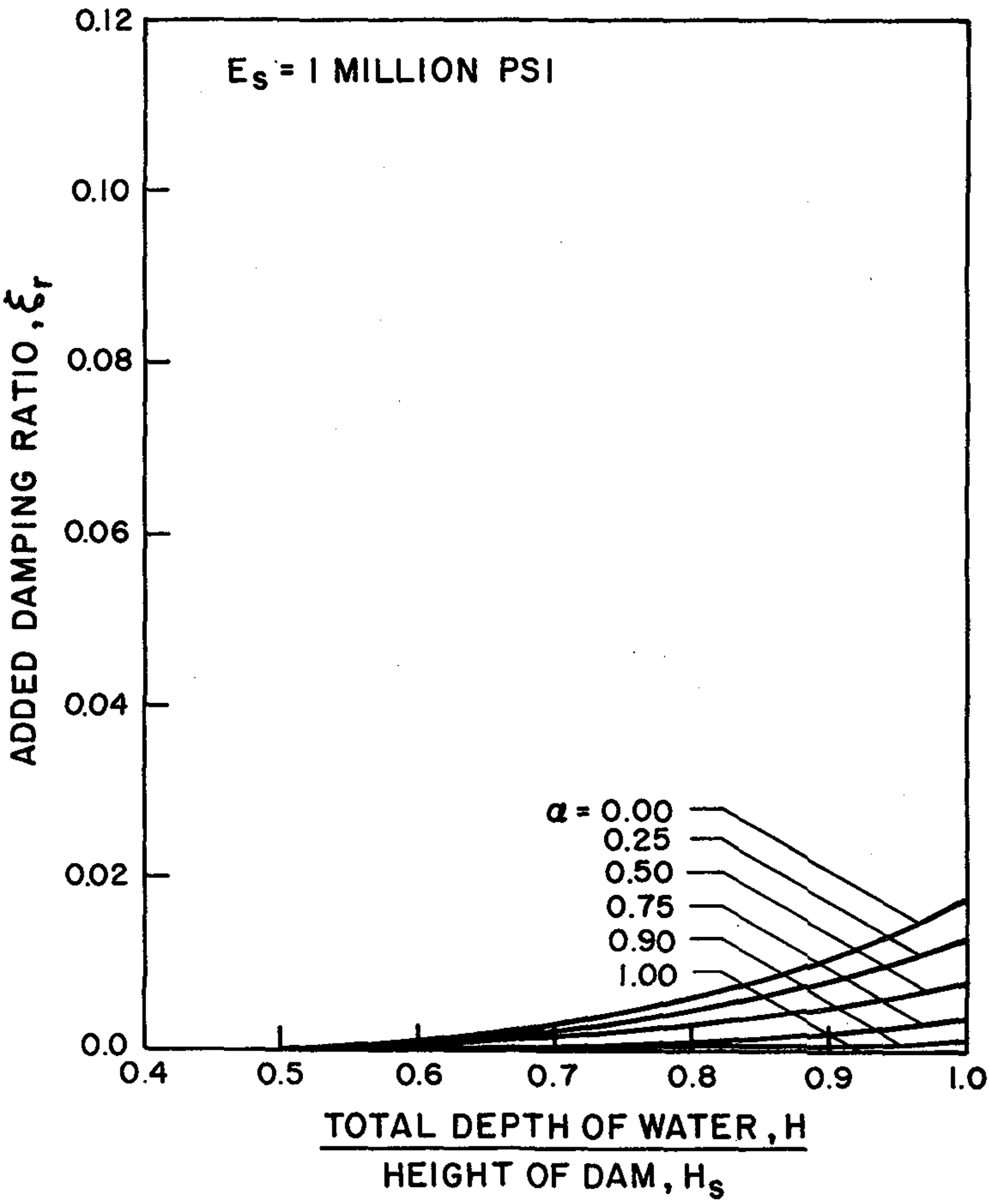


Figure 5(h). Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 1.0 million psi.

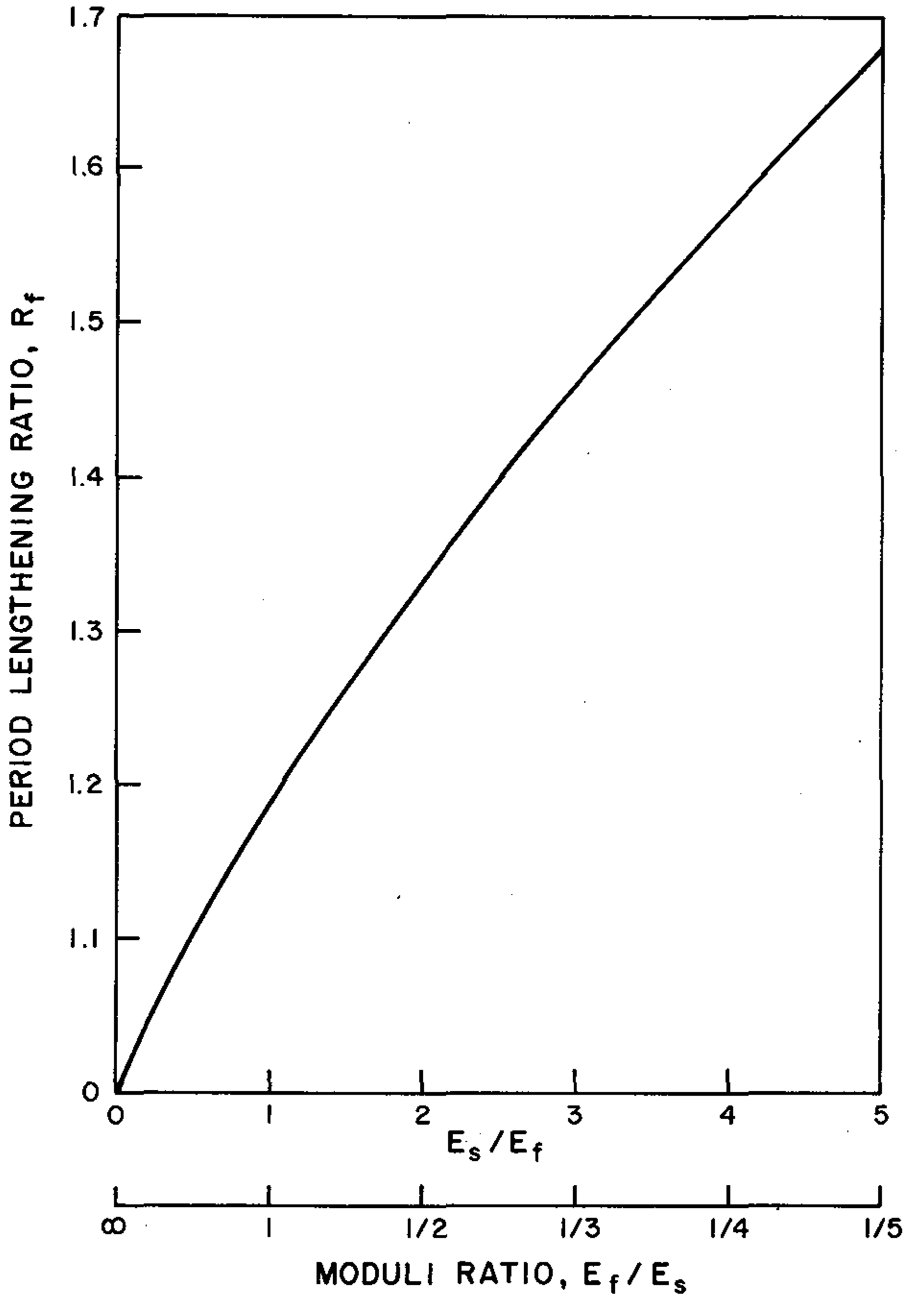


Figure 6. Standard Values for R_f, the Period Lengthening Ratio due to Dam-Foundation Rock Interaction

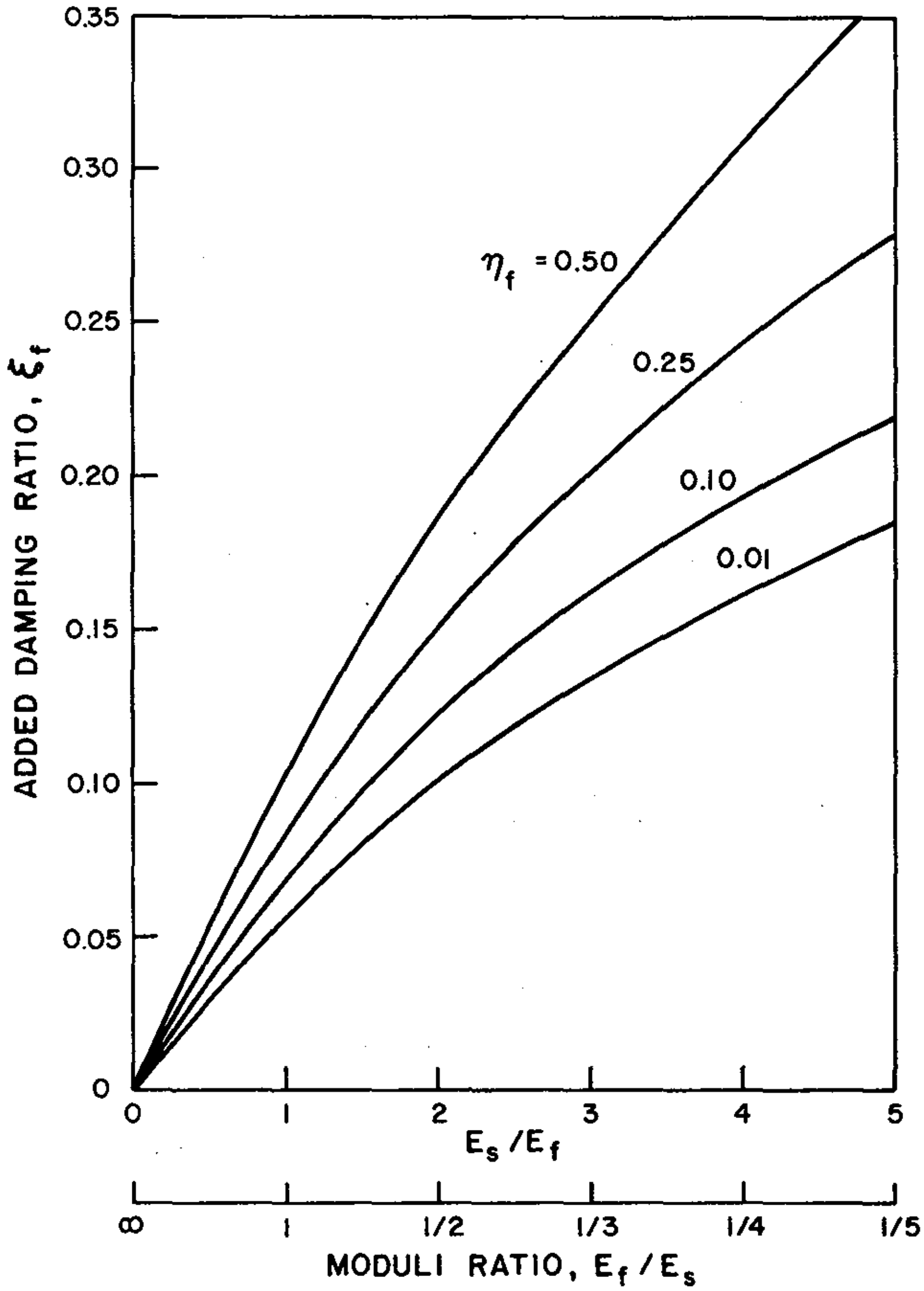


Figure 7. Standard Values for ξ_f, the Added Damping Ratio due to Dam-Foundation Rock Interaction

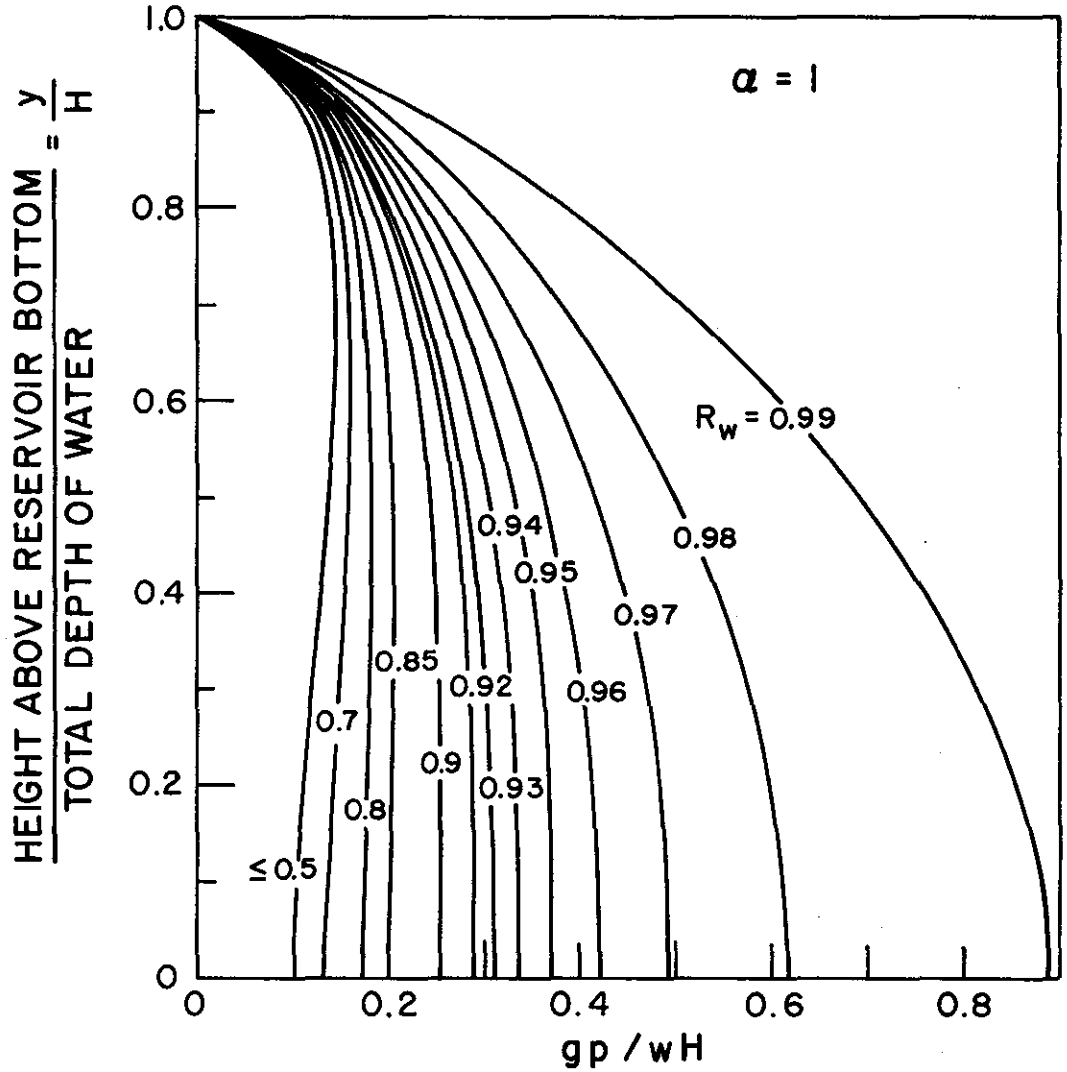


Figure 8(a). Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e. $H/H_s = 1$; $\alpha = 1.00$.

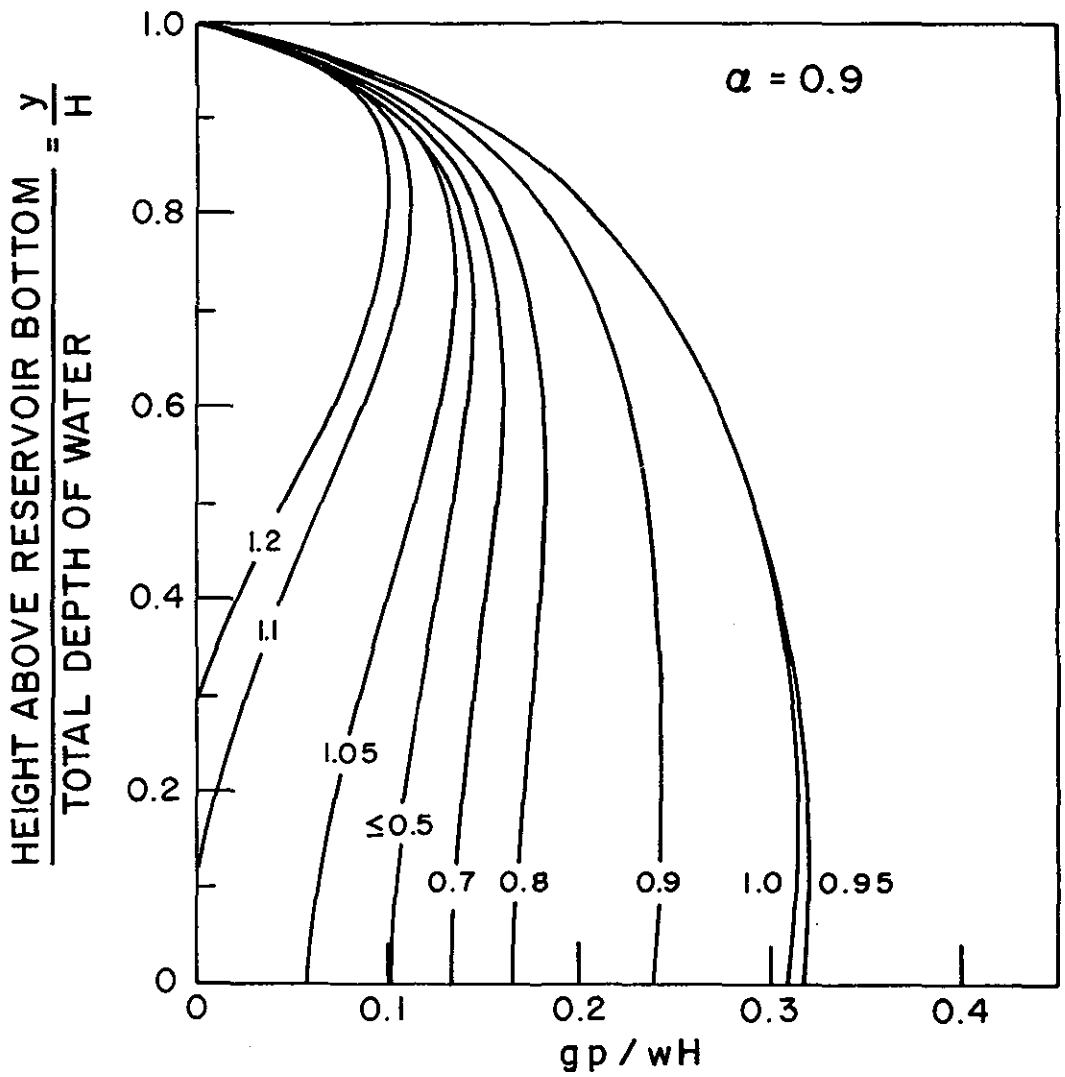


Figure 8(b). Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e. $H/H_s = 1$; $\alpha = 0.90$

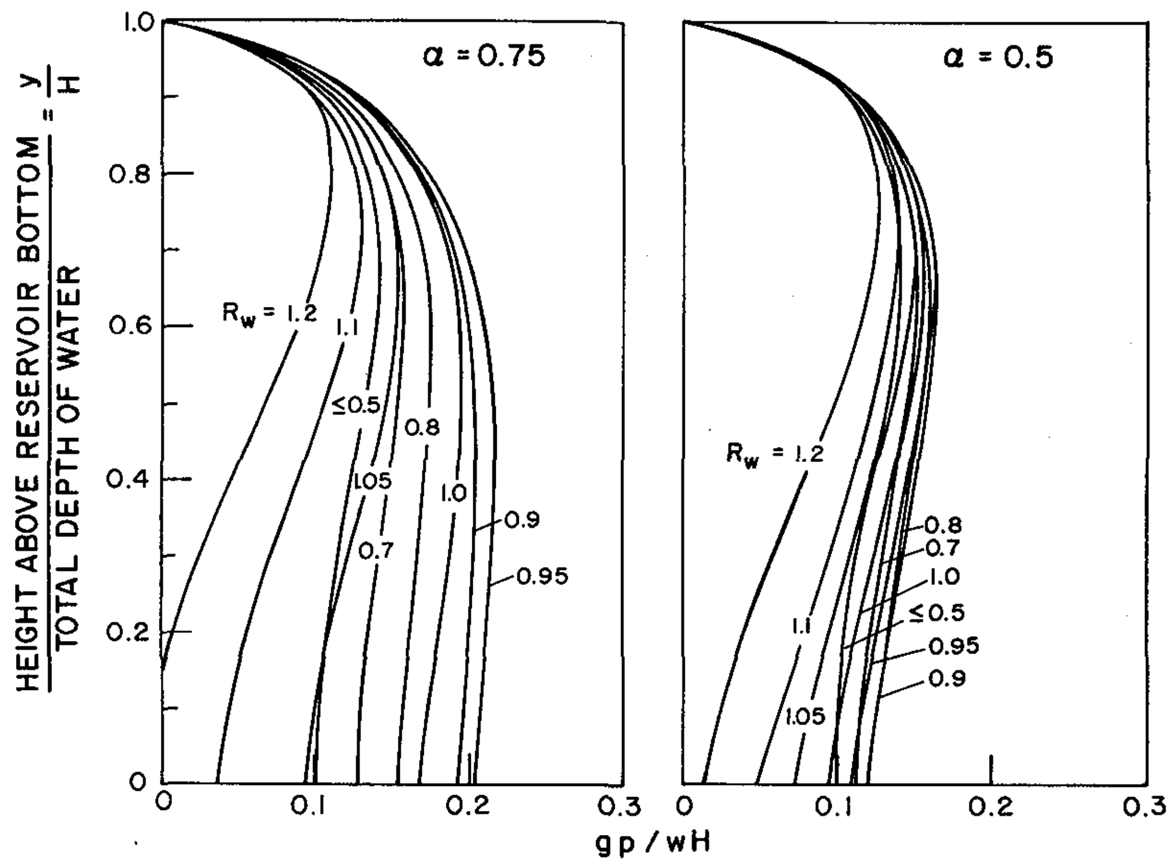


Figure 8(c). Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e. $H/H_s = 1$; $\alpha = 0.75$ and 0.50.

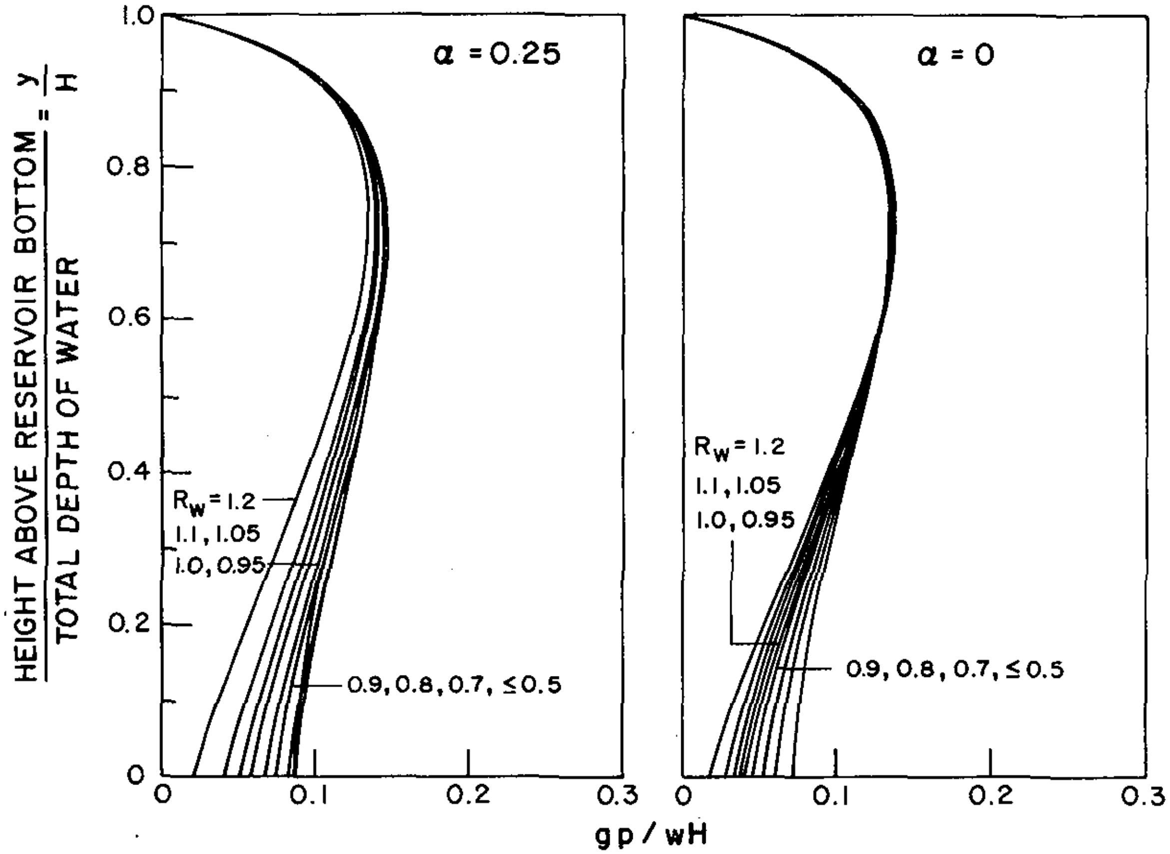


Figure 8(d). Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e. $H/H_s = 1$; $\alpha = 0.25$ and 0.00.

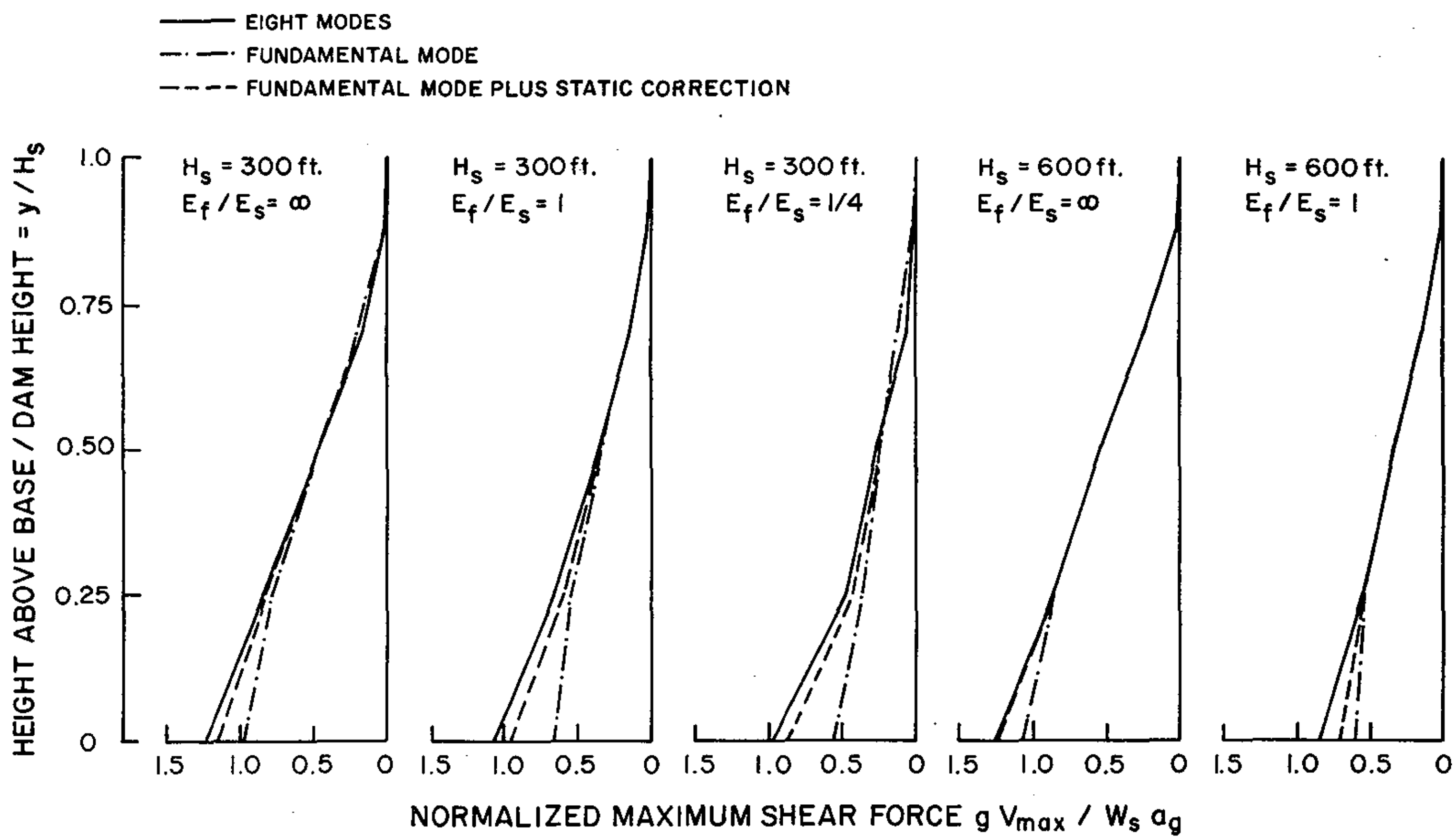


Figure 9. Heightwise Distribution of Maximum Shear Force in Dams with Empty Reservoir due to S69E Component of Taft Ground Motion

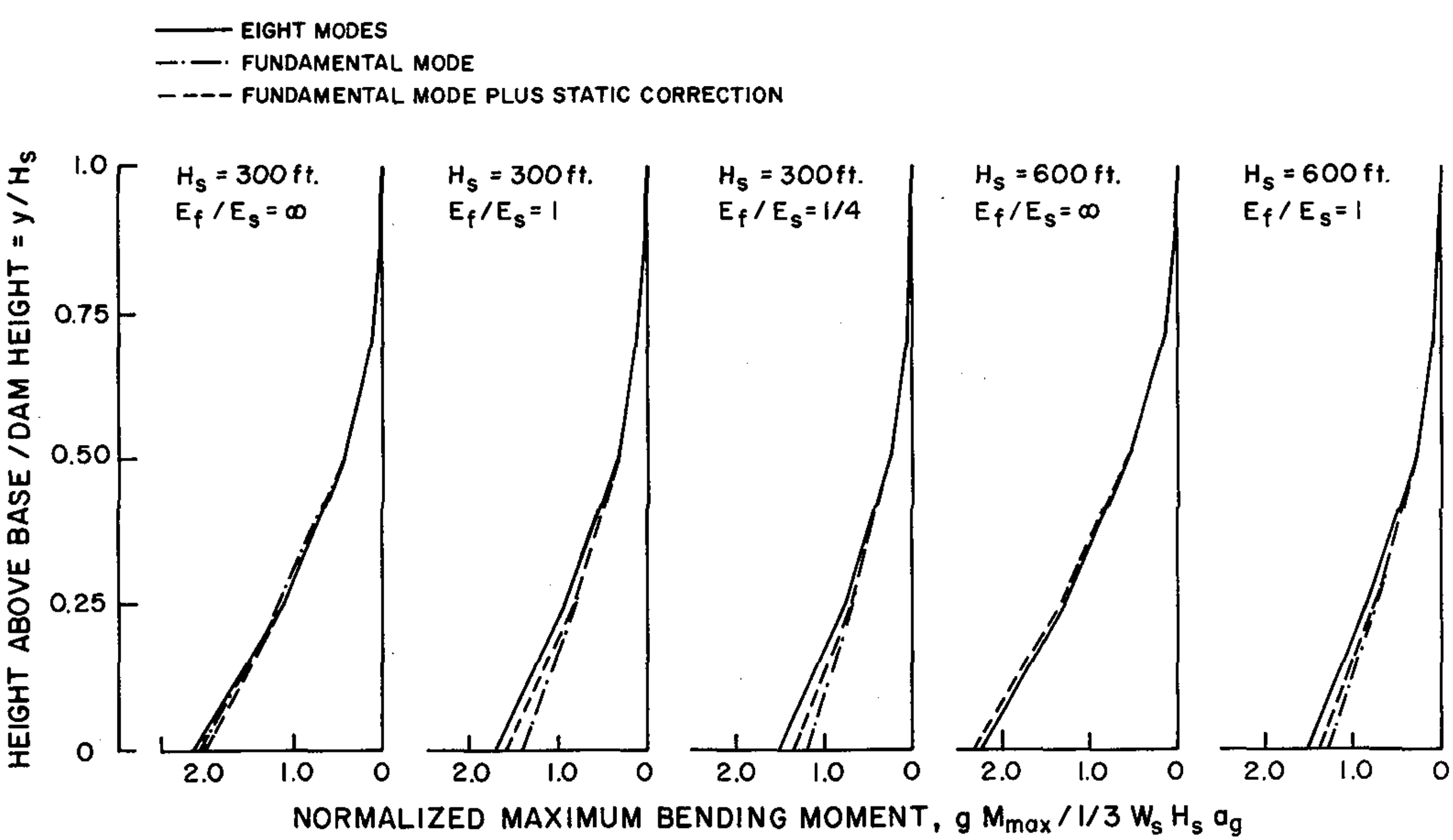


Figure 10. Heightwise Distribution of Maximum Bending Moment in Dams with Empty Reservoir due to S69E Component of Taft Ground Motion

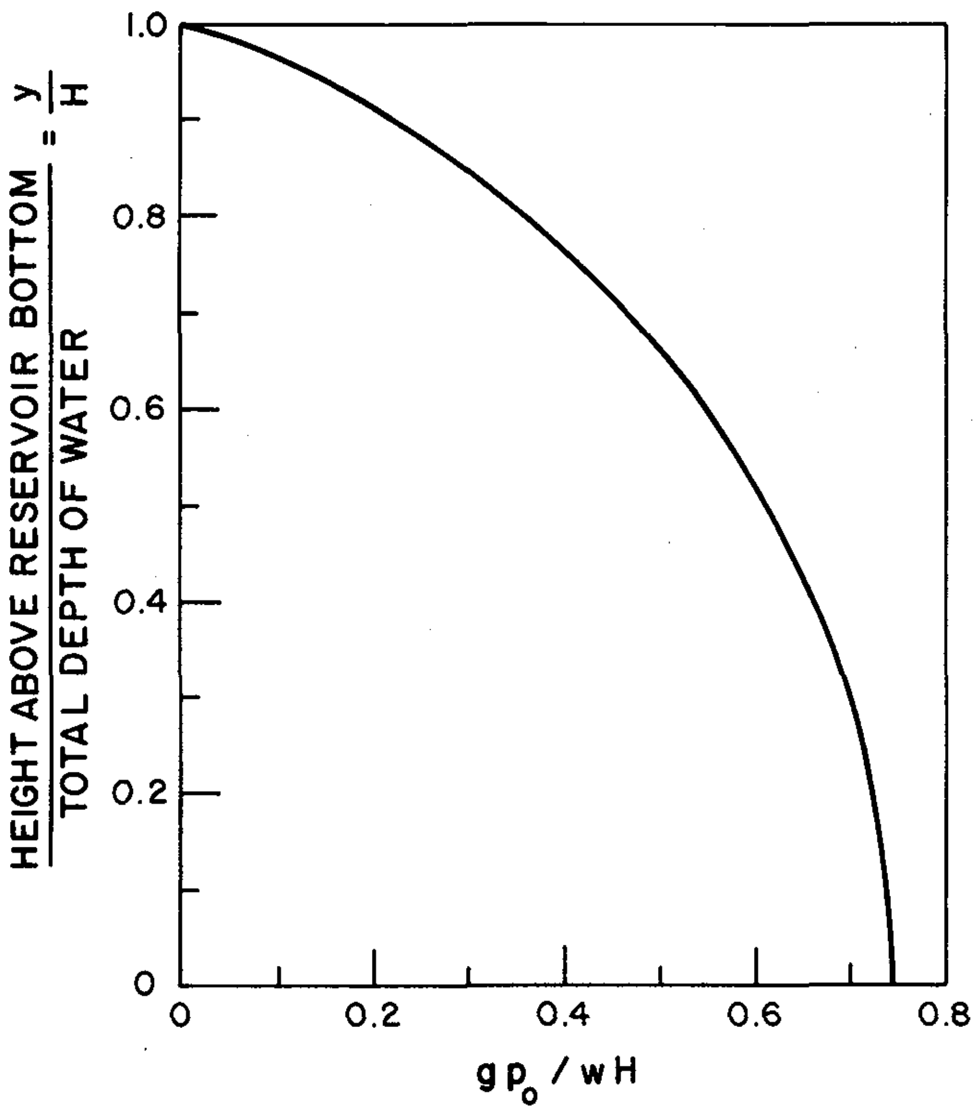


Figure 11. Standard Values for the Hydrodynamic Pressure Function $p_o(\hat{y})$

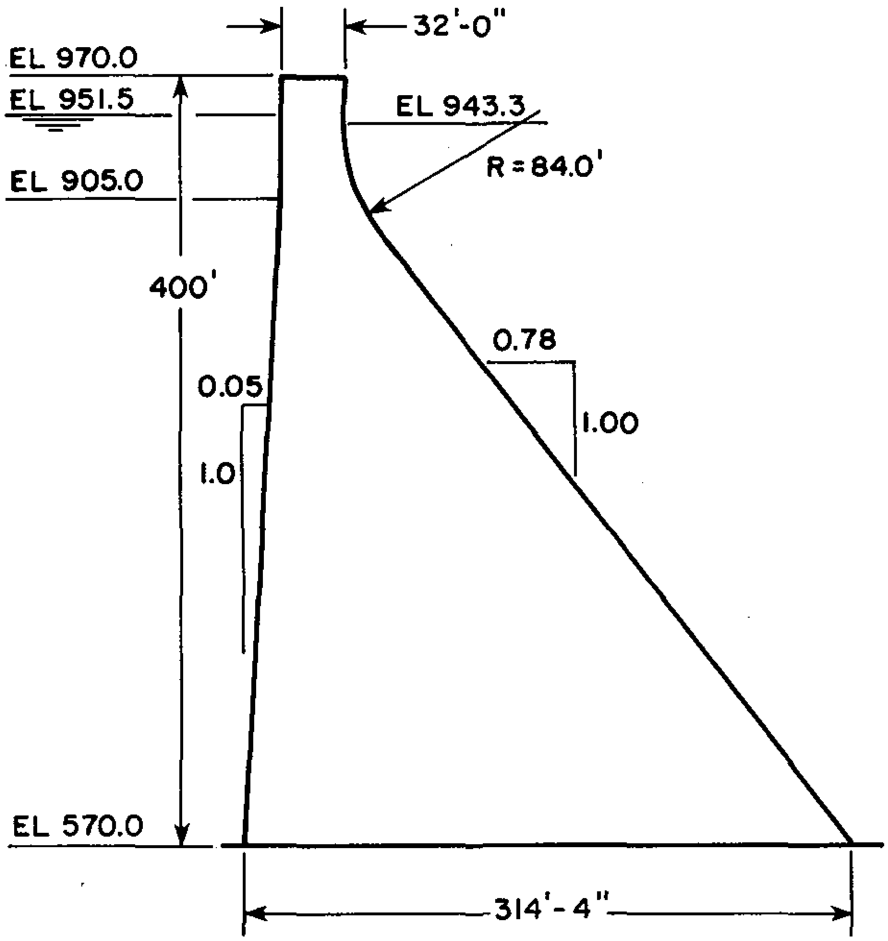


Figure 12. Tallest, Nonoverflow Monolith of Pine Flat Dam

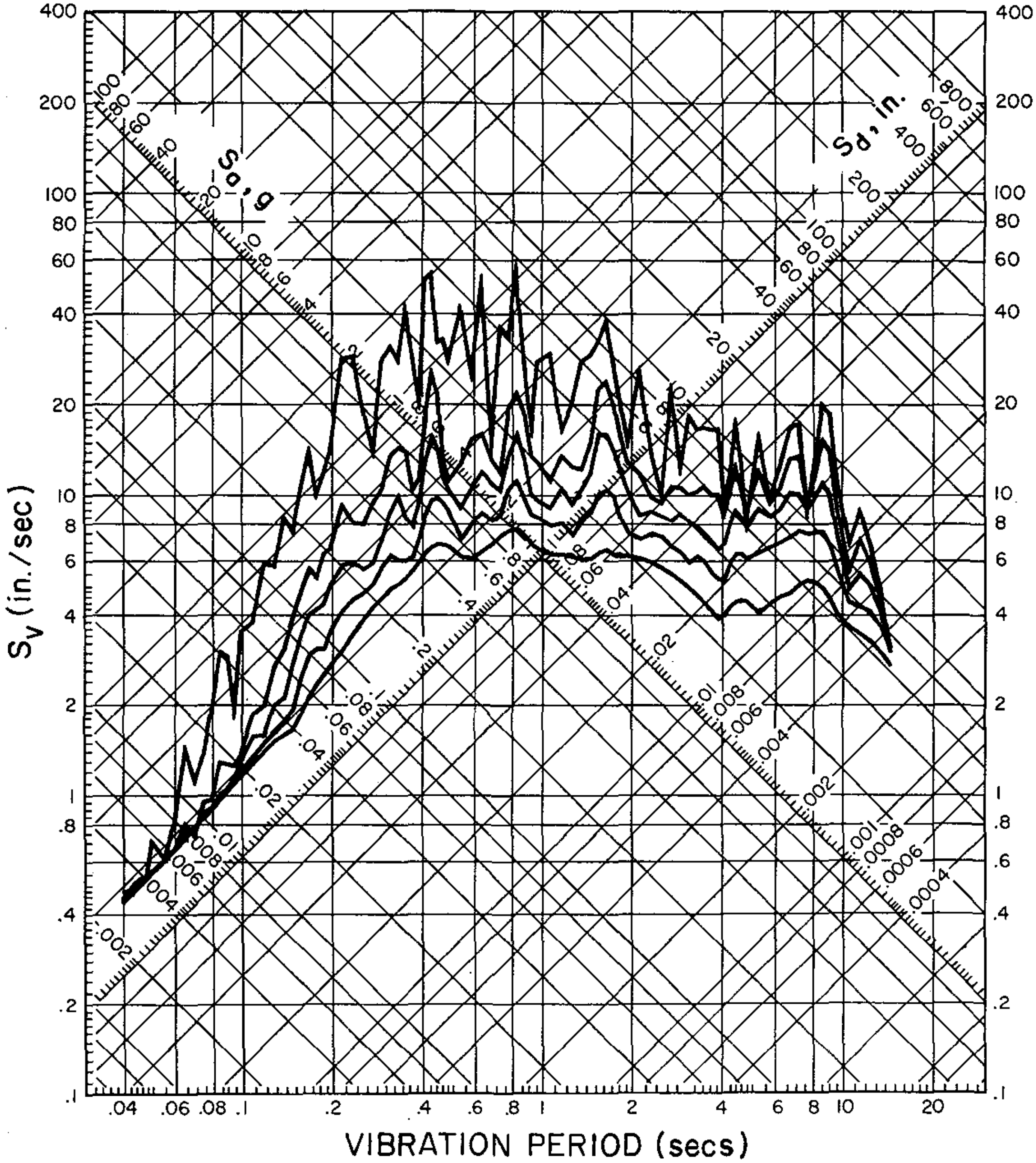


Figure 13. Response Spectrum for the S69E Component of Taft Ground Motion; damping ratios = 0, 2, 5, 10 and 20 percent.

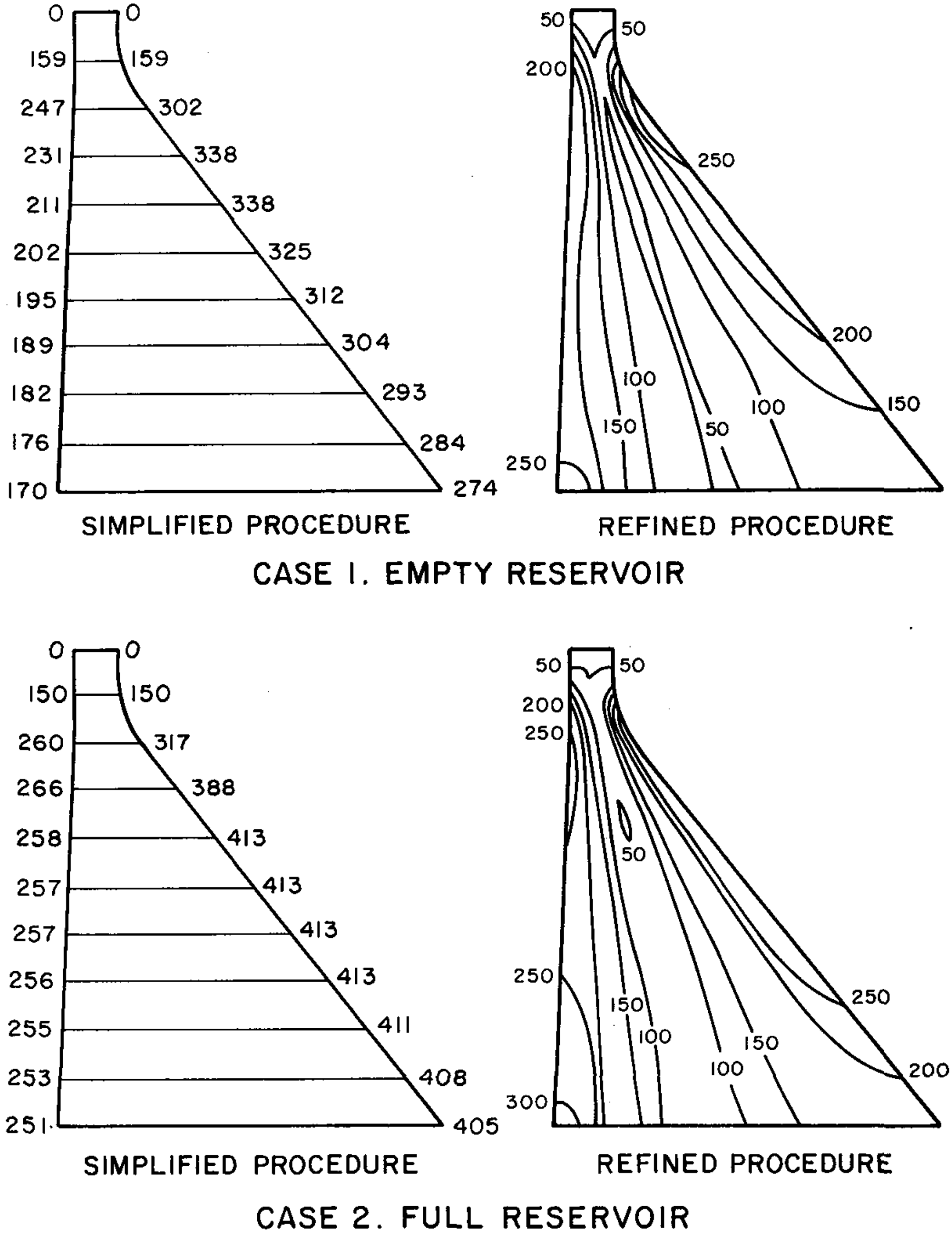


Figure 14. Maximum Principal Stresses (in psi) in Pine Flat Dam on Rigid Foundation Rock due to S69E Component of Taft Ground Motion; Cases 1 and 2. Initial Static Stresses are Excluded.

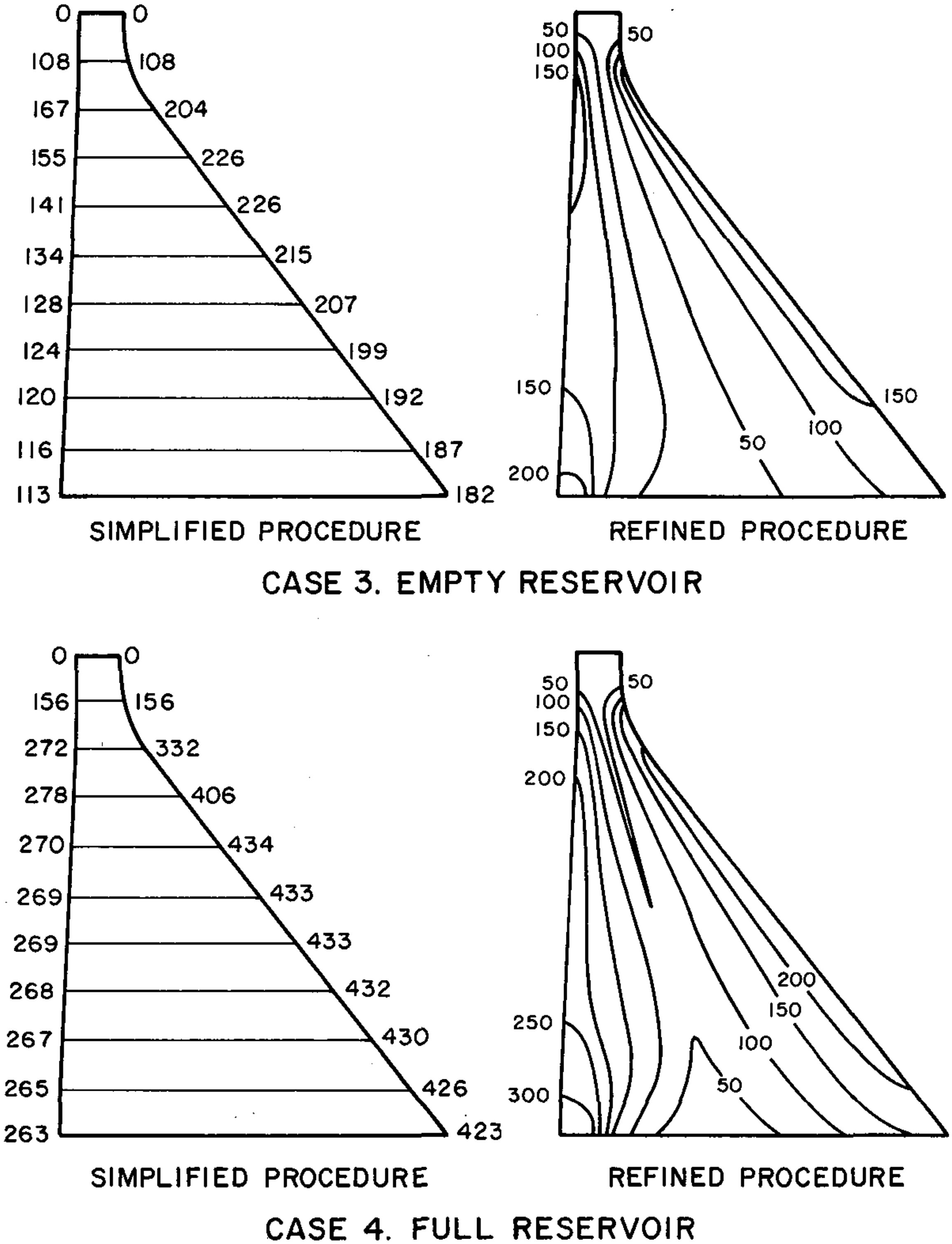


Figure 15. Maximum Principal Stresses (in psi) in Pine Flat Dam on Flexible Foundation Rock due to S69E Component of Taft Ground Motion; Cases 3 and 4. Initial Static Stresses are Excluded.

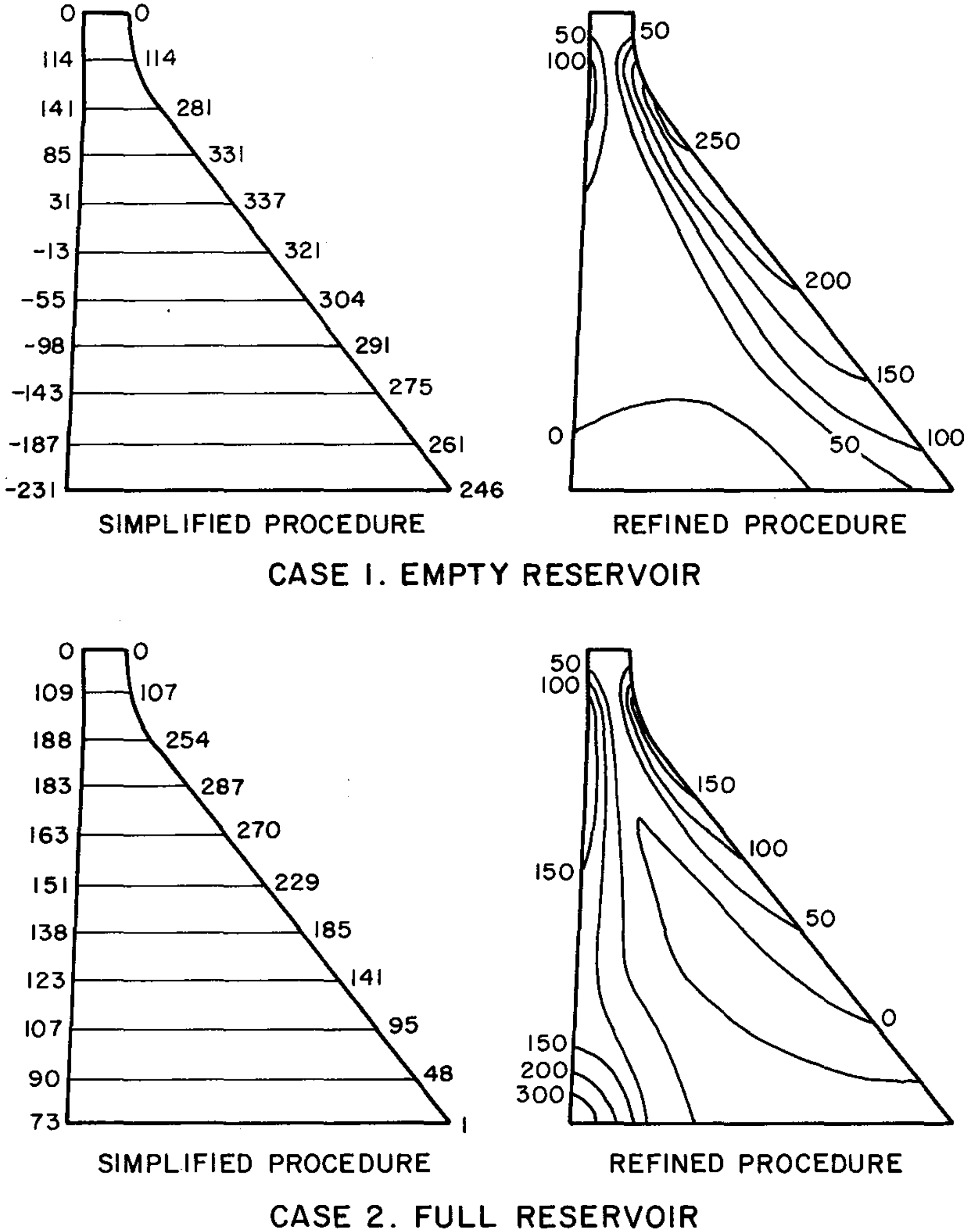


Figure 16. Maximum Principal Stresses (in psi) in Pine Flat Dam on Rigid Foundation Rock due to S69E Component of Taft Ground Motion; Cases 1 and 2. Initial Static Stresses are Included.

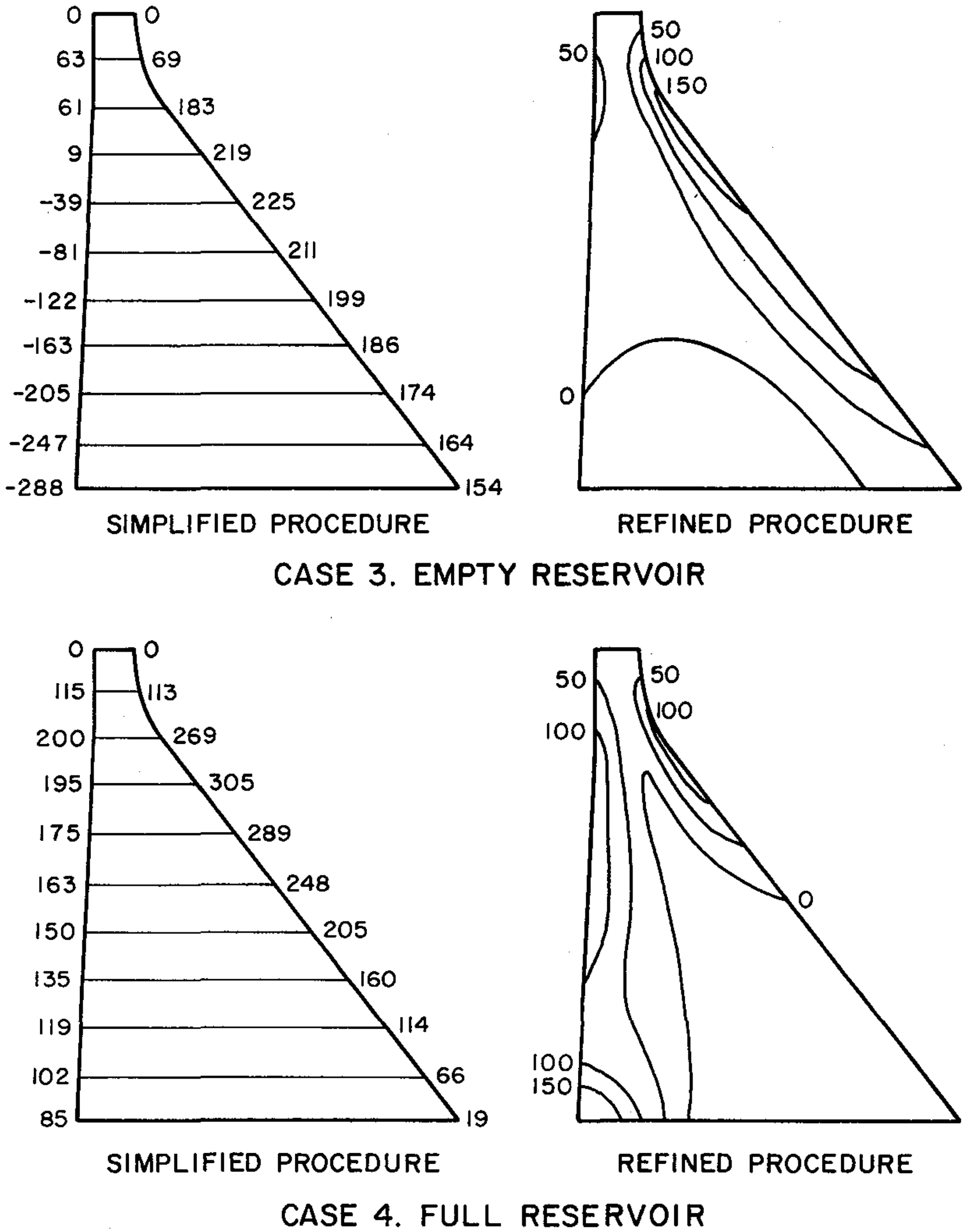


Figure 17. Maximum Principal Stresses (in psi) in Pine Flat Dam on Flexible Foundation Rock due to S69E Component of Taft Ground Motion; Cases 3 and 4. Initial Static Stresses are Included.

APPENDIX A: LIMITATIONS OF THE SIMPLIFIED ANALYSIS PROCEDURE FOR DAM-WATER SYSTEMS

The equivalent SDF system that approximately represents the fundamental mode response of dams was developed in earlier work (see Refs. 4 and 5). In particular, Ref. 4 presented a procedure for computing the vibration properties of the equivalent SDF system for dams on rigid foundation rock including the effects of dam-water interaction and reservoir bottom absorption. The equivalent SDF system representation was shown to be valid for Young's modulus of elasticity of dam concrete, E_s, of 5 million psi and less (Fig. A.1).

Subsequent investigation has demonstrated that E_s = 5 million psi is an upper limit for which an equivalent SDF system can represent the fundamental mode response of dams including the effects of dam-water interaction and reservoir bottom absorption. Furthermore, while an equivalent SDF system adequately represents the fundamental mode response of dam-water systems with E_s = 4 to 5 million psi the simplified expression derived in Ref. 4 for the natural vibration frequency $\tilde{\omega}_r$ of the equivalent SDF system is not reliable when reservoir bottom absorption effects are included. This appendix describes further the limitations of the simplified analysis procedure for dam-water systems.

Limitations of the Equivalent SDF System

The exact fundamental mode response of the standard dam monolith due to unit harmonic horizontal ground acceleration is presented in Figs. A.2 and A.10 for E_s = 6 million psi and various values of the wave reflection coefficient, α, for the reservoir bottom materials and depth ratio, H/H_s, of the impounded water. The absolute value of the displacement at the dam crest was evaluated by Eqs. 1 and 6 in Ref. 4. For $H/H_s \geq 0.75$ and $\alpha \geq 0.90$, the response functions exhibit two resonant peaks (Figs. A.2 to A.4). The peak at the lower excitation frequency has a larger magnitude than the second peak at the higher frequency. This phenomenon, described in Ref. 10, is due to interaction between the compressible water and stiff dam. Also described in Ref. 10 was the observation that for $\alpha \leq 0.50$ the added damping associated with reservoir bottom absorption results in the two resonant peaks coalescing into a single resonant peak at an intermediate frequency value (see Figs. A.8 to A.10). Thus the response function exhibits one dominant resonant peak if the response bottom materials are highly absorptive ($\alpha \leq 0.50$) or close to nonabsorptive ($\alpha \geq 0.90$). Because the equivalent SDF system can satisfactorily approximate only a single resonant peak (Fig. A.1), it is effective in representing the dam response for $\alpha \geq 0.90$ and $\alpha \leq 0.50$ when E_s = 6.0 million psi.

However, the response functions in Figs. A.5 to A.7 show that dam-water systems with an intermediate absorptiveness of the reservoir bottom materials, $0.50 < \alpha < 0.90$, do not have a well-defined resonant peak. This is particularly true in the case of $\alpha = 0.75$ for H/H_s between 0.85 and 1.0, where the frequency response function shows nearly flat response over a wide range of excitation frequencies. Clearly the representation of this broad-bandwidth response function by a SDF system is prone to substantial error.

Limitations of the Approximate Expression for $\mathbf{R_r}$

The period lengthening ratio R_r is defined as $\tilde{T}_r/T_1$ where $\tilde{T}_r = 2\pi/\tilde{\omega}_r$, the natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam with impounded water, and T_1 is the fundamental vibration period of the dam with empty reservoir. In the simplified procedure, an approximate value for $\tilde{\omega}_r$ is determined from Eq. 18 of Ref. 4.

In preparation of this report, it was discovered that this approximate expression for $\tilde{\omega}_r$ is not reliable for dams with E_s greater than 4 million psi. The assumption behind the expression is that the resonant frequency of the dam-water system is approximately the excitation frequency that makes the real-valued component of the denominator in the exact fundamental mode response function zero. This is a good approximation except for relatively stiff dams, where reservoir bottom absorption may reduce the added hydrodynamic mass to such an extent that the real-valued component of the denominator does not equal zero in the excitation frequency range of interest. For such cases, the resonant frequency should be determined directly from the exact fundamental mode response function, as opposed to the approximate procedure that seeks to find a zero of the real-valued component in the denominator of the response function. Once the resonant frequency is determined, the added hydrodynamic damping ratio ξ_r can be evaluated, as before, by Eq. 20 in Ref. 4. This improved procedure was followed for computing the standard values presented in this report for the period lengthening ratio, R_r, and added hydrodynamic ratio, ξ_r, for dams with E_s = 5.0 and 4.5 million psi, and all H/H_s values; E_s = 4.0 million psi with $H/H_s \geq 0.80$; and E_s = 3.5 million psi with $H/H_s \geq 0.85$. The data for other values of E_s and H/H_s were generated by the approximate expression presented in Ref. 4.

Comments on Added Hydrodynamic Damping

Figures A.2 to A.10 show how the frequency response functions of dams vary with the depth of impounded water; these results have not been presented earlier. It is apparent that the resonant frequency of a dam-water system decreases with increasing water depth because of the increasing added hydrodynamic mass (4). The variation of the resonant response magnitude with water depth depends on the effective damping of the dam-water system which is affected by the absorptiveness of the reservoir bottom materials. As shown in Figs. A.2 to A.4, for essentially non-absorptive reservoir bottom materials ($\alpha \geq 0.90$), the resonant response magnitude does not increase monotonically with increasing water depth (except for α = 1.0). On the other hand, Figs. A.8 to A.10 show that, for highly absorptive reservoir bottom materials ($\alpha \leq 0.5$), the resonant response magnitude decreases with increasing water depth. In the intermediate range of reservoir bottom absorptiveness ($0.5 < \alpha < 0.9$), the magnitude of the resonant peak is not very sensitive to the water depth for reservoirs at least three-quarters full (Figs. A.5 to A.7).

The trend in resonant response magnitude can be explained in terms of the overall damping of the dam-water system. There are two sources of damping in the system: damping of the dam alone, and hydrodynamic damping due to propagation of hydrodynamic pressure waves upstream and refraction into the absorptive reservoir bottom materials (4). The combination of opposing trends in the two contributions to effective damping ratio determine the variation of the resonant response magnitude with depth of the impounded water. As the water depth increases, the effectiveness of the structural damping decreases (3,4), tending to increase the resonant response, but the added hydrodynamic damping ratio generally increases, tending to decrease the resonant response. The increase in the added hydrodynamic damping depends on the absorptiveness of the reservoir bottom materials. For relatively absorptive reservoir bottom materials, the added hydrodynamic damping ratio is greater than the reduction in the effective structural damping, so the overall damping ratio increases as the water depth increases (Figs. A.8 to A.10). The variation with water depth is not as straightforward for relatively non-absorptive reservoir bottom materials, in that the added hydrodynamic damping ratio may increase with water depth and then decrease slightly for $0.90 < H/H_s \leq 1.0$. This latter trend develops because with a nearly full reservoir the resonant frequency of a stiff dam is reduced far enough below ω_1^r, the fundamental resonant frequency of the impounded water alone, that the added hydrodynamic damping is due mainly to refraction of pressure waves into the relatively non-absorptive reservoir bottom materials, thus limiting the energy radiation. As the water depth decreases below $H/H_s = 0.90$, the fundamental resonant frequency shifts closer to ω_1^r, thus allowing more radiation of energy by propagation of pressure waves upstream. This phenomenon, which is pronounced for $E_s = 6$ million psi, further explains the rather complicated looking variation of the added hydrodynamic damping ratio presented in Refs. 4 and 5 and in Fig. 5 of this report for other values of E_s.

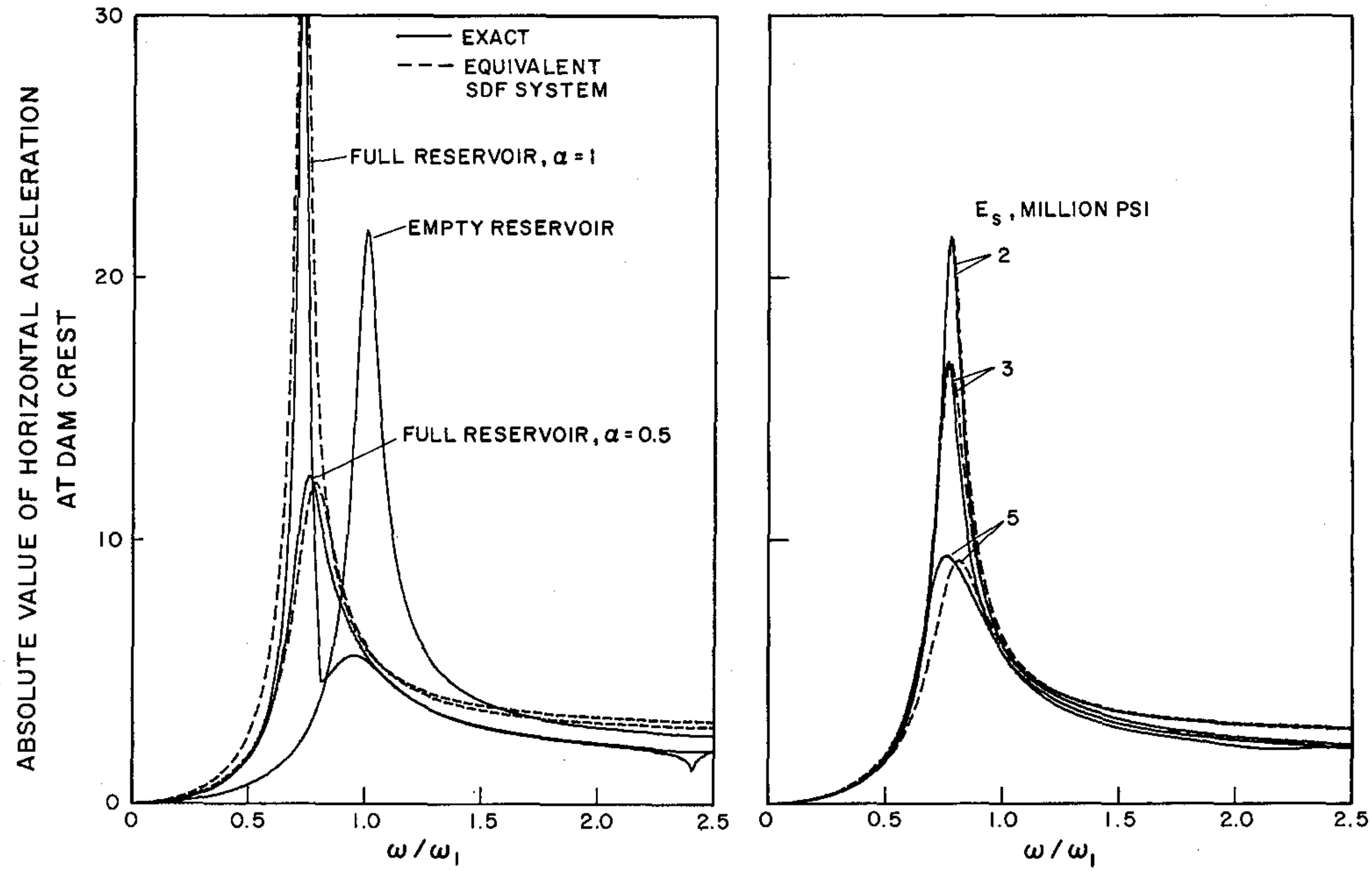


Figure A.1. Comparison of Exact and Equivalent SDF System Response of Dam on Rigid Foundation Rock with Impounded Water due to Harmonic Horizontal Ground Motion.

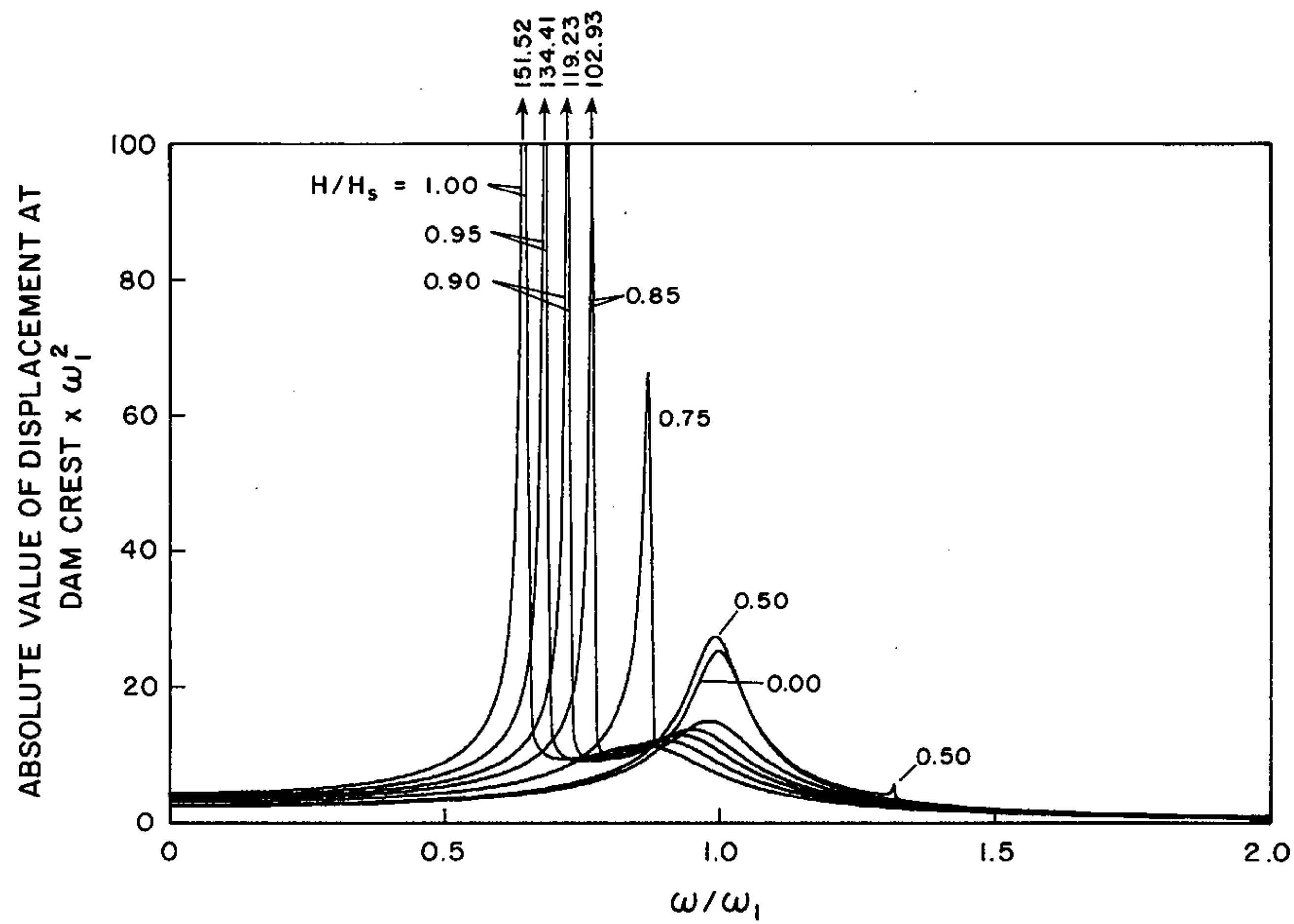


Figure A.2. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 1.0

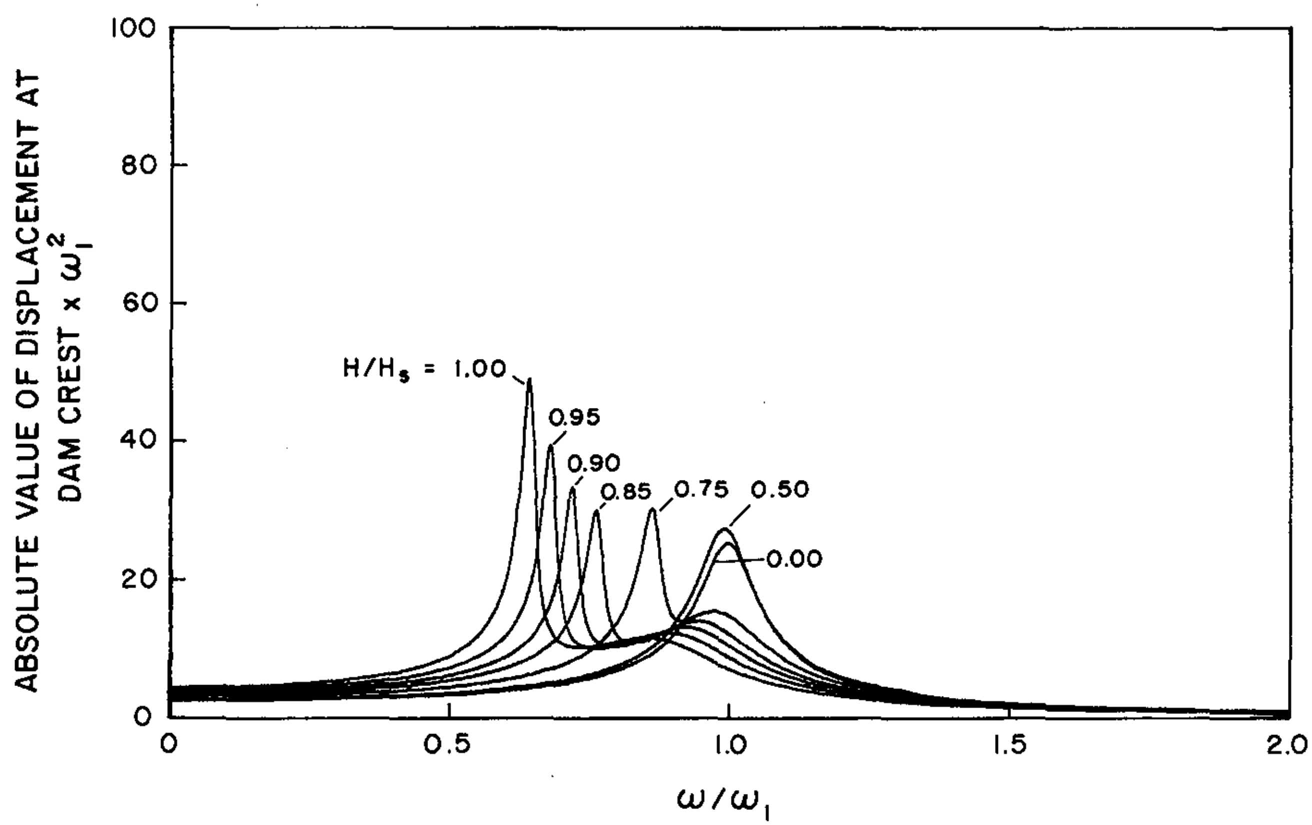


Figure A.3. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.95

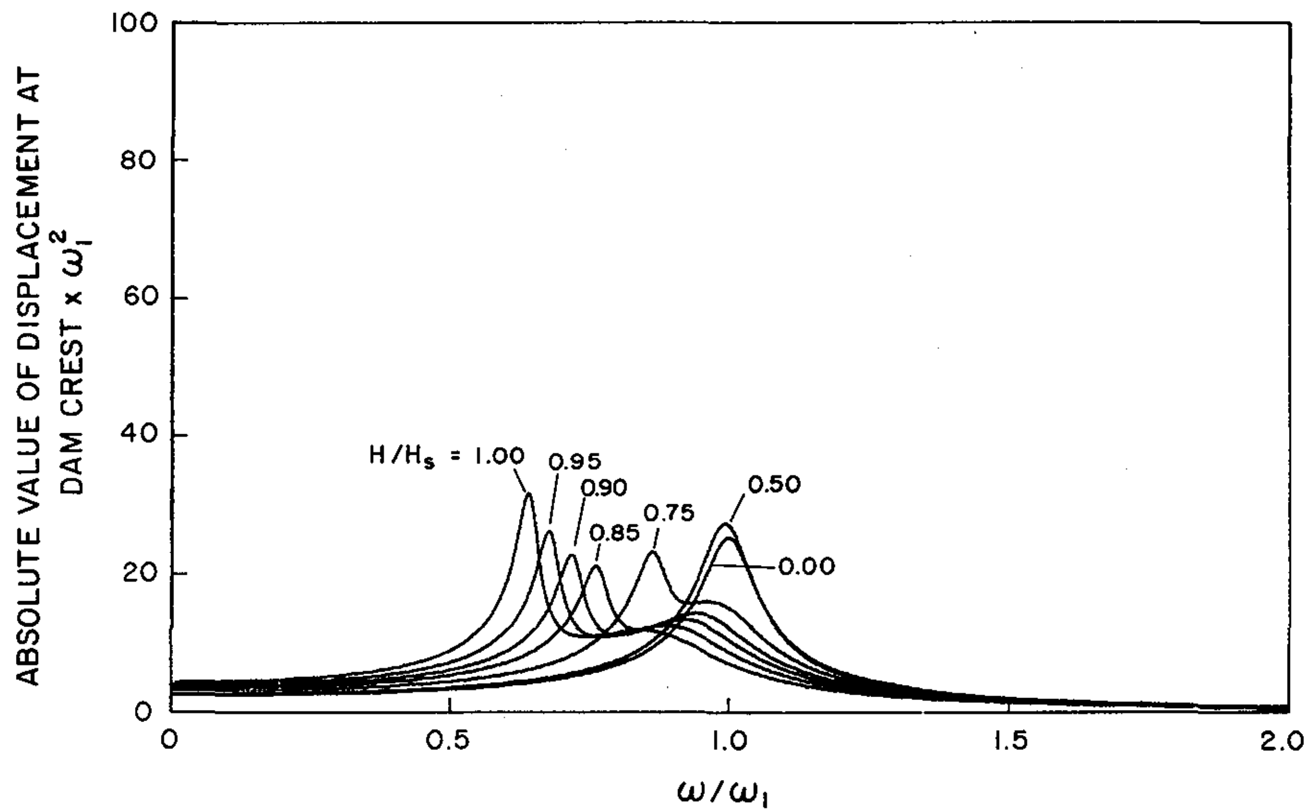


Figure A.4. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.90

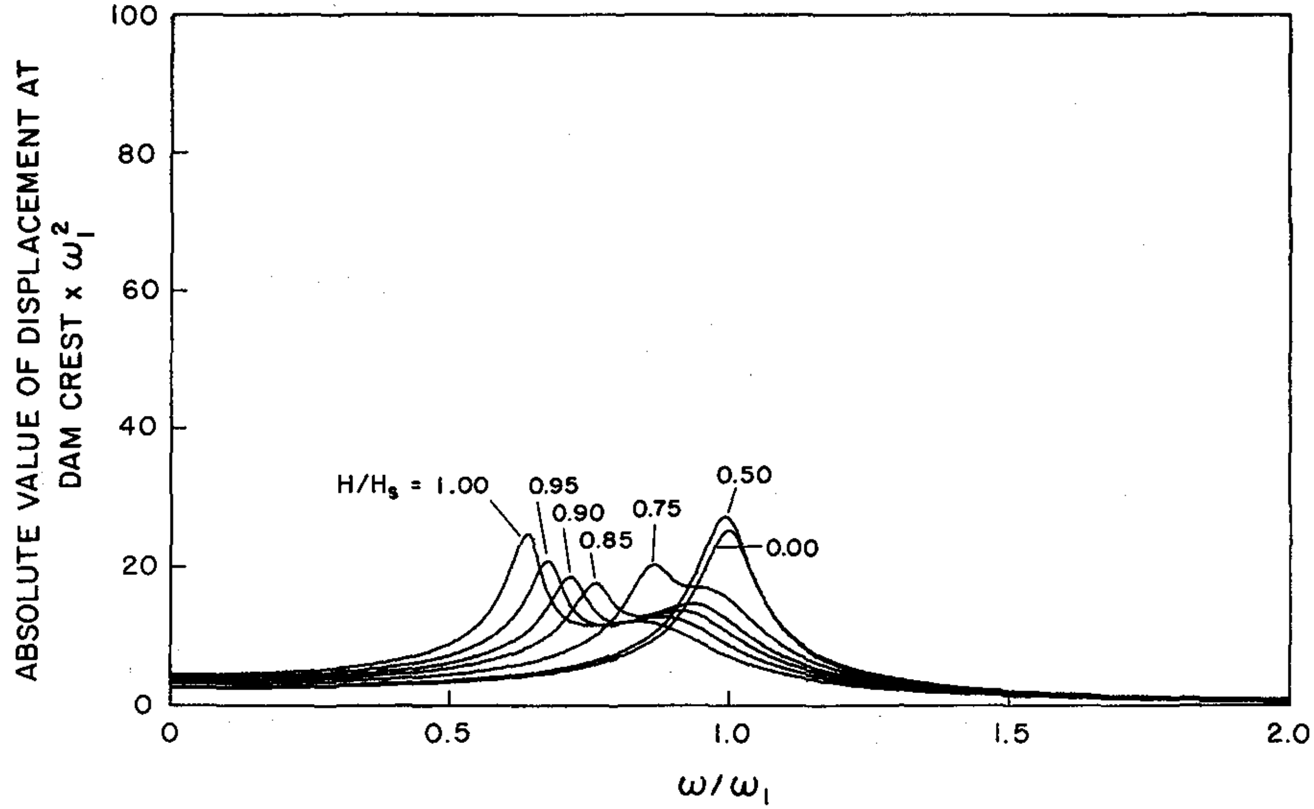


Figure A.5. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.85

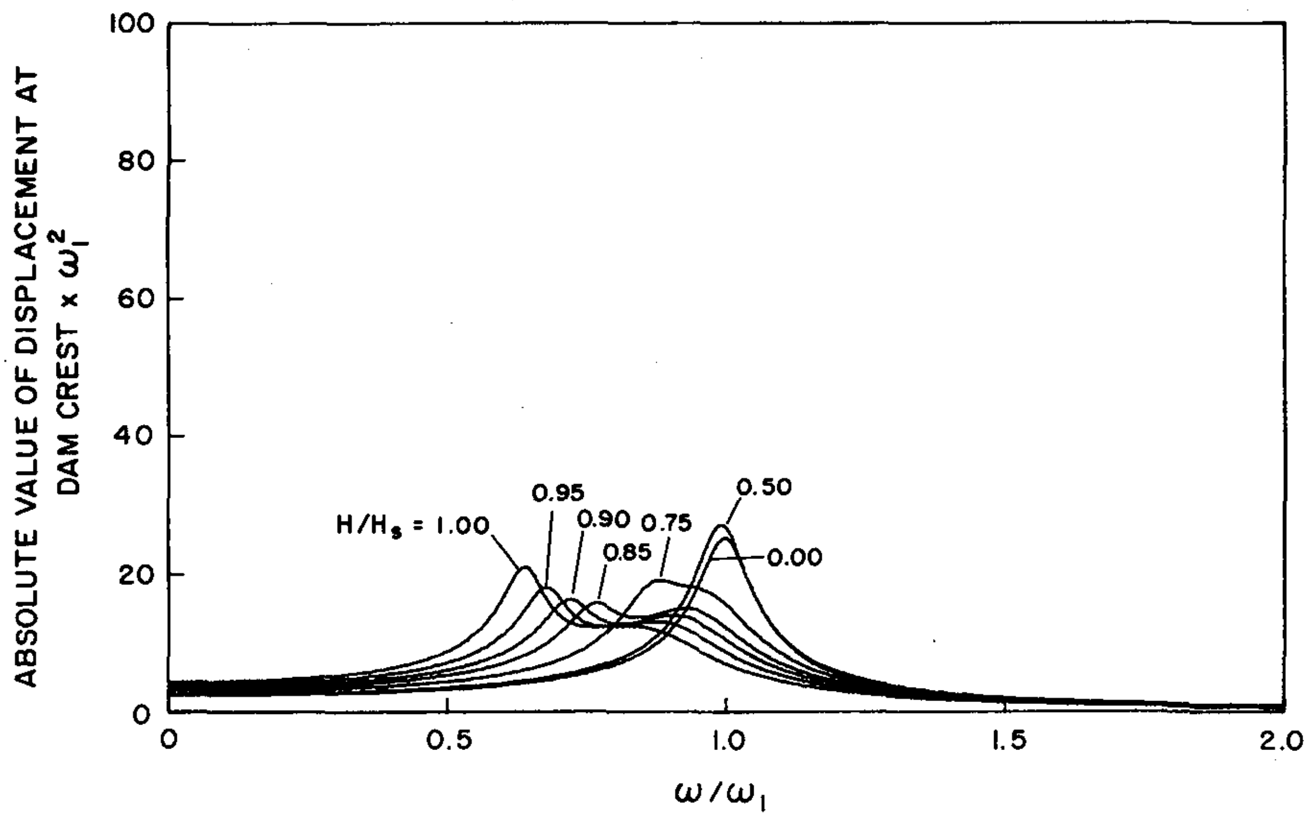


Figure A.6. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.80

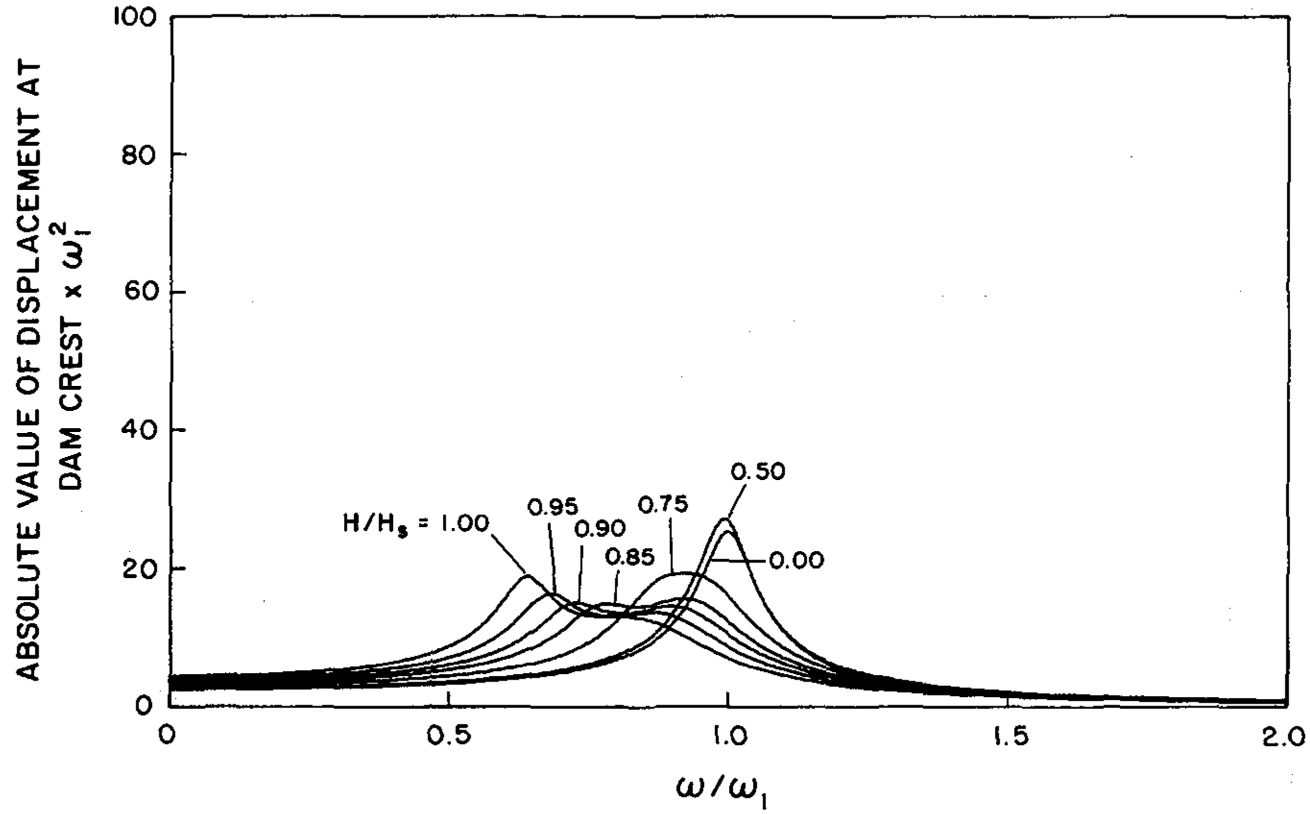


Figure A.7. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.75

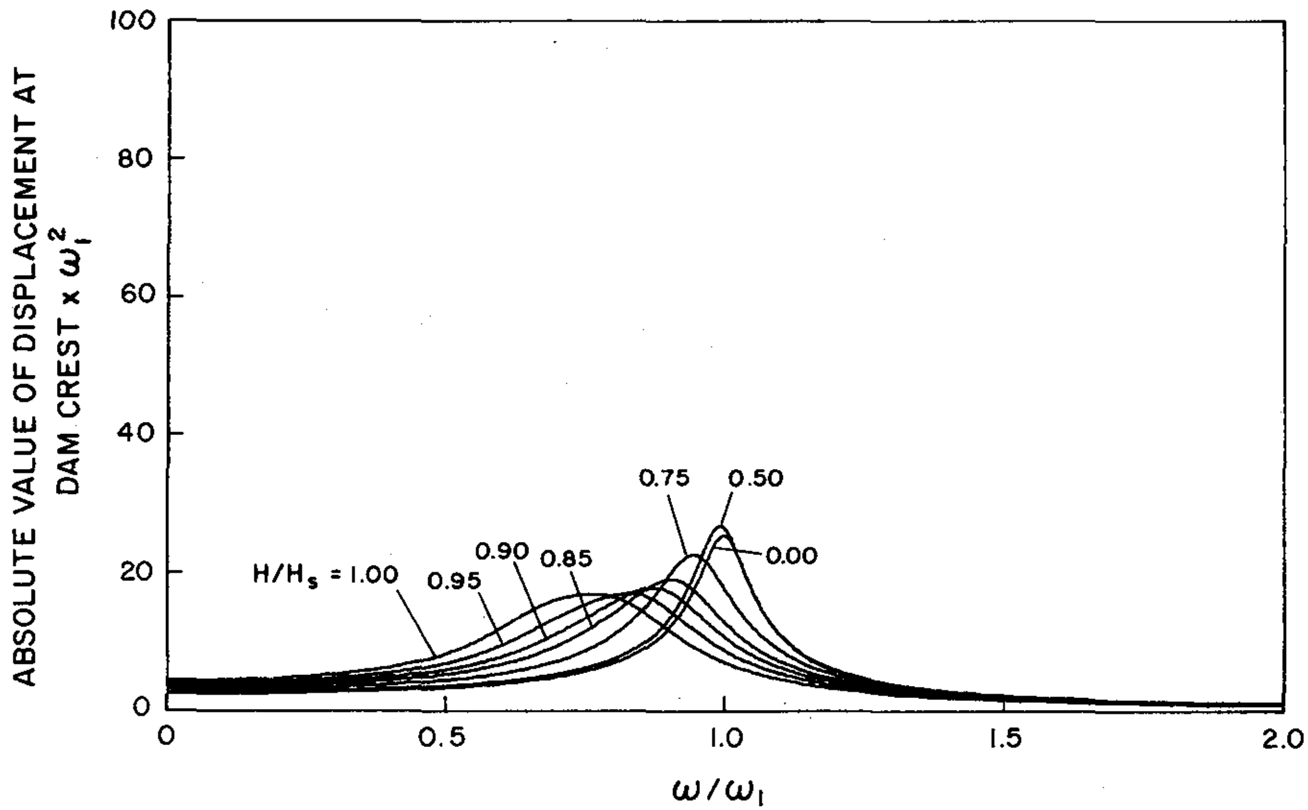


Figure A.8. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.50

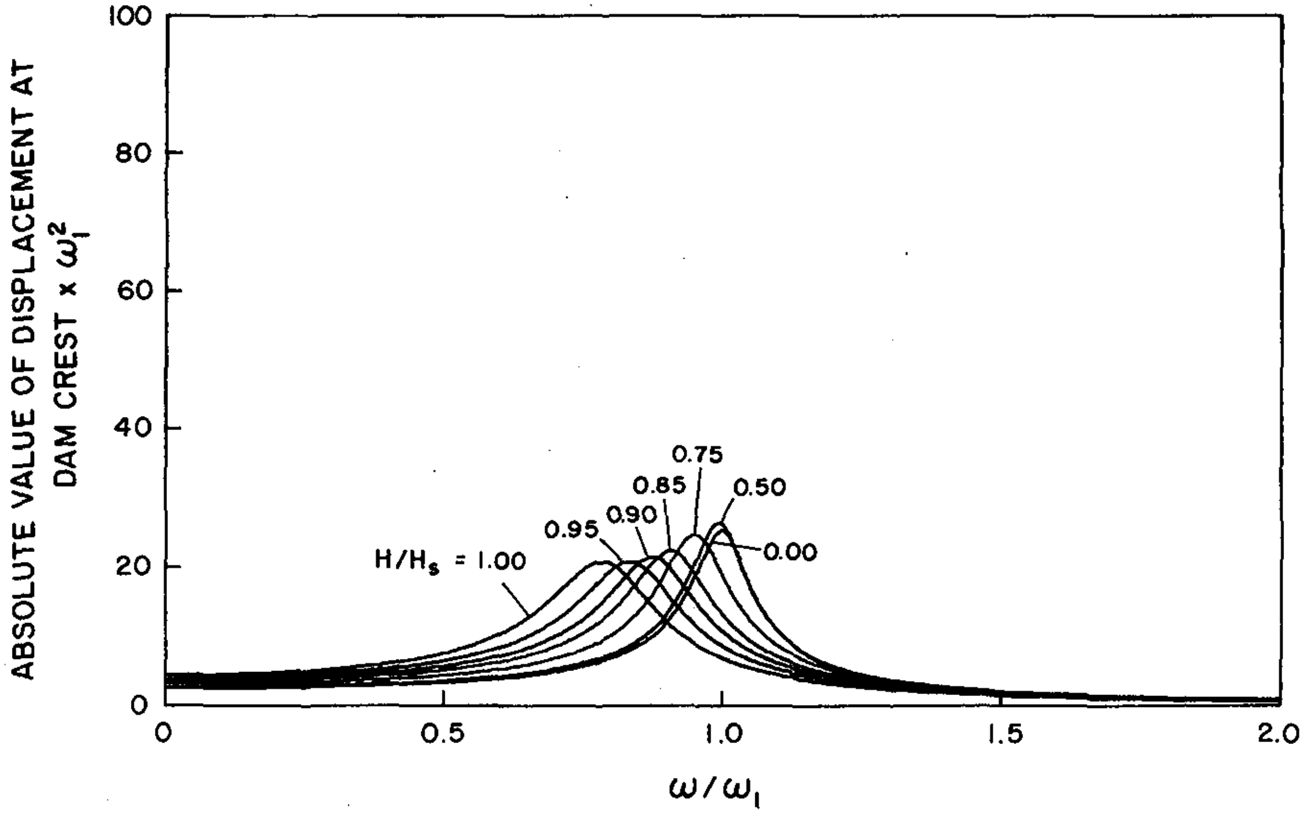


Figure A.9. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0.25

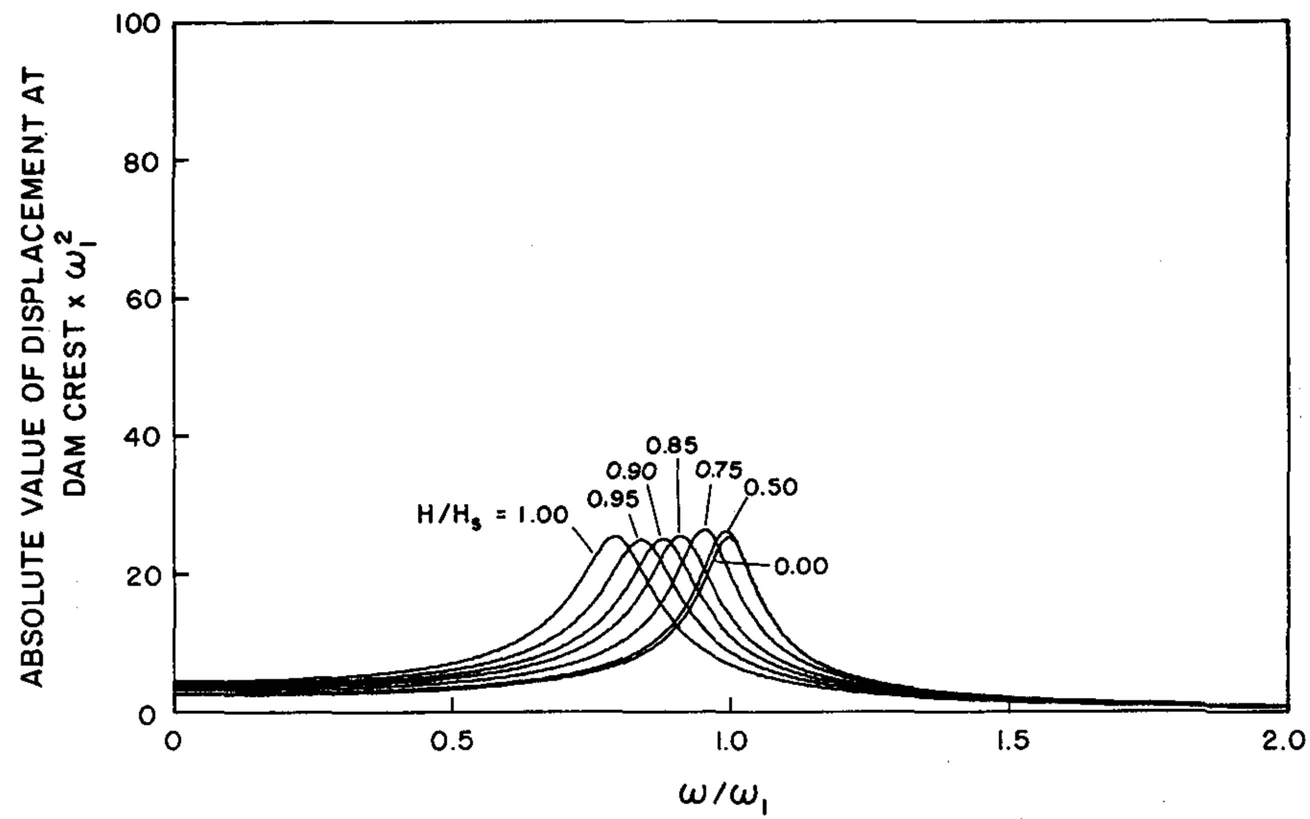


Figure A.10. Fundamental Mode Response of Dams on Rigid Foundation Rock due to Harmonic Horizontal Ground Motion; E_s = 6 million psi, α = 0

APPENDIX B: RESPONSE CONTRIBUTIONS OF HIGHER VIBRATION MODES

Dams with Empty Reservoir

The maximum earthquake effects associated with the contribution of the nth mode of dam vibration to the response of the dam can be represented by equivalent lateral forces (2)

$$f_n\,(y) = m_s(y)\,\phi_n(y)\,\omega_n^2\,\bar{Y}_n \tag{B.1}$$

in which $\bar{Y}_n$ is the maximum value of $Y_n(t)$ which is governed by the nth modal equation

$$\ddot{Y}_n + 2\xi_n\,\omega_n\,\dot{Y}_n + \omega_n^2\,Y_n = -\frac{L_n}{M_n}\,a_g(t) \tag{B.2}$$

In Eqs. B.1 and B.2, the mass per unit height of the dam $m_s(y) = w_s(y)/g$; ω_n and $\phi_n(y)$ are the natural frequency and the horizontal component of the shape of the nth mode of vibration; ξ_n is the damping ratio for this mode; $a_g(t)$ is the ground acceleration; the generalized mass M_n and the generalized earthquake force coefficient L_n are:

$$M_n = \int_0^{H_s} m_s(y)\,\phi_n^2(y)\,dy \tag{B.3a}$$

$$L_n = \int_0^{H_s} m_s(y)\,\phi_n(y)\,dy \tag{B.3b}$$

Just as in the case of multistory buildings (14), soil-structure interaction effects may be neglected in a simplified procedure to compute the contributions of the higher vibration modes in the earthquake response of dams. Therefore these interaction effects have not been included in Eqs. B.1-B.3.

Because the periods of the higher vibration modes of concrete gravity dams are very short, an approximation to $Y_n(t)$ is given by a "static" solution of Eq. B.2, i.e., by dropping the inertial and damping terms:

$$\omega_n^2 Y_n(t) = -\frac{L_n}{M_n}\,a_g(t), \qquad n = 2, 3, \cdots \tag{B.4}$$

Thus, $\bar{Y}_n$, the maximum value of $Y_n(t)$, is given by

$$\omega_n^2 \overline{Y}_n = \frac{-L_n}{M_n} \, \bar{a}_g \tag{B.5}$$

where $\bar{a}_g$ is the maximum ground acceleration. Substitution of Eq. B.5 in Eq. B.1 gives

$$f_n\,(y) = \frac{-L_n}{M_n}\, m_s\,(y)\,\phi_n\,(y)\,\bar{a}_g \tag{B.6}$$

This is the "static correction" method for representing the contributions of higher vibration modes (1, 12).

Alternatively we could start with the expression for the maximum equivalent lateral forces in terms of the earthquake response spectrum (2):

$$f_n\,(y) = \frac{L_n}{M_n}\, S_{an}\; m_s\,(y)\,\phi_n\,(y) \tag{B.7}$$

in which $S_{an} = S_a(T_n,\xi_n)$ is the ordinate of the pseudo-acceleration response spectrum for the ground motion evaluated at the nth mode vibration period $T_n = 2\pi/w_n$ and damping ratio ξ_n. Because the periods of the higher vibration modes of concrete gravity dams are very short, the corresponding ordinates of the pseudo-acceleration response spectrum will be essentially equal to the maximum ground acceleration, i.e. $S_{an} = \bar{a}_g$. With little dynamic amplification, the higher vibration modes respond essentially in a "static" manner leading to:

$$f_n\,(y) = \frac{L_n}{M_n}\, m_s\,(y)\,\phi_n\,(y)\,\bar{a}_g \tag{B.8}$$

which is the same as Eq. B.6, but for the negative sign which had been dropped in Eq. B.7 (2).

Thus the maximum response in each higher vibration mode is attained at the same instant of time, when the ground acceleration attains its maximum value $\bar{a}_g$. Based on this implication of the "static correction" concept, the maximum earthquake effects associated with all vibration modes higher than the fundamental are given by the equivalent lateral forces $f_{sc}(y) = \sum_{n=2}^{\infty} f_n(y)$ which upon substitution of Eq. B.6 gives

$$f_{sc}\,(y) = -\sum_{n=2}^{\infty} \frac{L_n}{M_n}\; m_s\,(y)\,\phi_n\,(y)\,\bar{a}_g \tag{B.9}$$

The deformation response of a dam to ground acceleration $a_g(t)$ will be identical to the response of the structure on fixed base subjected to external forces equal to mass per unit height times the ground acceleration, acting opposite to the sense of ground acceleration. As shown in Ref. 2, the ground motion can therefore be replaced by effective forces = $-\, m_s(y)\, a_g(t)$. Corresponding to the maximum ground acceleration these forces are $-\, m_s(y)\, \bar{a}_g$ which can be expressed as the summation of modal contributions:

$$-\, m_s(y)\, \bar{a}_g \;=\; -\sum_{n=1}^{\infty} \frac{L_n}{M_n}\, m_s(y)\, \phi_n(y)\, \bar{a}_g \tag{B.10}$$

which can be utilized to rewrite Eq. B.9 as

$$f_{sc}(y) \;=\; -\frac{1}{g}\, w_s(y) \left[1 - \frac{L_1}{M_1}\, \phi_1(y)\right] \bar{a}_g \tag{B.11}$$

Because the direction of the lateral earthquake forces is reversible, the sign of $f_{sc}(y)$ is not relevant. Dropping the negative sign in Eq. B.11 and replacing $\phi_1(y)$ by $\phi(y)$ and $\bar{a}_g$ by a_g, for convenience of notation, leads to Eq. 13.

Dams with Impounded Water

The effects of dam-water interaction can be interpreted as introducing an added force and modifying the properties of the dam by an added mass and an added damping, which result in a modification of Eq. B.2 (10). Because the inertial and damping terms are dropped from Eq. B.2 in the "static correction" method of obtaining higher mode response, only the added hydrodynamic force (or pressure) $-\, p_o(y) a_g(t)$ need be considered in representing hydrodynamic effects in Eq. B.2. Thus Eq. B.5 becomes

$$\omega_n^2 \bar{Y}_n \;=\; -\left[\frac{L_n}{M_n} + \frac{1}{M_n} \int_0^H p_o(y)\, \phi_n(y)\, dy\right] \bar{a}_g \tag{B.12}$$

in which $p_o(y)$ is a real-valued, frequency-independent function describing the hydrodynamic pressure on a rigid dam, undergoing unit acceleration, with water compressibility neglected.

Substitution of Eq. B.12 into Eq. B.1 and summation over all vibration modes higher than the fundamental gives

$$f_{sc}(y) = -\sum_{n=2}^{\infty} \frac{L_n}{M_n} m_s(y)\, \phi_n(y)\, \bar{a}_g - \sum_{n=2}^{\infty} \frac{B_n}{M_n} m_s(y)\, \phi_n(y)\, \bar{a}_g \qquad \text{(B.13)}$$

in which

$$B_n = \int_0^H p_o(y)\, \phi_n(y)\, dy \qquad \text{(B.14)}$$

Representing the maximum value of the added hydrodynamic pressure as a summation of modal contributions leads to

$$-p_o(y)\, \bar{a}_g = -\sum_{n=1}^{\infty} \frac{B_n}{M_n} m_s(y)\, \phi_n(y)\, \bar{a}_g \qquad \text{(B.15)}$$

Equations B.10 and B.15 can be utilized to rewrite Eq. B.13 as

$$f_{sc}(y) = -\frac{1}{g}\left\{ w_s(y)\left[1 - \frac{L_1}{M_1}\phi_1(y)\right] + \left[g p_o(y) - \frac{B_1}{M_1} w_s(y)\, \phi_1(y)\right]\right\} \bar{a}_g \qquad \text{(B.16)}$$

As before, dropping the negative sign in Eq. B.16 and replacing $\phi_1(y)$ by $\phi(y)$ and $\bar{a}_g$ by a_g leads to Eq. 14.

For reasons mentioned earlier in this Appendix, dam-foundation rock interaction effects are neglected in the simplified analysis of higher mode response; thus Eq. B.16 is still applicable.

APPENDIX C: DETAILED CALCULATIONS FOR PINE FLAT DAM

This appendix presents the detailed calculations required in the simplified analysis procedure as applied to the tallest, nonoverflow monolith of Pine Flat Dam. The simplified procedure is directly applicable because the upstream face of the dam is nearly vertical and the tail water effects are neglected. All computations are performed for a unit width of the dam monolith. Only the details for Case 4 in Table 7 (full reservoir and flexible foundation rock) are presented.

Simplified Model of Monolith

The tallest, nonoverflow monolith of Pine Flat Dam is divided into ten blocks of equal height. Using a unit weight of 155 pcf for the concrete, the properties of the blocks are presented in Table C.1, from which the total weight is 9486 kips. Replacing the integrals in Eq. 7 by summations over the blocks gives:

$$M_1 \approx \frac{1}{g} \sum_{i=1}^{10} w_i \, \phi^2(y_i) = \frac{1}{g} (500 \text{ kip}) \tag{C.1}$$

$$L_1 \approx \frac{1}{g} \sum_{i=1}^{10} w_i \, \phi(y_i) = \frac{1}{g} (1390 \text{ kip}) \tag{C.2}$$

where w_i and y_i are the weight of block i and the elevation of its centroid, respectively. Additional properties of the simplified model are listed in Table C.2.

Equivalent Lateral Forces -- Fundamental Mode

The equivalent lateral earthquake forces $f_1(y)$ are given by Eq. 8, evaluated at each level using $S_a(\tilde{T}_1, \tilde{\xi}_1)/g = 0.327$ (from Table 7) and $\tilde{L}_1/\tilde{M}_1 = 3.41$ (from step 8 in the simplified procedure). The calculations are summarized in Table C.3.

Stress Computation -- Fundamental Mode

The equivalent lateral earthquake forces $f_1(y)$ consist of forces associated with the mass of the dam (the first term of Eq. 8) and the hydrodynamic pressure at the upstream face (the second term). For the purpose of computing bending stresses in the monolith, the forces associated with the mass are applied at the centroids of the blocks. The forces due to the hydrodynamic pressure are applied as a linearly distributed load to the upstream face of each block. Due to these two sets of lateral forces (Table C.3), the resultant bending moments in the monolith are computed at each level from the equations of equilibrium. The normal bending stresses are obtained from elementary beam theory. A computer program (described in Appendix D) was developed for computation of the normal bending stresses in a dam monolith due to equivalent lateral earthquake forces. Using this procedure, the normal bending stresses σ_{y1} at the two faces of Pine Flat Dam associated with the fundamental vibration mode response of the dam to the S69E component of Taft ground motion were computed (Table C.4).

The maximum principal stresses at the upstream and downstream faces due to the fundamental vibration mode response can be computed from the normal bending stresses σ_{y1} by an appropriate transformation (see Ref. 13, Figs. 4-2 and 4-3, pp. 41-42):

$$\sigma_1 = \sigma_{y1} \sec^2\theta + p_1 \tan^2\theta \qquad \text{(C.3)}$$

where θ is the angle of the face with respect to the vertical. Because 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. 8:

$$p_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} S_a(\tilde{T}_1, \tilde{\xi}_1)\, p(y, \tilde{T}_r) \qquad \text{(C.4)}$$

which was computed in Steps 1-9 of the simplified procedure. For the upstream face of Pine Flat Dam, $\theta = 0°$ near the top and $\theta = 2.86°$ at lower elevations. For such small values of θ, the second term in Eq. C.3 turns out to be negligible.

The maximum principal stresses σ_1 due to the fundamental vibration mode response are given in Table C.5. Note that Eq. C.3 is evaluated at each level with θ for the block above that level.

Equivalent Lateral Forces -- Higher Vibration Modes

The equivalent lateral earthquake forces $f_{sc}(y)$ due to the higher vibration modes are given by Eq. 14, evaluated at each level using the maximum ground acceleration for the S69E component of the Taft Ground motion $a_g = 0.18$ g, and $L_1/M_1 = 2.78$ and $B_1/M_1 = 0.425$. The results are summarized in Table C.6. The calculation of bending moments due to the higher vibration modes is similar to the moment calculations for the fundamental vibration mode, as described previously.

Stress Computation -- Higher Vibration Modes

The normal bending stresses at the faces of the monolith due to the equivalent lateral earthquake forces $f_{sc}(y)$ are computed by the procedure described above for stresses due to forces $f_1(y)$. The resulting normal bending stresses $\sigma_{y,sc}$ presented in Table C.7 are due to the response contributions of the higher vibration modes.

The maximum principal stresses at the upstream and downstream faces due to the higher vibration modes can be computed from the normal bending stresses $\sigma_{y,sc}$ by a transformation similar to Eq. C.3:

$$\sigma_{sc} = \sigma_{y,sc} \sec^2\theta + p_{sc} \tan^2\theta \qquad \text{(C.5)}$$

Because no tail water is included 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. 14:

$$p_{sc}(y) = \left[g p_o(y) - \frac{B_1}{M_1} w_s(y) \ \phi(y) \right] \frac{\bar{a}_g}{g} \qquad \text{(C.6)}$$

which was computed in step 11 of the simplified procedure. However, the contribution of the second term in Eq. C.5 is negligible because, as mentioned earlier, θ is very small. The maximum principal stresses σ_{sc} due to the response contributions of the higher vibration modes are given in Table C.8.

Initial Static Stresses

The maximum principal stresses at the upstream and downstream faces due to the initial forces on the dam -- self weight, hydrostatic pressure, thermal effects, etc. -- can be computed from the normal stresses $\sigma_{y,st}$, which include the effects of direct forces, and a transformation similar to Eqs. 35 and 37:

$$\sigma_{st} = \sigma_{y,st} \sec^2\theta + p_{st} \tan^2\theta \tag{C.7}$$

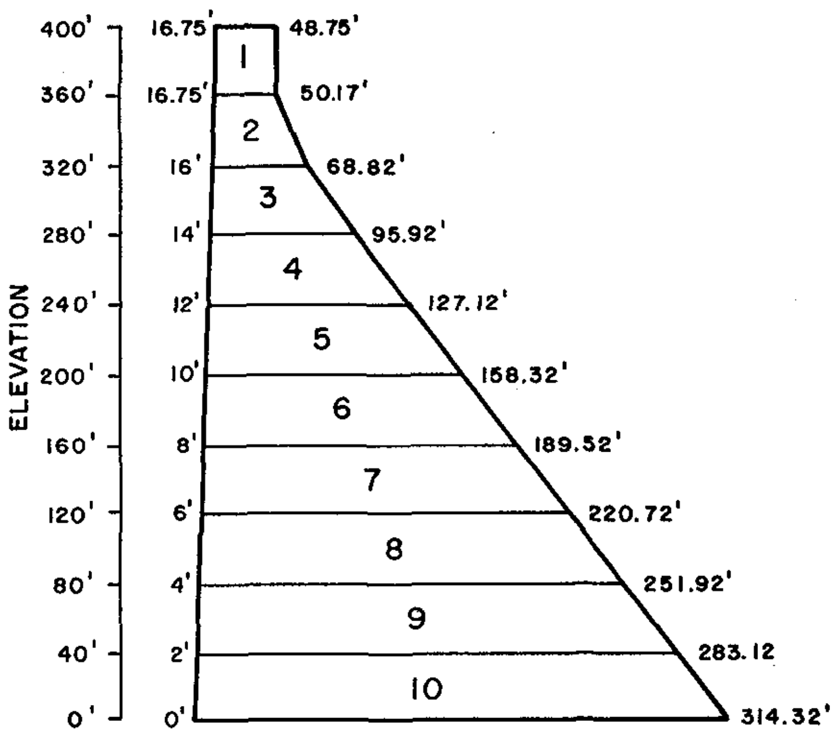
Because no tail water is included in the example analysis, the hydrostatic pressure $p_{st} = 0$ for the downstream face. At the upstream face, the hydrostatic pressure $p_{st}(y) = w(H-y)$. However, the contribution of the second term in Eq. C.7 is negligible because, as mentioned earlier, θ is very small for the upstream face. The maximum principal stresses σ_{st} due to the self weight of the dam and hydrostatic pressure are given in Table C.9.

Response Combination: Maximum Principal Stresses

The SRSS or ABSUM combination rules (Eqs. 16-17) are applicable to the computation of any response quantity that is proportional to the generalized modal coordinate responses. Thus these combination rules are generally inappropriate to determine the principal stresses. However, as shown in the preceding sections, the second terms in Eqs. C.3, C.5 and C.7 are negligible in the analysis of Pine Flat Dam. Such would be the case for most gravity dams because the upstream face is usually almost vertical and the effects of tail water at the downstream face are small. Thus, the principal stresses in Eqs. C.3 and C.5 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 obviously apply to the principal stresses.

The maximum principal stresses σ_1 due to the fundamental vibration mode (Table C.5) and σ_{sc} due to the higher vibration modes (Table C.8) are combined by the SRSS rule (Eq. 16) to obtain an estimate of the dynamic stress σ_d. The results at various levels in the dam are shown in Table C.9 and in Case 4 of Fig. 15.

The stresses σ_{st} at the faces of the monolith due to the initial static loads (weight of the dam and hydrostatic pressure), computed using elementary beam theory, are also shown in Table C.9. The total stresses, obtained by combining the static and dynamic stresses according to Eq. 18, are presented in Table C.9 and in Case 4 of Fig. 17.



COORDINATES OF UPSTREAM AND DOWNSTREAM FACES

Table C.1 - Properties of the Simplified Model

Block	Weight, w (k)	Elevation of centroid (ft)	ϕ at centroid	$w\,\phi$ (k)	$w\,\phi^2$ (k)
1	202.8	379.9	0.865	175.4	151.7
2	267.4	338.5	0.612	163.6	100.1
3	417.7	298.6	0.450	188.0	84.6
4	610.8	258.9	0.331	202.2	70.0
5	816.7	219.2	0.238	194.4	46.3
6	1022	179.3	0.164	167.6	27.5
7	1228	139.4	0.107	131.4	14.1
8	1434	99.5	0.065	93.2	6.1
9	1640	59.6	0.034	55.8	1.9
10	1846	19.6	0.010	18.5	0.2
Total	9486			1390	500

Table C.2 - Additional Properties of the Simplified Model

Level	Elevation y (ft)	Width $b^{(1)}$ (ft)	Weight per Unit Height w_s = 0.155b (k/ft)	Section Modulus S = 1/6 b^2 (ft^3)
Top	400	32.0	4.96	170.1
1	360	33.4	5.18	185.9
2	320	52.8	8.19	464.6
3	280	81.9	12.7	1,118
4	240	115.1	17.8	2,209
5	200	148.3	23.0	3,666
6	160	181.5	28.1	5,492
7	120	214.7	33.3	7,684
8	80	247.9	38.4	10,240
9	40	281.1	43.6	13,170
10	0	314.3	48.7	16,470

(1)From Figure in Table C.1.

Table C.3 - Equivalent Lateral Earthquake Forces -- Fundamental Vibration Mode

Level	y (ft)	w_s(1) (k/ft)	$\frac{y}{H_s}$	ϕ(2)	$w_s \phi$ (k/ft)	$\frac{y}{H}$	$\frac{gp}{wH}$(3)	gp (k/ft)	$f_1(y)$(4) (k/ft)
Top	400	4.96	1.00	1.00	4.96	1.05	0	0	5.52
1	360	5.18	0.90	0.73	3.80	0.94	0.079	1.68	6.11
2	320	8.19	0.80	0.53	4.35	0.84	0.137	2.96	8.15
3	280	12.7	0.70	0.39	4.95	0.73	0.159	3.39	9.30
4	240	17.8	0.60	0.28	4.98	0.63	0.163	3.50	9.46
5	200	23.0	0.50	0.20	4.60	0.52	0.159	3.43	8.95
6	160	28.1	0.40	0.13	3.65	0.42	0.150	3.24	7.69
7	120	33.3	0.30	0.084	2.80	0.31	0.144	3.09	6.57
8	80	38.4	0.20	0.047	1.80	0.21	0.132	2.85	5.19
9	40	43.6	0.10	0.021	0.92	0.10	0.125	2.68	4.01
10	0	48.7	0	0	0	0	0.117	2.51	2.80

(1) From Table C.2

(2) From Fig. 3 or Table 1

(3) From step 6, by linearly interpolatng the data of Fig. 8 or Table 4.

(4) From Eq. 8.

Table C.4 - Normal Bending Stresses -- Fundamental Mode

Level	Section[1] Modulus (ft^3)	Bending Moment (k-ft)	Bending Stress at Faces (psi)
Top	170.1	0	0
1	185.9	3,998	149
2	464.6	17,800	266
3	1,118	44,420	276
4	2,209	85,730	270
5	3,666	142,000	269
6	5,492	212,700	269
7	7,684	295,700	267
8	10,240	389,000	264
9	13,170	491,000	259
10	16,470	600,000	253

(1) From Table C.2

Table C.5 - Maximum Principal Stresses -- Fundamental Mode

Level	Bending Stress at Faces[1] (psi)	Upstream Face $\theta^{(2)}$ (°)	Upstream Face Max. Principal Stress, σ_1, (psi)	Downstream Face $\theta^{(2)}$ (°)	Downstream Face Max. Principal Stress, σ_1, (psi)
Top	0	0	0	0	0
1	149	0	149	2.06	149
2	266	1.09	266	25.2	324
3	276	2.86	276	34.4	403
4	270	2.86	270	38.0	434
5	269	2.86	269	38.0	433
6	269	2.86	269	38.0	433
7	267	2.86	267	38.0	429
8	264	2.86	264	38.0	425
9	259	2.86	259	38.0	417
10	253	2.86	253	38.0	407

(1) From Table C.4 with sign neglected

(2) θ is for the block above each level

Table C.6 - Equivalent Lateral Earthquake Forces -- Higher Vibration Modes

Level	y (ft)	$w_s[1-\frac{L_1}{M_1}\phi]^{(1)}$ (k/ft)	$\frac{y}{H}$	$\frac{gp_0}{wH}^{(2)}$	gp_0 (k/ft)	$[gp_0-\frac{B_1}{M_1}w_s\phi]^{(3)}$ (k/ft)	$f_{sc}(y)^{(4)}$ (k/ft)
Top	400	-8.83	1.05	0	0	-2.11	-1.98
1	360	-5.33	0.94	0.154	3.67	2.06	-0.59
2	320	-3.88	0.84	0.313	7.45	5.60	0.31
3	280	-1.07	0.73	0.437	10.3	8.20	1.29
4	240	3.94	0.63	0.523	12.5	10.4	2.58
5	200	10.2	0.52	0.598	14.1	12.1	4.03
6	160	17.9	0.42	0.650	15.5	14.0	5.74
7	120	25.5	0.31	0.693	16.4	15.2	7.33
8	80	33.4	0.21	0.720	17.1	16.3	8.96
9	40	41.1	0.10	0.737	17.5	17.1	10.5
10	0	48.7	0	0.742	17.6	17.6	11.9

(1) w_s and ϕ from Table C.3

(2) From linear interpolation of data from Fig. 11 or Table 6

(3) $w_s\phi$ from Table C.3

(4) From Eq. 14

Table C.7 - Normal Bending Stresses -- Higher Vibration Modes

Level	Section[1] Modulus $(ft)^3$	Bending Moment (k-ft)	Normal Bending Stress at Faces (psi)
Top	170.1	0	0
1	185.9	-1,239	-46
2	464.6	-3,736	-56
3	1,118	-5,764	-36
4	2,209	-5,713	-18
5	3,666	-1,584	-3
6	5,492	9,024	11
7	7,684	28,680	26
8	10,240	60,000	41
9	13,170	105,600	56
10	16,470	168,000	71

[1] From Table C.2

Table C.8 - Maximum Principal Stresses -- Higher Vibration Modes

Level	Bending Stress at Faces[1] (psi)	Upstream Face $\theta^{(2)}$ (°)	Upstream Face Max. Principal Stress, σ_{sc}, (psi)	Downstream Face $\theta^{(2)}$ (°)	Downstream Face Max. Principal Stress, σ_{sc}, (psi)
Top	0	0	0	0	0
1	46	0	46	2.06	46
2	56	1.09	56	25.2	68
3	36	2.86	36	34.4	53
4	18	2.86	18	38.0	29
5	3	2.86	3	38.0	5
6	11	2.86	11	38.0	18
7	26	2.86	26	38.0	42
8	41	2.86	41	38.0	66
9	56	2.86	56	38.0	90
10	71	2.86	71	38.0	114

(1) From Table C.7, with sign neglected

(2) θ is for the block above each level

Table C.9 - Response Combination of Maximum Principal Stresses (psi)

Level	Upstream Face					Downstream Face				
	$\sigma_1^{(1)}$	$\sigma_{sc}^{(2)}$	σ_d	σ_{st}	σ_{max}	$\sigma_1^{(1)}$	$\sigma_{sc}^{(2)}$	σ_d	σ_{st}	σ_{max}
Top	0	0	0	0	0	0	0	0	0	0
1	149	46	156	-41	115	149	46	156	-43	113
2	266	56	272	-72	200	324	68	332	-63	269
3	276	36	278	-83	195	403	53	406	-101	305
4	270	18	270	-95	175	434	29	434	-144	290
5	269	3	269	-106	163	433	5	433	-185	248
6	269	11	269	-119	150	433	18	433	-228	205
7	267	26	268	-133	135	429	42	432	-272	160
8	264	41	267	-148	119	425	66	430	-316	114
9	259	56	265	-163	102	417	90	426	-360	66
10	253	71	263	-178	85	407	114	423	-404	19

(1) From Table C.5

(2) From Table C.8

APPENDIX D: COMPUTER PROGRAM FOR STRESS COMPUTATION

This appendix describes a computer program for computing the stresses in a dam monolith using the results of the step-by-step simplified analysis procedure presented in this report. The program computes the bending stresses due to the equivalent lateral forces, $f_1(y)$ and $f_{sc}(y)$, representing the maximum effects of the fundamental and higher vibration modes of the dam, respectively. The program also computes the direct and bending stresses due to the self-weight of the dam and hydrostatic pressure. Transformation to principal stresses and combination of stresses due to the three load cases are not performed.

The program is written in FORTRAN 77 for interactive execution.

Simplified Model of Dam Monolith

A dam monolith is modeled as a series of blocks, numbered sequentially from the base to the crest. Increasing the number of blocks increases the accuracy of the computed stresses. The free surface of the impounded water may be at any elevation. The elevation of the reservoir bottom must be equal to the elevation of a block bottom. Figure D.1 shows the features of the simplified block model.

Program Input

The program queries the user for all input data, which are entered free-format. The program assumes that the unit of length is feet, the unit of force and weight is kip, the unit of acceleration is g's, and the unit of stress is psi. The input data are as follows:

1. N, the number of blocks in the simplified model.
2. The default unit weight of concrete in the dam.
3. For the bottom of each block i, the x-coordinate u_i at the upstream face, the x-coordinate d_i at the downstream face, the elevation, and the unit weight of concrete in the block (enter zero if default unit weight).

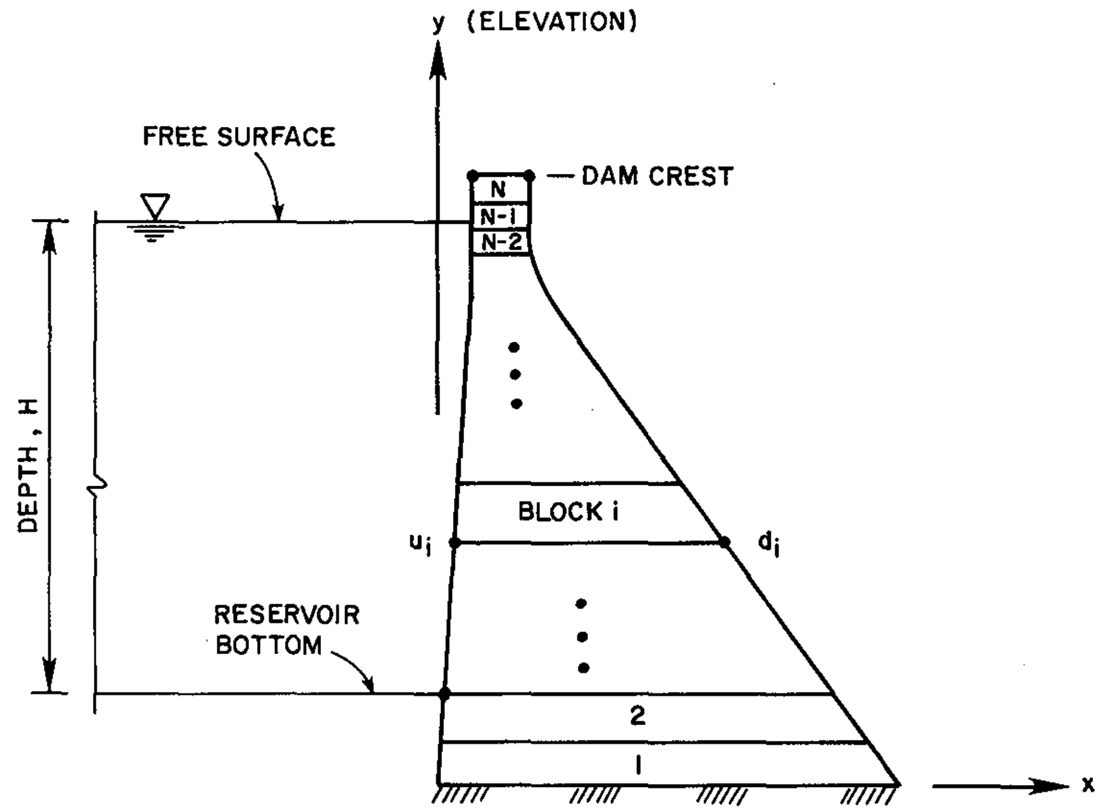


Figure D.1. Block Model of Dam Monolith

4. The x-coordinates of the upstream and downstream faces and the elevation of the dam crest.

5. An alternate value for the ratio L_1/M_1, if desired, where M_1 and L_1 are the generalized mass and earthquake force coefficient for the dam on rigid foundation rock with empty reservoir (Eq. 7). If not specified, the value of L_1/M_1 computed from the block model (as in steps 7 and 8 of the step-by-step procedure) is used.

The remaining data are entered for each case:

6. The elevations of the free surface of water and reservoir bottom.

7. The ordinates of the hydrodynamic pressure function, gp/wH, at the y/H values indicated. The ordinates are obtained from Step 6 of the step-by-step procedure.

8. The pseudo-acceleration ordinate of the earthquake design spectrum evaluated at the fundamental vibration period and damping ratio of the dam as evaluated in Step 9 of the step-by-step procedure.

9. The ratio $\tilde{L}_1/\tilde{M}_1$, where $\tilde{M}_1$ and $\tilde{L}_1$ = generalized mass and earthquake force coefficient including hydrodynamic effects determined in Steps 7 and 8 of the step-by-step procedure. This ratio reduces to L_1/M_1 for dam with empty reservoir.

10. An alternate value of B_1/M_1, if desired. If not specified the value computed in Step 11 of the step-by-step procedure is used.

11. The maximum ground acceleration of the design earthquake.

Computed Response

The program computes the vertical, normal stresses at the bottom of each block at the upstream and downstream faces based on simple beam theory. Stresses are computed for three loading cases: (1) static forces (self-weight of the dam and hydrostatic pressure); (2) equivalent lateral forces associated with the fundamental vibration mode; and (3) the equivalent lateral forces associated with the higher vibration modes. The unit of stress is pounds per square inch.

Example

The use of the computer program in the stress computation for Pine Flat Dam, detailed in Appendix C, is illustrated in the listing shown next wherein the computed vertical, normal stresses due to the three loading cases are also presented.

D>

ENTER THE NUMBER OF BLOCKS IN THE DAM: 10

ENTER THE DEFAULT UNIT WEIGHT: 0.155

```
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  1: 0,314.32,0,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  2: 2,283.12,40,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  3: 4,251.92,80,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  4: 6,220.72,120,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  5: 8,189.52,160,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  6: 10,158.32,200,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  7: 12,127.12,240,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  8: 14,95.92,280,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  9: 16,68.82,320,0
     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. 10: 16.75,50.172,360,0
```

ENTER X1, X2 AND Y AT THE CREST: 16.75,48.75,400

PROPERTIES OF THE DAM

BLOCK	CENTROID ELEV.	WEIGHT
10	379.855	202.808
9	338.500	267.350
8	298.560	417.694
7	258.877	610.824
6	219.160	816.664
5	179.329	1022.504
4	139.441	1228.344
3	99.522	1434.184
2	59.582	1640.024
1	19.628	1845.864
		9486.262

FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM

L1 = 1389.695   M1 = 499.738

THE FACTOR L1/M1 IS = 2.781

ENTER AN ALTERNATE VALUE FOR L1/M1:0

DO YOU WANT TO CONTINUE? (0=YES,1=NO):0

ENTER ELEVATION OF FREE - SURFACE: 381

ENTER ELEVATION OF RESERVOIR BOTTOM: 0

STATIC STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-41.217	-43.062
9	-72.259	-51.872
8	-83.050	-69.518
7	-94.724	-89.360
6	-106.442	-114.791
5	-119.295	-141.630
4	-133.116	-168.865
3	-147.660	-196.191
2	-162.739	-223.509
1	-178.218	-250.787

ENTER THE HYDRODYNAMIC PRESSURE FOR THE
FUNDAMENTAL VIBRATION MODE OF THE DAM

ENTER THE PRESSURE ORDINATE FOR Y/H = .945: 0.079

ENTER THE PRESSURE ORDINATE FOR Y/H = .840: 0.137

ENTER THE PRESSURE ORDINATE FOR Y/H = .735: 0.159

ENTER THE PRESSURE ORDINATE FOR Y/H = .630: 0.163

ENTER THE PRESSURE ORDINATE FOR Y/H = .525: 0.159

ENTER THE PRESSURE ORDINATE FOR Y/H = .420: 0.150

ENTER THE PRESSURE ORDINATE FOR Y/H = .315: 0.144

ENTER THE PRESSURE ORDINATE FOR Y/H = .210: 0.132

ENTER THE PRESSURE ORDINATE FOR Y/H = .105: 0.125

ENTER THE PRESSURE ORDINATE FOR Y/H = .000: 0.117

ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: 0.327

```
ENTER L1(TILDE)/M1(TILDE) FACTOR:3.4

      FUNDAMENTAL MODE STRESSES IN DAM

BLOCK       UPSTREAM FACE   DOWNSTREAM FACE
-------------------------------------------

 10           149.655          -149.655
  9           266.785          -266.785
  8           276.513          -276.513
  7           270.140          -270.140
  6           269.726          -269.726
  5           269.447          -269.447
  4           267.707          -267.707
  3           264.298          -264.298
  2           259.370          -259.370
  1           253.233          -253.233

THE FACTOR B1/M1 IS =        .428
ENTER AN ALTERNATE VALUE FOR B1/M1:0

ENTER MAX. GROUND ACCELERATION IN G: 0.18

          HIGHER MODE STRESSES IN DAM

BLOCK       UPSTREAM FACE   DOWNSTREAM FACE
-------------------------------------------

 10           -46.290           46.290
  9           -55.847           55.847
  8           -35.803           35.803
  7           -17.958           17.958
  6            -2.985            2.985
  5            11.409          -11.409
  4            25.917          -25.917
  3            40.693          -40.693
  2            55.697          -55.697
  1            70.840          -70.840

DO YOU WANT TO CONTINUE? (0=YES,1=NO):1
Stop - Program terminated.
```

```
C***********************************************************************
C
C     A COMPUTER PROGRAM TO PERFORM A SIMPLIED STRESS ANALYSIS
C            OF CONCRETE GRAVITY DAMS DUE TO EARTHQUAKES
C          INCLUDING THE EFFECTS OF DAM-WATER INTERACTION,
C  DAM-FOUNDATION ROCK INTERACTION, AND RESERVOIR BOTTOM ABSORPTION
C
C                        GREGORY FENVES
C             THE UNIVERSITY OF TEXAS AT AUSTIN
C
C                         VERSION 1.0
C                         JANUARY 1985
C
C***********************************************************************
C
      CALL SIMPL
      STOP
      END
      SUBROUTINE SIMPL
C
C         MAIN SUBPROGRAM -- CONTROL THE EXECUTION OF THE PROGRAM
C
      DIMENSION BLOCKS(5,21),PRESS(21),WEIGHT(20),STRSTA(2,20),
     1          STRWGT(2,20),STRDUM(2,20),STRFUN(2,20),
     2          STRDYN(2,20),STRCOR(2,20),STRWCR(2,20)
C
      DIMENSION PARTFC(3)
      CHARACTER*40 TITSTA,TITFUN,TITCOR
C
      DATA TITSTA/'           STATIC STRESSES IN DAM       '/,
     1     TITFUN/'     FUNDAMENTAL MODE STRESSES IN DAM   '/,
     2     TITCOR/'        HIGHER MODE STRESSES IN DAM     '/
C
C         THESE DATA STATEMENTS, AND THE FIRST DIMENSION STATEMENT,
C         DETERMINE THE MAXIMUM NUMBER OF BLOCKS THAT MAY BE USED
C
      DATA NMAX/20/
C
C         THIS DATA STATEMENT CONTAINS UNIT-DEPENDENT CONSTANTS
C
      DATA GAMMA/0.0624/,STRCON/0.144/
C
C         READ THE PROPERTIES OF THE BLOCKS, COMPUTE OTHER BLOCK
C         PROPERTIES, AND COMPUTE THE STATIC STRESSES DUE TO THE
C         WEIGHT AND EFFECTIVE EARTHQUAKE FORCE
C
      CALL DAMPRP (BLOCKS,NBLOCK,NMAX,WEIGHT,STRWGT,
     1             STRWCR,STRFUN,PARTFC)
C
C         TEST TO CONTINUE WITH EXECUTION OF PROGRAM
C
   10 WRITE (*,99)
      READ  (*,*) I
      IF (I.NE.0) GO TO 20
C
```

```
C         READ IN PROPERTIES OF THE IMPOUNDED WATER
C         AND COMPUTE STRESSES DUE TO HYDROSTATIC PRESSURE
C
      CALL REDWAT (H,HB)
      CALL DAMSTA (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
      STRFAC = 1.0/STRCON
      CALL COMSTA (NBLOCK,STRWGT,STRDUM,STRSTA,STRFAC,TITSTA)
C
C         COMPUTE THE DYNAMIC  STRESSES DUE TO THE FUNDAMENTAL
C         MODE SHAPE
C
      CALL DAMDYN (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
      WRITE (*,97)
      READ  (*,*) SA
      STRFAC = SA/STRCON
      WRITE (*,95)
      READ (*,*) SA
      STRFAC = STRFAC*SA
      CALL COMSTA (NBLOCK,STRFUN,STRDUM,STRDYN,STRFAC,TITFUN)
C
C         COMPUTE THE HIGHER MODE STRESSES
C
      CALL DAMCOR (BLOCKS,NBLOCK,GAMMA,H,HB,PARTFC(2),STRFUN,
     1             PRESS,STRDUM)
      WRITE (*,94)
      READ  (*,*) SA
      STRFAC = SA/STRCON
      CALL COMSTA (NBLOCK,STRWCR,STRDUM,STRCOR,STRFAC,TITCOR)
      GO TO 10
C
   20 RETURN
C
   99 FORMAT (/////' DO YOU WANT TO CONTINUE? (0=YES,1=NO):' )
   97 FORMAT (//' ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: ' )
   95 FORMAT (//' ENTER L1(TILDE)/M1(TILDE) FACTOR:' )
   94 FORMAT (//' ENTER MAX. GROUND ACCELERATION IN G: ' )
C
      END
      SUBROUTINE DAMPRP (BLOCKS,NBLOCK,NMAX,WEIGHT,STRWGT,
     1                   STRWCR,STRFUN,PARTFC)
C
C         INPUT THE PROPERTIES OF THE DAM, AND COMPUTE THE STATIC
C         AND FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM
C
      DIMENSION BLOCKS(5,1),WEIGHT(1),STRWGT(2,1),
     1          STRWCR(2,1),STRFUN(2,1),PARTFC(3)
      EXTERNAL VALWGT,VALHOR
C
C         INPUT BLOCK PROPERTIES AND COMPUTE STATIC STRESSES
C
      CALL REDBLK (BLOCKS,NBLOCK,NMAX,WEIGHT)
      CALL BLCKVL (BLOCKS,NBLOCK,WEIGHT,WEIGHT)
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALWGT,STRWGT)
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALHOR,STRWCR)
C
```

```
      WRITE (*,99)
      DO 10 J=1,NBLOCK
         I = NBLOCK + 1 - J
         WRITE (*,98) I, BLOCKS(5,I),WEIGHT(I)
   10    CONTINUE
C
C        COMPUTE THE FUNDAMENTAL VIBRATION PROPERTIES AND STRESSES
C        DUE TO THE EFFECTIVE EARTHQUAKE FORCE
C
      CALL FUNMOD (BLOCKS,NBLOCK,WEIGHT,WEIGHT,XTOT,PARTFC(1),
     1             PARTFC(2))
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALHOR,STRFUN)
C
      PARTFC(3) = PARTFC(1)/PARTFC(2)
      WRITE (*,97) XTOT,(PARTFC(I),I=1,3)
      READ  (*,*) DUM
      IF (DUM.GT.0.0) PARTFC(3) = DUM
C
C        COMPUTE THE HIGHER MODE STRESSES DUE TO THE
C        WEIGHT OF THE DAM
C
      DO 20 J=1,NBLOCK
         STRWCR(1,J) = STRWCR(1,J) - PARTFC(3)*STRFUN(1,J)
         STRWCR(2,J) = STRWCR(2,J) - PARTFC(3)*STRFUN(2,J)
   20    CONTINUE
C
      RETURN
C
   99 FORMAT (////3X,'PROPERTIES OF THE DAM'//' BLOCK',2X,'CENTROID',
     1        4X,'WEIGHT'/10X,'ELEV.'/1X,27('-')/)
   98 FORMAT (3X,I2,3X,F8.3,3X,F8.3)
   97 FORMAT (19X,'--------'/19X,F8.3//
     1        ' FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM'//
     2        5X,' L1 =',F9.3,5X,' M1 =',F9.3//
     3        5X,' THE FACTOR L1/M1 IS =',F9.3//
     4        ' ENTER AN ALTERNATE VALUE FOR L1/M1:' )
C
      END
      SUBROUTINE DAMSTA (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
C
C        COMPUTE THE STATIC STRESSES IN THE DAM DUE TO IMPOUNDED WATER
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRDUM(2,1)
      EXTERNAL VALHST
C
C        COMPUTE STATIC STRESSES DUE TO IMPOUNDED WATER
C
      CALL CALHST (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHST,STRDUM)
C
C        COMPUTE STATIC STRESSES DUE TO TAILWATER -- NOT
C        IMPLEMENTED IN THIS VERSION OF THE PROGRAM
C
      RETURN
      END
```

```
      SUBROUTINE DAMDYN (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
C
C         READ HYDRODYNAMIC PRESSURE AND COMPUTE STRESSES
C         DUE TO THE HYDRODYNAMIC PRESSURE
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRDUM(2,1)
      EXTERNAL VALHDY
C
      CALL REDHDY (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHDY,STRDUM)
C
      RETURN
      END
      SUBROUTINE DAMCOR (BLOCKS,NBLOCK,GAMMA,H,HB,XM1,STRFUN,
     1                   PRESS,STRDUM)
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRFUN(2,1),STRDUM(2,1)
      EXTERNAL VALHDY
      DATA BFACT/0.052/
C
      CALL CORPRS (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHDY,STRDUM)
C
      D = H - HB
      HS = BLOCKS(3,NBLOCK+1) - BLOCKS(3,1)
      B01M1 = 0.5*BFACT*GAMMA*D*D*D*D/(HS*HS*XM1)
      WRITE (*,99) B01M1
      READ  (*,*) D
      IF (D.GT.0.0) B01M1 = D
C
      DO 10 J = 1,NBLOCK
          STRDUM(1,J) = STRDUM(1,J) - B01M1*STRFUN(1,J)
          STRDUM(2,J) = STRDUM(2,J) - B01M1*STRFUN(2,J)
   10     CONTINUE
C
      RETURN
C
   99 FORMAT (//' THE FACTOR B1/M1 IS = ',F9.3/
     1         ' ENTER AN ALTERNATE VALUE FOR B1/M1:' )
      END
      SUBROUTINE REDBLK (BLOCKS,NBLOCK,NMAX,UNITWT)
C
C         READ THE PROPERTIES OF THE BLOCKS IN THE DAM
C
      DIMENSION BLOCKS(5,1),UNITWT(1)
C
      WRITE (*,99)
      READ  (*,*) NBLOCK
      IF (NBLOCK.GT.NMAX) GO TO 20
C
      WRITE (*,97)
      READ  (*,*) DEFWGT
C
      DO 10 I=1,NBLOCK
          WRITE (*,95) I
```

```
         READ  (*,*) (BLOCKS(J,I),J=1,3),UNT
         IF (UNT.LE.0.0) UNT = DEFWGT
         UNITWT(I) = UNT
   10    CONTINUE
C
      WRITE (*,93)
      READ  (*,*) (BLOCKS(J,NBLOCK+1),J=1,3)
C
      RETURN
C
C        TOO MANY BLOCKS REQUESTED FOR STORAGE ALLOCATED
C
   20 STOP
C
   99 FORMAT (//' ENTER THE NUMBER OF BLOCKS IN THE DAM: ' )
   97 FORMAT (//' ENTER THE DEFAULT UNIT WEIGHT: ' )
   95 FORMAT (5X,' ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. ',
     1          I2,': ' )
   93 FORMAT (//5X,' ENTER  X1, X2 AND  Y  AT THE CREST: ')
C
      END
      SUBROUTINE CORPRS (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
C
C        COMPUTE THE HYDRODYNAMIC PRESSURE ON THE UPSTREAM FACE OF A
C        RIGID DAM WITH INCOMPRESSIBLE WATER.  USED FOR THE
C        COMPUTATION OF HIGHER MODE STRESSES.
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
      DEPTH = H - HB
      IF (DEPTH.LE.0.0) RETURN
      NBL1 = NBLOCK + 1
      HS = BLOCKS(3,NBL1) - BLOCKS(3,1)
C
      DO 10 I = 1,NBL1
         PRESS(I) = 0.0
         Y = (BLOCKS(3,I) - HB)/DEPTH
         IF (Y.GT.1.0.OR.Y.LT.0.0) GO TO 10
         CALL POYFUN (Y,P0)
         PRESS(I) = GAMMA*DEPTH*P0
   10    CONTINUE
C
      RETURN
      END
      SUBROUTINE BLCKVL (BLOCKS,NBLOCK,UNITWT,WEIGHT)
C
C        COMPUTE THE LOCATIONS OF THE CENTROIDS AND
C        WEIGHTS OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),UNITWT(1),WEIGHT(1)
C
C        LOOP OVER THE BLOCKS, ONE AT A TIME, TOP TO BOTTOM
C
      DO 10 J=1,NBLOCK
         I = NBLOCK + 1 - J
```

```
C
          TOP = BLOCKS(2,I+1) - BLOCKS(1,I+1)
          BOT = BLOCKS(2,I  ) - BLOCKS(1,I  )
          DX  = BLOCKS(1,I+1) - BLOCKS(1,I  )
          DY  = BLOCKS(3,I+1) - BLOCKS(3,I  )
C
          CALL CENTRD (TOP,BOT,0.0,DY,AREA,DUM,DUM,RY)
          BLOCKS(4,I) = BLOCKS(1,I) +
     1                  (2.0*DX*TOP + DX*BOT + TOP*BOT
     2                   + TOP*TOP + BOT*BOT)/
     3                  (3.0*(TOP + BOT))
          BLOCKS(5,I) = BLOCKS(3,I) + RY
C
          WEIGHT(I)   = AREA*UNITWT(I)
   10     CONTINUE
C
      RETURN
      END
      SUBROUTINE FUNMOD (BLOCKS,NBLOCK,WEIGHT,WPHI,W1,W2,W3)
C
C         COMPUTE THE EFFECTIVE LATERAL LOAD FOR EACH BLOCK
C         AND THE TOTAL WEIGHT, EFFECTIVE EARTHQUAKE FORCE,
C         AND GENERALIZED WEIGHT OF THE DAM
C
      DIMENSION BLOCKS(5,1),WEIGHT(1),WPHI(1)
C
      HS = BLOCKS(3,NBLOCK+1) - BLOCKS(3,1)
      W1 = 0.0
      W2 = 0.0
      W3 = 0.0
C
C         LOOP OVER BLOCKS, ONE AT A TIME, BOTTOM TO TOP
C
      DO 10 I=1,NBLOCK
          Y = (BLOCKS(5,I) - BLOCKS(3,1))/HS
          CALL PHIONE (Y,PHI)
          W  = WEIGHT(I)
          WP = W*PHI
          WPHI(I) = WP
          W1 = W1 + W
          W2 = W2 + WP
          W3 = W3 + WP*PHI
   10     CONTINUE
C
      RETURN
      END
      SUBROUTINE PHIONE (Y,PHI)
C
C         OBTAIN THE ORDINATE OF THE FUNDAMENTAL VIBRATION MODE
C         OF THE DAM, USE THE STANDARD MODE SHAPE
C
      DIMENSION PHI1(22)
      DATA DY/0.05/,PHI1/0.000 , 0.010 , 0.021 , 0.034 , 0.047 ,
     1                   0.065 , 0.084 , 0.108 , 0.135 , 0.165 ,
     2                   0.200 , 0.240 , 0.284 , 0.334 , 0.389 ,
```

```
     3                   0.455 , 0.530 , 0.619 , 0.735 , 0.866 ,
     4                   1.000 , 1.000 /
C
      A   = Y/DY
      I   = IFIX(A) + 1
      A   = FLOAT(I) - A
      PHI = A*PHI1(I) + (1.0-A)*PHI1(I+1)
C
      RETURN
      END
      SUBROUTINE POYFUN (Y,PO)
C
C         OBTAIN THE HYDRODYNAIC PRESSURE ON A RIGID DAM WITH
C         INCOMPRESSIBLE WATER.
C
      DIMENSION POY(22)
      DATA DY/0.05/,POY/0.742 , 0.741 , 0.737 , 0.731 , 0.722 , 0.711 ,
     1                  0.696 , 0.680 , 0.659 , 0.637 , 0.610 , 0.580 ,
     2                  0.546 , 0.509 , 0.465 , 0.418 , 0.362 , 0.301 ,
     3                  0.224 , 0.137 , 0.000 , 0.000 /
C
      A = Y/DY
      I = IFIX(A) + 1
      A = FLOAT(I) - A
      PO = A*POY(I) + (1.0-A)*POY(I+1)
      RETURN
      END
      SUBROUTINE REDWAT (H,HB)
C
C         READ THE ELEVATIONS OF THE RESERVOIR
C
      WRITE (*,99)
      READ  (*,*) H
      WRITE (*,98)
      READ  (*,*) HB
C
      RETURN
C
   99 FORMAT (//' ENTER  ELEVATION  OF FREE - SURFACE: ' )
   98 FORMAT (/ ' ENTER ELEVATION OF RESERVOIR BOTTOM: ' )
      END
      SUBROUTINE CALHST (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
C
C         COMPUTE THE HYDROSTATIC PRESSURE ON THE FACE OF THE DAM
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
C         LOOP OVER THE BLOCK LEVELS, ONE AT A TIME, BOTTOM TO TOP
C
      NBL1 = NBLOCK + 1
      DO 10 I=1,NBL1
          PRESS(I) = 0.0
          Y = BLOCKS(3,I)
          IF (Y.LT.H.AND.Y.GE.HB) PRESS(I) = GAMMA*(H-Y)
   10     CONTINUE
```

```
C
      RETURN
      END
      SUBROUTINE REDHDY (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
C
C         READ AND COMPUTE THE HYDRODYNAMIC PRESSURE AT THE
C         BLOCK LEVELS ON THE UPSTREAM FACE OF THE DAM
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
      DEPTH = H - HB
      IF (DEPTH.EQ.0.0) RETURN
C
      NBL1   = NBLOCK + 1
      HHS    = DEPTH/(BLOCKS(3,NBL1) - BLOCKS(3,1))
      HHS2   = HHS*HHS
C
C         LOOP OVER BLOCK LEVELS, ONE AT A TIME, TOP TO BOTTOM
C
      WRITE (*,99)
      DO 10 J=1,NBL1
          I = NBLOCK + 2 - J
          PRESS(I) = 0.0
          Y = (BLOCKS(3,I) - HB)/DEPTH
          IF (Y.GT.1.0.OR.Y.LT.0.0) GO TO 10
C
C             READ PRESSURE COEFFICIENT AND COMPUTE HYDRODYNAMIC
C             PRESSURE AT THE BLOCK LEVEL
C
          WRITE (*,98) Y
          READ  (*,*) P
          PRESS(I) = GAMMA*DEPTH*HHS2*P
C
   10     CONTINUE
      RETURN
C
   99 FORMAT (//' ENTER THE HYDRODYNAMIC PRESSURE FOR THE '/
     1          ' FUNDAMENTAL VIBRATION MODE OF THE DAM ')
   98 FORMAT (/5X,' ENTER THE PRESSURE ORDINATE FOR Y/H =',
     1          F5.3,': ' )
      end
      SUBROUTINE STRLOD (BLOCKS,NBLOCK,LOADS,VALUES,STRESS)
C
C         COMPUTE THE NORMAL STRESSES DUE TO LOADS APPLIED AT THE
C         CENTROID OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),LOADS(1),STRESS(2,1)
      REAL LOADS,M
C
      HSUM  = 0.0
      HYSUM = 0.0
      VSUM  = 0.0
      VXSUM = 0.0
C
C         LOOP OVER BLOCKS, ONE AT A TIME, TOP TO BOTTOM
```

```
C
      DO 10 J=1,NBLOCK
          I = NBLOCK + 1 - J
C
C             OBTAIN THE LOADS AT THE CENTROID OF BLOCK I
C
          CALL VALUES (I,LOADS,V,H)
          HSUM  = HSUM  + H
          HYSUM = HYSUM + H*BLOCKS(5,I)
          VSUM  = VSUM  + V
          VXSUM = VXSUM + V*BLOCKS(4,I)
C
C             COMPUTE THE BENDING MOMENT AND STRESSES AT THE
C             BOTTOM OF BLOCK I
C
          M = HYSUM - VXSUM - BLOCKS(3,I)*HSUM
     1        + 0.5*(BLOCKS(2,I)+BLOCKS(1,I))*VSUM
C
          T = BLOCKS(2,I) - BLOCKS(1,I)
          M = 6.0*M/(T*T)
          STRESS(1,I) = VSUM/T + M
          STRESS(2,I) = VSUM/T - M
C
   10     CONTINUE
      RETURN
      END
      SUBROUTINE STRPRS (BLOCKS,NBLOCK,PRESS,IUPDN,H,HB,VALUES,STRESS)
C
C         COMPUTE THE NORMAL STRESSES DUE TO PRESSURE APPLIED
C         AT THE FACE, UPSTREAM (IUPDN=1) OR DOWNSTREAM (IUPDN=2),
C         OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRESS(2,1)
      REAL M
      LOGICAL YCOMP
C
      HSUM  = 0.0
      HYSUM = 0.0
      VSUM  = 0.0
      VXSUM = 0.0
C
      YB = BLOCKS(3,NBLOCK+1)
C
C         LOOP OVER BLOCKS, ONE AT A TIME, TOP TO BOTTOM
C
      DO 40 J=1,NBLOCK
C
          I = NBLOCK + 1 - J
          YBT = YB
          YB  = BLOCKS(3,I)
C
          IF (YB.GE.H.OR.YBT.LE.HB) GO TO 30
C
C             THE BLOCK TOUCHS WATER, OBTAIN THE WATER PRESSURE
C             AT THE TOP AND BOTTOM OF THE BLOCK
```

```
C
         CALL VALUES (I,PRESS,P1,P2,YCOMP)
         DX = 0.0
         IF (YCOMP) DX = BLOCKS(IUPDN,I+1) - BLOCKS(IUPDN,I)
         DY = BLOCKS(3,I+1) - BLOCKS(3,I)
         IF (YBT.LE.H) GO TO 10
C
C            TOP OF WATER IS IN BLOCK, MODIFY TOP PRESSURE POINT
C
         DUM = H - YB
         DX  = DX*DUM/DY
         DY  = DUM
         P1  = 0.0
         GO TO 20
C
C            CHECK THAT BOTTOM OF WATER CORRESPONDS TO A BLOCK
C
   10    IF (YB.LT.HB) WRITE (*,99)
C
C            COMPUTE PRESSURE AND FORCES ACTING ON BLOCK I
C
   20    CALL CENTRD (P1,P2,DX,DY,H1,V,RX,RY)
C
C            COMPUTE THE STRESS RESULTANTS AT THE BOTTOM OF BLOCK
C
         HSUM  = HSUM  + H1
         HYSUM = HYSUM + H1*(YB+RY)
         VSUM  = VSUM  + V
         VXSUM = VXSUM + V*(BLOCKS(IUPDN,I)+RX)
C
C            COMPUTE THE BENDING MOMENTS AND STRESSES AT THE
C            BOTTOM OF BLOCK I
C
   30    M = HYSUM - VXSUM - BLOCKS(3,I)*HSUM
     1       + 0.5*(BLOCKS(2,I)+BLOCKS(1,I))*VSUM
C
         T = BLOCKS(2,I) - BLOCKS(1,I)
         M = 6.0*M/(T*T)
         STRESS(1,I) = VSUM/T + M
         STRESS(2,I) = VSUM/T - M
C
   40    CONTINUE
      RETURN
C
   99 FORMAT (//' ERROR IN MODEL - RESERVOIR BOTTOM DOES NOT'/
     1          ' COINCIDE WITH THE BOTTOM OF A BLOCK'/)
C
      END
      SUBROUTINE VALWGT (I,LOADS,V,H)
C
C        OBTAIN THE WEIGHT OF BLOCK I
C
      DIMENSION LOADS(1)
      REAL LOADS
C
```

```
      H = 0.0
      V = - LOADS(I)
C
      RETURN
      END
      SUBROUTINE VALHOR (I,LOADS,V,H)
C
C         OBTAIN THE EFFECTIVE LATERAL FORCE ON BLOCK I
C
      DIMENSION LOADS(1)
      REAL LOADS
C
      H = LOADS(I)
      V = 0.0
C
      RETURN
      END
      SUBROUTINE VALHST (I,PRESS,P1,P2,YCOMP)
C
C         OBTAIN THE HYDROSTATIC PRESSURE ON BLOCK I
C
      DIMENSION PRESS(1)
      LOGICAL YCOMP
C
      P1 = PRESS(I+1)
      P2 = PRESS(I  )
      YCOMP = .TRUE.
C
      RETURN
      END
      SUBROUTINE VALHDY (I,PRESS,P1,P2,YCOMP)
C
C         OBTAIN THE HYDRODYNAMIC PRESSURE ON BLOCK I
C
      DIMENSION PRESS(1)
      LOGICAL YCOMP
C
      P1 = PRESS(I+1)
      P2 = PRESS(I  )
      YCOMP = .FALSE.
C
      RETURN
      END
      SUBROUTINE CENTRD (P1,P2,DX,DY,PX,PY,RX,RY)
C
C         COMPUTE THE RESULTANT PRESSURE FORCE ON A SURFACE,
C         ALSO LOCATES THE VERTICAL CENTROID OF A BLOCK
C
      A  = 0.5*(P1+P2)
      PX =   A*DY
      PY = - A*DX
C
      RX = 0.0
      RY = 0.0
      IF (A.EQ.0.0) RETURN
```

```
C
      A  = (2.0*P1+P2)/(6.0*A)
      RX = A*DX
      RY = A*DY
C
      RETURN
      END
      SUBROUTINE COMSTA (NBLOCK,STR1,STR2,STR3,STRFAC,TITLE)
C
C         ADD STRESSES STR2 TO STR1 AND PUT IN STR3
C
      DIMENSION STR1(2,1),STR2(2,1),STR3(2,1)
      CHARACTER*40 TITLE
C
      WRITE (*,99) TITLE
C
      DO 10 J=1,NBLOCK
          I = NBLOCK + 1 - J
          STR3(1,I) = (STR1(1,I) + STR2(1,I))*STRFAC
          STR3(2,I) = (STR1(2,I) + STR2(2,I))*STRFAC
          WRITE (*,98) I,STR3(1,I),STR3(2,I)
C
   10 CONTINUE
C
      RETURN
C
   99 FORMAT (//2X,A40//' BLOCK',5X,'UPSTREAM FACE',2X,'DOWNSTREAM FACE'
     1        /1X,40('-')/)
   98 FORMAT (3X,I2,10X,F8.3,9X,F8.3)
C
      END
```

NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

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