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SOME CRUCIAL ISSUES IN EARTHQUAKE RESPONSE ANALYSIS OF GRAVITY DAMS

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ABSTRACT

The traditional pseudo-static method for design of gravity dams is governed mainly by the static stability against sliding, and not necessarily by the stresses. This method is based on highly reduced values of the seismic forces leading to unrealistically small values of the dynamic stresses due to earthquakes. On the other hand, the standard finite element method used for detailed time-history analyses is based on several unrealistic assumptions, which result in unrealistically high stresses in the dam. To ensure the seismic safety of gravity dams in a realistic way, the dynamic response analysis is required to be carried out with realistic modeling of the dam-water-foundation interaction effects. The effect of the stochastic nature of earthquake excitation on the response is also required to be accounted. Finally, the material properties of the dam and the foundation rock and the performance criteria used to assess the safety of dams also play very important role. This paper highlights the significance of all these crucial issues and suggests the ways to handle them appropriately for arriving at reliable design and evaluation of gravity dams.

KEYWORDS: Gravity Dam, Foundation-Reservoir Interactions, Pseudo-Dynamic Method, Time-History Response Analysis, Substructure Method, Stochastic Effects, Material Properties

INTRODUCTION

Dams are constructed across rivers to create reservoirs of water for human consumption, irrigation, industrial use, and generation of electricity. These are thus the infrastructures of great socioeconomic importance to be safeguarded against all major hazards. History of dams in Indian sub-continent goes back to pre-Harappan period. The earliest dams are believed to have been built of stone rubbles by Zoroastrians in Baluchistan. Earthen embankments were constructed in Sri Lanka to store water as early as 400 B.C. Sinhalese built several dams in 500 A.D., some of which are still in use. Rajatarangini, a chronicle of ancient Kashmir by Kalhana, gives a detailed account of well-conceived and maintained dams and irrigation canals during 8-12th century A.D. in Kashmir. The 11th century Veeranam dam in central India and Mudduk Maur Dam near Bhopal constructed in 1500 A.D. are among the oldest dams in India. Out of nearly 57,000 large dams (15 m or higher) existing the world over, China has the maximum number of about 19,000, followed by United States with 5,500. Only about 7.7 % (about 4400) of the world's large dams exist in India against a population of about 17 %. Large number of dams may have to be constructed in India to meet the future water needs for various uses.

The concrete or masonry type of gravity dams and the earth and rock-fill type of embankment dams are the two most common types of dams. During early days, gravity dams were designed to resists the water pressure and other overturning forces only by their own weight, and hence named as gravity dams. Considerations for seismic forces in dam design were introduced only after the Kwanto earthquake of 1923 in Japan. Provisions for earthquake loads were not in vogue in India till the code on earthquake resistant design of structures was first published in 1962. Gravity dams in India are commonly designed using the pseudo-static method of analysis outlined in the code IS: 6512-1984 [1]. The biggest drawback in this approach is that the values of the seismic coefficient used to define the earthquake forces are very small compared to the true earthquake motion. This leads to unrealistically low values of the stresses to govern the design and seismic safety of dams. Many other issues crucial to the earthquake response of gravity dams are ignored or not handled appropriately in the traditional pseudo-static method as well as in the standard finite element (FE) analysis. The seismic safety of gravity dams can be ensured only if they are designed for the dynamic stresses expected during real earthquakes plus the initial static stresses due to self-weight and the hydrostatic force. However, the standard dynamic response analysis by commercial software is unable to model the effects of the dam-foundation rock interaction and the hydrodynamic forces accurately [2, 3].

The dam engineering has evolved through trial and error since ancient times, but the most significant developments have taken place only during last 100 years or so. Methods for detailed dynamic response analysis of gravity dams have been developed more recently as a result of considerable research after the observed damage to the Koyna dam during the earthquake of 11 December 1967. Methods with varying levels of sophistication are now available for computation of dynamic stresses in gravity dams considering all important aspects. However, in majority of practical applications, the design of gravity dams can be finalized only from two dimensional linear elastic stress analyses with judicious interpretation of the results [e.g., 4-6]. The initial design can be based on a simplified pseudo-dynamic method [7, 8], and if found necessary, the linear time-history analysis based on the substructure method [9] may be used to finalize the design.

The dynamic response analysis of gravity dams is also required to account for the effects of the stochastic nature of the ground motion, because the common use of a single set of horizontal and vertical time-histories of ground motion is not adequate to account for the uncertainties due to non-repeatable nature of the earthquake ground motion. Finally, to arrive at dependable engineering decisions about the safety of existing as well as new dams, it is necessary that dynamic stresses are estimated using actual material properties determined by testing of core samples, and the safety is also assessed using actual strength of the concrete.

This paper presents a critical discussion on the various issues crucial to the dynamic response analysis of gravity dams, which includes: (i) methods of dynamic analyses, (ii) dam-foundation rock interaction, (iii) dam-reservoir water interaction and reservoir bottom absorption, (iv) effects of the stochastic nature of the input earthquake excitation, (iv) input material properties of the dam and the foundation rock and (v) performance criteria to ensure the safety of dams. However, a brief review is first presented on the seismic coefficient and the conventional pseudo-static method and its limitations. The issue of site-specific estimation of design earthquake ground motion has been skipped, because there is already good awareness to use the site-specific design ground motion for all important dams [10].

SEISMIC COEFFICIENT AND THE PSEUDO-STATIC METHOD

Gravity dams in India are commonly designed by pseudo-static method [1, 11-13], in which the earthquake effects are represented by equivalent static forces defined in terms of the 'Seismic Coefficients'. The design of dams by this method is governed mainly by the stability against sliding, and not by the stresses. Though the stresses are estimated on the outer two faces of the dam using the formulas for bending of a cantilever beam [14], those are unrealistically small due to small values of the seismic coefficients used. The pseudo-static method is thus considered an outdated approach for the design of dams in most parts of the world. Before describing the other modern methods for the design of gravity dams in the subsequent sections, this section presents a brief overview on the seismic coefficient and the pseudo-static method and its limitations.

1 Seismic Coefficient

The need for earthquake-resistant design of structures was felt in view of several destructive earthquakes in different parts of the world in the beginning of the twentieth century. However, the first seismic design code was introduced in Japan in 1924 after the Kwanto earthquake of 1923. This code proposed to consider the seismic effects by horizontal static force equal to a small fraction (termed as seismic coefficient) of the weight of the building. In India, values of seismic coefficients varying from 0.05 to 0.15 were first suggested by Kumar [15] after the M 7.4 Mach earthquake of 1931. However, the recommendations for earthquake loads were not formulated in India till the code for earthquake resistant design of structures was first published in 1962, which divided the country into several zones and proposed the values of seismic coefficients for different types of structures in each zone. Since then, the Indian code has been revised during 1966, 1970 and 1984, and the current edition of the code IS: 1893-2016, Part 1 [16] gives a zoning map with four zones as shown in Figure 1.

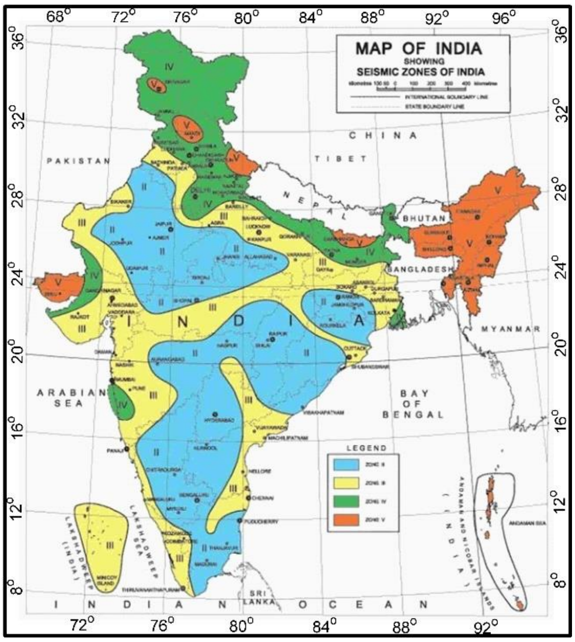

Fig. 1 Seismic zoning map of India as per IS: 1893 (Part 1)-2016 [16]

As per the current Indian code, the horizontal seismic coefficient, A_h, for different types of structures, is defined as

$$A_h = \frac{(Z/2)(S_a/g)}{(R/I)} \tag{1}$$

where Z is the zone factor equivalent to the expected peak ground acceleration (PGA) in each zone, S_a/g is the amplitude at natural period of the structure from the normalized standard response spectral shapes given in the code for three different types of site condition, R and I are the response reduction and importance factors, values of which for different types of structures are given in the corresponding parts of the code (e.g., Part 1 for buildings, Part 2 for liquid retaining tanks, Part 3 for bridges, and Part 4 for industrial structures). Part 5 for dams has not been yet published, and hence the seismic coefficient for dams is still estimated according to the provisions of the 1984 edition of the code along with the zoning map given in Figure 1. The seismic coefficient for the gravity dams thus estimated can be shown to be equal to 3/4th the zone factor Z in the current code, which gives horizontal seismic coefficient of 0.075, 0.12, 0.18, and 0.27 for zones II, III, IV and V, respectively. The guidelines issued by National Committee on Seismic Design Parameters [10] proposes to define the site-specific horizontal seismic coefficient as 2/3rd of the amplitude at natural period of 0.2 s of the site-specific acceleration response spectrum for design basis earthquake with damping ratio of 5 % divided by 2.5. However, the seismic coefficient is not a standard strong motion parameter and no methodology exits to compute it from recorded accelerograms. Thus, the practice of defining sit-specific seismic coefficient needs to be discarded.

2 Pseudo-Static Method and its Limitations

The pseudo-static method described in the Indian code IS: 6512-1984 [1] is mainly used to check the static stability of a dam against sliding along a potential failure surface at the base or a higher level under different load combinations prescribed in the code. The factor of safety, F, against sliding is defined in terms of the partial factors of safety in respect of friction (F_ϕ) and cohesion (F_c) as follows

$$F = \frac{\dfrac{\sum (W-U)\tan\phi}{F_\phi} + \dfrac{C \cdot A}{F_c}}{P} \tag{2}$$

In this expression, W is the total vertical load, U is total uplift force, $\tan\phi$ is coefficient of internal friction of the material, C is cohesion of the material at the plane considered, A is the area of plane under consideration for cohesion, and P is the total horizontal force. For stability against sliding for a given load combination, the value of F is required to be greater than or equal to 1.0 with the corresponding F_{ϕ} and F_c factors given in the code for that load combination. The code does not recommend any separate check for the stability against overturning under the assumption that the dam will be safe against overturning if it is safe against sliding and the stresses are within specified limits.

The pseudo-static method permits only nominal tensile stresses in the range of 1 to 4 % of the compressive strength of the concrete for different combinations of the earthquake and other static loads [1]. The maximum value of 4 % is permitted under the most severe load combination of earthquake with full reservoir and extreme uplift pressure. The safety criteria for compressive stresses require the compressive strength to be more than or equal to four times the maximum compressive stress or 14 MPa, whichever is higher. Such stringent stress criteria have no resemblance to the stresses during real earthquakes and can be satisfied only by using unrealistically low values of the seismic coefficient. Further, the pseudo-static method does not account for the dam-foundation rock interaction effects and define the hydrodynamic pressure by added mass formulation due to Westergaard [17] or Zangar [18], which are based on unrealistic assumptions of rigid dam and incompressible water.

PSEUDO-DYNAMIC METHOD

To overcome the limitations of the conventional pseudo-static method it is necessary that the philosophy of dam design is changed from the static stability to the stresses due to actual site-specific earthquake excitation plus the stresses due to the static loads (self-weight, hydrostatic pressure, uplift pressure, etc.). The method of dynamic analysis should be able to model the effects of the semi-infinite extents of the foundation and the reservoir domains in a realistic way. These requirements are fortunately satisfied by a simplified method developed by Fenves and Chopra [7], which is equivalent in simplicity to the pseudo-static method. This method, termed as pseudo-dynamic method in this paper, can be considered to provide a better alternative to the pseudo-static method for preliminary design of gravity dams.

The pseudo-dynamic method [7] is a simplified version of the substructure method developed by Fenves and Chopra [9] for rigorous time-history response analysis of gravity dams. This method can be used to conveniently estimate the maximum principal stresses on the upstream and downstream faces of the dam with the earthquake excitation specified by site-specific response spectrum amplitudes. The original method developed for the non-overflow section of the dam has been generalized by Chopra and Tan [19] to analyze a gated spillway monolith also. The latest version of the simplified method of analysis for non-overflow section as given in Løkke and Chopra [8, 20] is recommended to be used in design applications.

1 Approximations for Earthquake Forces

The pseudo-dynamic method is fundamentally based on approximating the lateral earthquake force on the upstream face of the dam at elevation y by two equivalent static forces $f_1(y)$ and $f_{sc}(y)$ in the fundamental (first) mode and the higher modes of vibration of the dam, respectively. Both these forces are defined by accounting for the dam-water-foundation interaction effects using a number of variables which can be obtained directly from the standard tables and plots given in [8, 19, 20] as a function of the following parameters of the dam-foundation-reservoir system:

E_s = Young's modulus of elasticity of the dam,

ξ_1 = Viscous damping of the dam on rigid foundation and empty reservoir,

H_s = Height of dam,

$\phi_1(y/H_s)$ = Standard first mode deformation shape of the dam at elevation y,

$w_s(y)$ = Weight per unit height of the dam at elevation y,

E_f = Young's modulus of elasticity of the foundation rock,

η_f = Hysteretic damping ratio of the foundation rock,

H = Depth of water in the reservoir,
α = Fraction of hydrodynamic pressure waves reflected from reservoir bottom,
$p(y)$ = Hydrodynamic pressure at elevation y on the upstream face of the dam.

Further, the earthquake excitation for the force $f_1(y)$ is specified in terms of the pseudo-acceleration ordinate, $PSA(\tilde{T}_1, \tilde{\xi}_1)$, at vibration period $\tilde{T}_1$ and damping ratio $\tilde{\xi}_1$ of the dam-water-foundation system. The period $\tilde{T}_1$ is obtained from the period T_1 of the dam on rigid foundation with empty reservoir using period lengthening ratios R_r and R_f due to the dam-reservoir and dam-foundation interaction effects, respectively, as $\tilde{T}_1 = R_r R_f \tilde{T}_1$, with

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_s}} \tag{3}$$

Standard values of R_r are tabulated in Fenves and Chopra [7] in terms of H/H_s, E_s and α; and that of R_f are tabulated in terms of E_f / E_s. Fenves and Chopra [7] have also given an expression for $\tilde{\xi}_1$ in terms of ξ_1 (viscous damping ratio of the dam on rigid foundation with empty reservoir), ξ_r (added damping due to dam-water interaction and reservoir bottom absorption), and ξ_f (added damping due to dam-foundation interaction), with the standard values of ξ_r tabulated in terms of H/H_s, E_s and α; whereas that of ξ_f in terms of E_f / E_s and η_f. As it is difficult to assign the values of ξ_1 and η_f with good accuracy, it may be a good choice to directly assume a value of $\tilde{\xi}_1$ = 0.05 in practical applications. On the other hand, the earthquake excitation for the force $f_{sc}(y)$ in higher modes is defined simply in terms of the PGA, which can be approximated by the zero-period pseudo-acceleration ordinate.

For the convenience of practical use, the hydrodynamic pressure at elevation y on the upstream face of the dam, $p(y)$, is given by Fenves and Chopra [7] in a non-dimensional form $gp(\hat{y})/\gamma_w H$ with $\hat{y} = y/H$ and $\gamma_w = 9.81$ kN/m^3 as the unit weight of water. Standard values of this non-dimensional function is tabulated in terms of the wave reflection coefficient α and the period ratio $R_w = T_1^r / \tilde{T}_r$ with $\tilde{T}_r = R_r T_1$ and T_1^r as the fundamental vibration period of the impounded water given by $4H/C_w$, where $C_w = 1440$ m/s is the velocity of pressure waves in the water.

2 Estimation of Stresses

The stresses in the pseudo-dynamic method are estimated by static analysis of the dam alone to the equivalent lateral forces $f_1(y)$ and $f_{sc}(y)$. For this purpose, the dam section is divided into several horizontal blocks and the normal bending stresses are computed at the base of each block on the upstream and the downstream faces of the dam using the elementary beam theory, similar to that in the pseudo-static method.

The forces $f_1(y)$ and $f_{sc}(y)$ acting per unit height on the upstream face of the dam consist of two parts: the first part representing the inertial force associated with the mass of the dam and the second part representing the hydrodynamic force due to the impounded water. The bending moment at the base of each block are computed by applying the resultant inertial part of the force at the centroid of the block and varying the hydrodynamic part of the force linearly between the base and the top of the block. Assuming the dam monolith to behave like a cantilever beam, the normal bending stresses $\sigma_{y,1}$ and $\sigma_{y,st}$ at the base of a block at elevation y due to the first and the higher modes of vibration, respectively, are obtained simply by dividing the bending moments with the section modulus of the base of the selected block. By comparing the stresses from beam theory with the exact stresses computed by finite element method, Løkke and Chopra [8, 20] have found that the formulas based on beam theory overestimate the stresses at the downstream sloping face of gravity dams. For locations on downstream face with slope not much steeper than 0.8:1.0, they have suggested a multiplicative correction factor of 0.75 to get the stresses in good agreement with the values obtained by finite element method.

The combined vertical dynamic stress $\sigma_{y,d}$ at elevation y due to the first and the higher modes under earthquake excitation can be obtained by the SRSS method as:

$$\sigma_{y,d} = \pm\sqrt{\sigma_{y,1}^2 + \sigma_{y,sc}^2} \tag{4}$$

If the earthquake force is assumed to act from upstream to downstream direction, the stresses will be positive (tensile stresses) on the upstream face and negative (compressive stresses) on the downstream face and vice versa. If $\sigma_{y,st}$ is the initial normal stress at elevation y due to the various static forces (self-weight, hydrostatic pressure, uplift pressure, etc.) the combined dynamic and static normal stresses have to be obtained with the plus or minus sign for $\sigma_{y,d}$ such that the resulting normal stresses on the upstream and the downstream faces have maximum and minimum values, respectively. If the upstream face of the dam is nearly vertical and the effects of tail-water are negligible, the vertical stresses $\sigma_{y,d}$ and $\sigma_{y,st}$ can be used to obtain the corresponding principal stresses σ_d and σ_{st} as follows [14]

$$\sigma_d = \sigma_{y,d} \sec^2\theta \text{ and } \sigma_{st} = \sigma_{y,st} \sec^2\theta \tag{5}$$

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

3 Illustrative Numerical Results

To compute the illustrate numerical results by pseudo-dynamic method we have considered the tallest non-overflow section of monolith number 17 of Koyna dam under excitation by the longitudinal component of the accelerogram recorded at the foundation gallery of the dam during the Koyna earthquake of 11 December 1967 with magnitude 6.5. The dam, which was designed for horizontal seismic coefficient of 0.05 by the traditional pseudo-static method experienced significant horizontal cracks during this earthquake in the upstream and/or downstream faces of a number of non-overflow monoliths near the elevation at which there is an abrupt change in the slope of the downstream face. Figure 2(a) shows the discretization of the Koyna dam monolith 17 into 12 horizontal blocks. With the origin at the heel of the dam, x-axis in the downstream direction and y-axis in the vertically upward direction, Figure 2(a) also gives the coordinates of the upstream and downstream faces at the base of each block and at the crest of the dam. Figure 2(b) gives the pseudo acceleration response spectra for damping ratios of 0, 2, 5, 10 and 20 % of the critical damping for the longitudinal component of the accelerogram recorded at the foundation gallery of the dam during earthquake of 11 December 1967 with magnitude 6.5, which has been used as the input excitation.

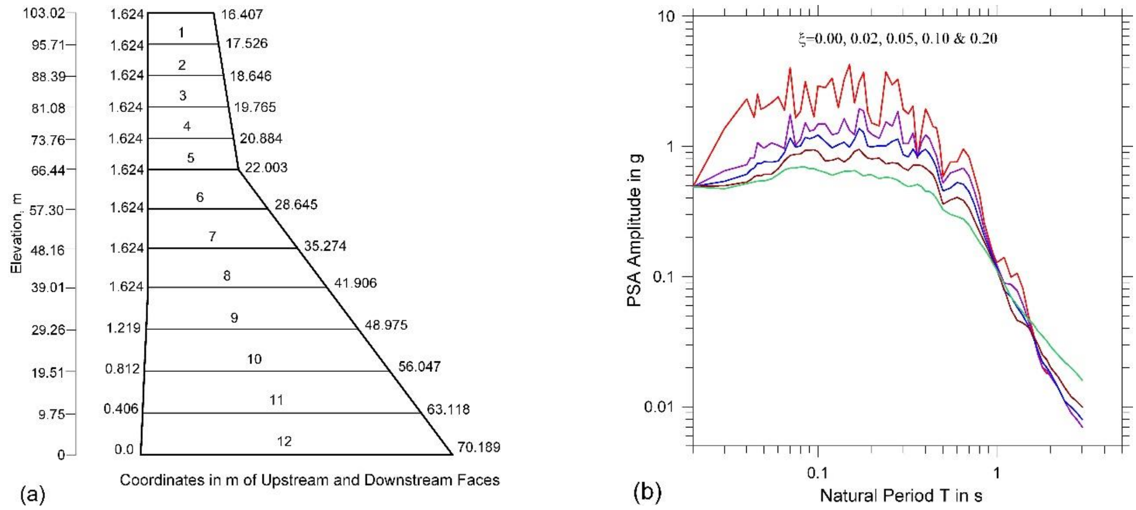

Fig. 2 (a) Tallest non-overflow monolith of Koyna dam divided into horizontal blocks for the purpose of response computation and (b) PSA spectra of the accelerogram used as input excitation

To model the hydrodynamic effects, the free-surface of the reservoir is taken at elevation of 91.8 m, which is the water level at the time of the December 1967 earthquake. The value of the wave reflection coefficient α to model the reservoir bottom absorption effects is taken as 0.50. The Young's modulus of elasticity of the dam concrete is taken equal to 23,000 MPa corresponding to M21 grade of concrete used in the dam construction. The modulus of elasticity for the basalt type of foundation rock is taken equal to 38,000 MPa from the laboratory tests conducted at USBR [21]. The dynamic tensile stresses obtained on the upstream and the downstream faces of the dam monolith due to the earthquake excitation alone are shown in Figure 3(a) and the combined dynamic and static tensile stresses due to self-weight and the hydrostatic forces are shown in Figure 3(b).

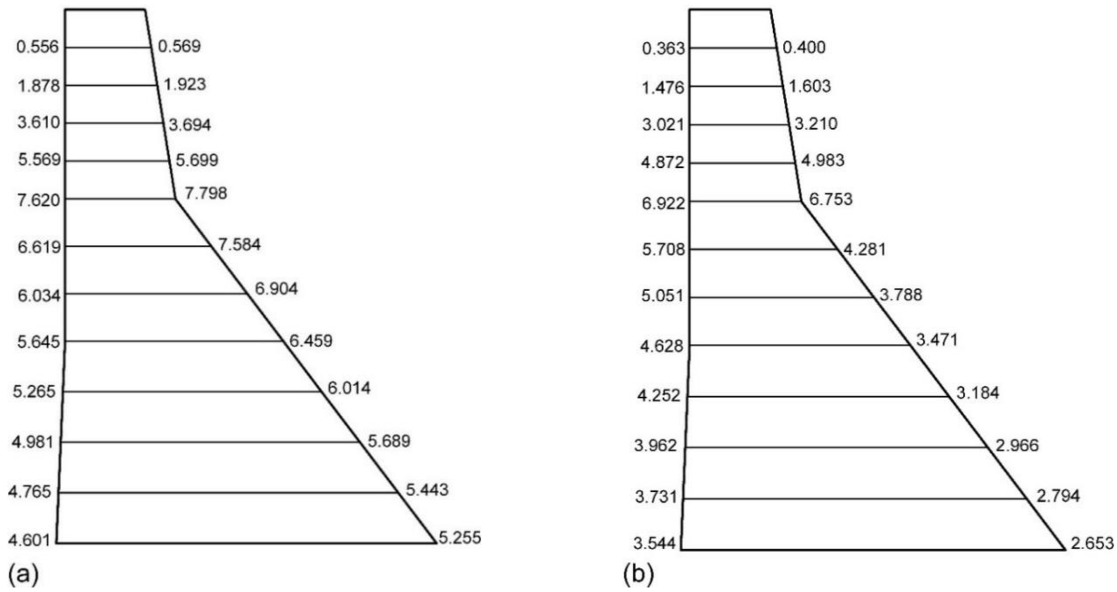

Fig. 3 Maximum principal stresses (in MPa) in the tallest monolith of Koyna dam with (a) static stresses excluded and (b) static stresses included

The self-weight generates compressive stresses at both the upstream and the downstream faces, which are much higher on the upstream face. However, the hydrostatic force generates tensile stress on the upstream face and compressive stress on the downstream face, but the tensile stresses on the upstream face are significantly smaller than the compressive stress due to the self-weight. Thus the combined tensile stresses on both the faces are lower than the dynamic tensile stresses, but these are still very high with the greatest values near the elevation where the slope of the downstream face changes abruptly. The computed stresses at several locations are higher than the permitted apparent dynamic tensile strength of about 4.0 MPa as per the performance criteria to be described later. Hence, significant cracking, consistent with what was observed after the 1967 earthquake, could have been anticipated even by the simplified pseudo-dynamic analysis.

It may be concluded that the pseudo-dynamic method provides a better alternative to the conventional pseudo-static method for the preliminary design of gravity dams. This method defines the earthquake forces more realistically using actual response spectrum amplitude at the fundamental period of the dam, and the dam-water-foundation interaction effects are also taken into account. The hydrodynamic force is defined more realistically in terms of a pressure function obtained by considering the flexibility of water and the absorption of the pressure wave at the reservoir bottom. This method is thus able to predict the maximum stresses on the upstream and downstream faces of the dam in good comparison with what may happen during real earthquakes. The design or safety evaluation of a dam on the basis of these stresses would be able to ensure the seismic safety in a more realistic way. The safety against sliding and overturning is then expected to be ensured automatically, because earthquake damage will be indicated much before any static instability takes place.

TIME HISTORY RESPONSE ANALYSIS

The pseudo-dynamic method is able to predict only the maximum values of the principal stresses on the two faces of the dam cross section. If these values at certain elevations of the dam exceed the strength of the concrete that cannot be taken to imply that the dam is unsafe. The safety of a dam is in reality governed by the area of the dam cross-section overstressed concurrently as well as by the duration of the overstressing. A comprehensive safety evaluation of a dam thus requires computing the time-histories of the response with realistic modeling of dam-foundation rock interaction and the hydrodynamic forces on the upstream face of the dam. The standard finite element (FE) method, used commonly for time-history response analysis with the help of commercially available software like ABAQUS, ANSYS, SAP 2000, LSDYNA, LUSAS, FLAC, etc., suffers from several drawbacks and does not provide realistic estimate of the stresses in the dam. On the other hand, commercial software cannot be used directly to perform a rigorous FE time-history analysis to overcome these drawbacks. A more convenient approach for time-history analysis of gravity dams including all the significant effects of dam-water-foundation interaction and reservoir bottom absorption is the substructure method, which can be implemented easily in practical applications using the special purpose computer program EAGD-84 developed by Fenves and Chopra [9]. This section first provides an overview of the standard and the rigorous FE methods, and then describes the substructure method in some details.

1 Standard Finite Element Method

Analysis of the dam alone with rigid foundation is a simple problem to be solved by FE analysis, but the actual problem is greatly complicated due to the presence of the reservoir of water on the upstream side and the flexibility of the foundation rock, both extending to infinity. In the standard FE analysis using commercial software, the dam and only a limited portion of the foundation rock are included in the finite element modeling, and the hydrodynamic effects are modeled simply by attaching a mass of water to the upstream nodes of the dam as shown in Figure 4. The bottom boundary of the foundation block is assumed to be fixed in both horizontal and vertical directions and the side boundaries to be fixed in vertical direction. The foundation block is assumed to have no mass, so that the design ground motion defined at the ground surface (free-field motion) can be applied at the bottom fixed boundary. No ground motion is applied at the side boundaries of the foundation block. Though, this approach is simple to implement, it suffers from several major limitations.

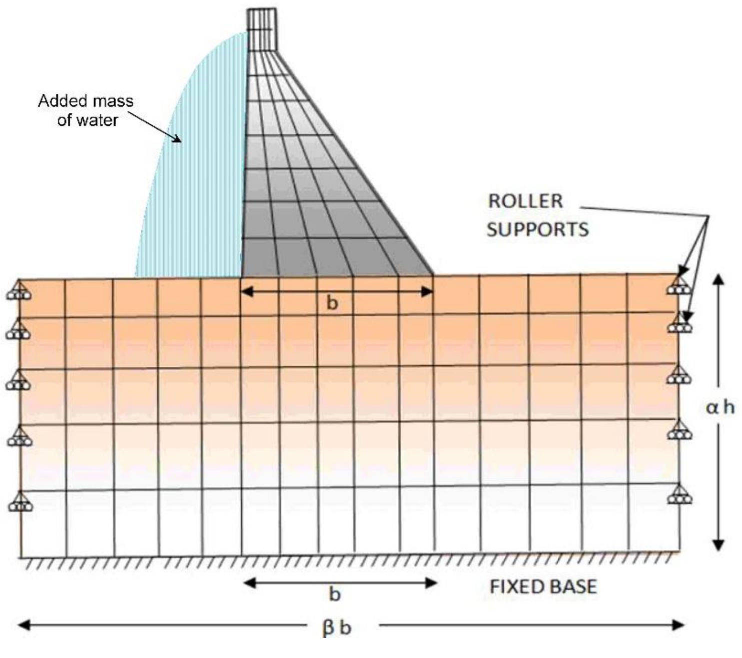

Fig. 4 Standard FE analysis by including a limited portion of the foundation with no mass and fixed base, and the hydrodynamic forces modeled by a mass of water attached to the upstream face of the dam

For the concrete dam foundations, similar type of rock usually extends to large depths without any natural rigid boundary at a finite depth. Due to the artificial rigid boundary introduced in the analysis, the stress waves induced in the foundation block are reflected back towards the dam rather than radiating outward infinitely. The dissipation of energy by internal hysteresis is also not accounted in the massless foundation. Thus the stresses in the dam are greatly overestimated due to the loss of the damping mechanisms in the foundation. The modeling of hydrodynamic force by the inertial effect of the added mass of water is based on the unrealistic assumptions of rigid dam and incompressible water [17, 18], which causes unacceptable errors in the estimation of hydrodynamic forces [4, 22]. The computed dam response may be under or over-estimated if the effects of the flexibility of dam and the absorption of hydrodynamic pressure waves at the reservoir bottom are not accounted [10, 23 and 24].

2 Rigorous Finite Element Analysis

To overcome the limitations of the standard FE method it is necessary that the finite parts of the foundation rock and the reservoir water are both modeled by finite element with absorbing boundaries at (i) the upstream end of the finite length of the reservoir and (ii) the bottom and the sides of the foundation domain included into the analysis as illustrated in Figure 5. The mass and the material damping of the foundation rock and the compressibility of water should also be taken into account. The fluid elements have to be connected to the foundation rock at the reservoir bottom and to the upstream face of the dam through tied contact surfaces.

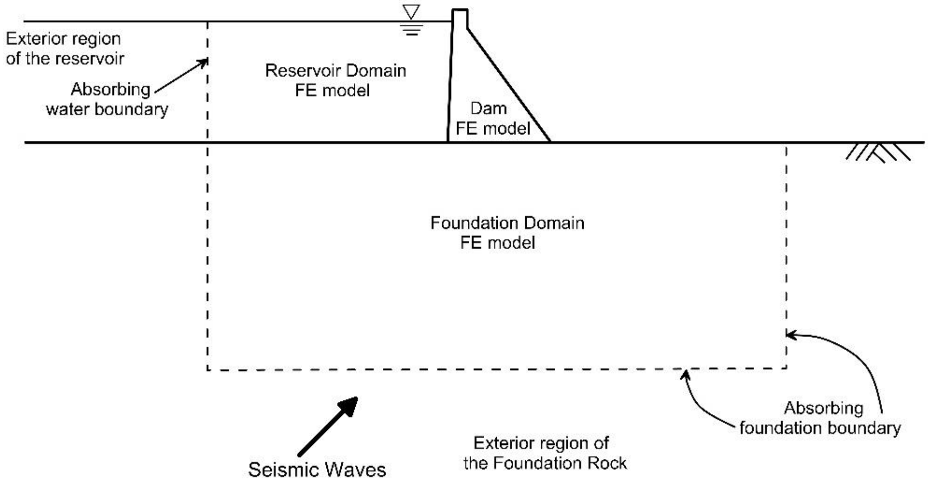

Fig. 5 Dam-reservoir-foundation system with truncated foundation and reservoir domains with wave-absorbing boundaries

The absorbing boundaries can be modeled by applying viscous dampers representing the effects of the semi-unbounded exterior regions of the foundation and the reservoir domains [25]. But the earthquake motion cannot be applied directly at such boundaries, as that would make the absorbing boundary ineffective. The design earthquake motion at the free-surface of the ground is required to be transformed into the effective earthquake forces at the absorbing boundaries at the bottom and the vertical sides of the foundation block and at vertical upstream boundary of the reservoir. The commercial FE software lacks the facility to compute these effective earthquake forces, and these have to be thus computed separately and integrated with the commercial software, which requires very high level of expertise.

3 The Substructure Method

The substructure method, developed by Chopra and co-workers [26-28], provides a convenient alternative to the rigorous FE time-history analysis of gravity dams. In this method, the complete system comprising the dam, the foundation-rock as a semi-infinite continuum and the reservoir water extending to infinity in the upstream direction (Figure 6) is analyzed as three independent substructures. The dam-foundation rock interaction and the hydrodynamic forces are obtained by solving the wave equation separately for the foundation rock and the reservoir water substructures, respectively. The selected cross-section of the dam is then analyzed as a separate substructure by two-dimensional FE analysis under linear elastic behavior with the soil-structure and hydrodynamic forces incorporated into the equations of motion.

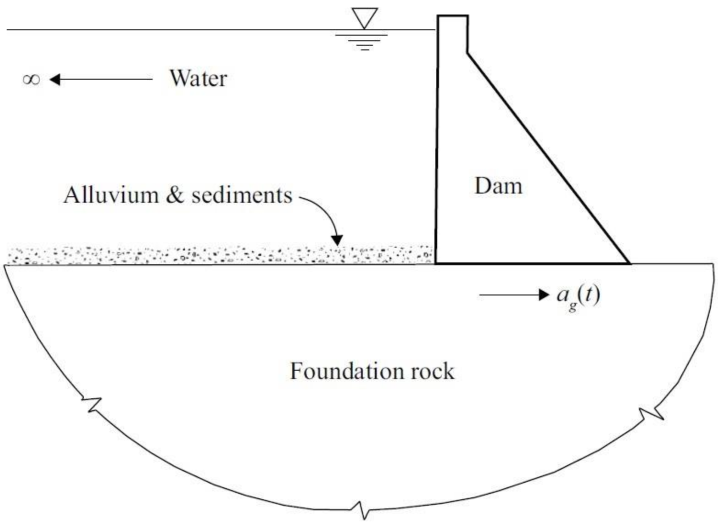

Fig. 6 Dam-foundation-reservoir system in substructure modeling (after [9])

The solutions of the wave equation for both the foundation rock and the reservoir water sub-structures can be performed more conveniently in the frequency domain for a harmonic acceleration of unit amplitude [26]. Response to an arbitrary ground motion is then determined by Fourier transforming the ground motion, determining the steady state response for a range of frequencies over which the ground motion and structural response have significant components, and then performing a Fourier synthesis of the frequency-response to obtain the response in the time domain [27-29].

3.1 Mathematical Formulation in Frequency Domain

In the substructure approach, equations of motion are formulated and solved for the frequency response functions $\bar{r}_c^l(\omega)$ of the nodal displacements $r_c^l(t)$ of the dam under excitation by a harmonic excitation $a_g^l(t) = e^{i\omega t}$ of unit amplitude, with $l = x$ (horizontal) and $l = y$ (vertical) direction. The hydrodynamic forces $R_h^l(t)$ at the upstream face of the dam are specified explicitly through their frequency response function $\bar{R}_h^l(\omega)$; whereas the dam-foundation interaction forces are introduced implicitly through a complex-valued dynamic stiffness matrix $S_f(\omega)$ for the foundation rock region and the unknown frequency response functions $\bar{r}_b^l(\omega)$ of the displacements of the base nodes of the dam. The functions $\bar{R}_h^l(\omega)$ and the matrix $S_f(\omega)$ are obtained by analyses of the reservoir and foundation substructures, independently.

For efficient solution of the frequency domain equations of motion, the $\bar{r}_c^l(\omega)$ is expressed in terms of a limited number of the complex-valued frequency-response functions $\bar{Z}_j^l(\omega)$ of the generalized coordinates $Z_j^l(t)$ and the corresponding mode shapes ψ_j for the j^th normal mode (Ritz vector) as

$$\bar{r}_c^l(\omega) = \sum_{j=1}^{J} \psi_j \bar{Z}_j^l(\omega) \tag{6}$$

With this transformation, the equations of motions to be solved for J number of generalized coordinates $\bar{Z}_j^l(\omega)$ can be written in the following matrix form

$$H(\omega))\bar{Z}^l(\omega) = L^l(\omega) \tag{7}$$

where the elements of the matrix $H(\omega)$ and the vector $L^l(\omega)$ are given by

$$H_{ij}(\omega) = \left[-\omega^2 + (1+i\eta_s)\lambda_i^2\right]\delta_{ij} + \psi_i^T\left[\widetilde{S}_f(\omega) - (1+i\eta_s)\widetilde{S}_f(0)\right]\psi_j \tag{8a}$$

$$L_i^l = -\psi_i^T M_c I_c^l + \psi_{fi}^T \bar{R}_h^l(\omega) \tag{8b}$$

in which δ_{ij} is the Kronecker delta function, η_s is the constant hysteretic damping factor for the dam concrete (equal to twice the equivalent viscous damping), $\tilde{S}_f(0)$ is the static value of the foundation stiffness matrix, and ψ_{fi} is a sub-vector of ψ_j that contains only the elements corresponding to the nodal points at the upstream face of the dam. From knowledge of the complex-valued frequency-response functions $\bar{Z}_j^l(\omega)$ obtained by simultaneous solution of Equation (7) at each frequency ω, the complex Fourier transform of the response of the dam to actual ground accelerations $a_g^l(t)$ can be obtained as

$$\bar{r}_c(\omega) = \sum_{j=1}^{J} \psi_j \left[\bar{Z}_j^x(\omega) A_g^x(\omega) + \bar{Z}_j^y(\omega) A_g^y(\omega) \right] \tag{9}$$

where $A_g^l(\omega)$ is the complex Fourier transform of the l-component of the actual free-field ground acceleration $a_g^l(t)$. The time-history of the displacement response can then be obtained by the inverse Fourier transformation of $\bar{r}_c(\omega)$ as

$$r_c(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} \bar{r}_c(\omega) e^{i\omega t} d\omega \tag{10}$$

These can be used to estimate the components of the stresses, $\sigma_n(t)$, at the centroid of the nth element as

$$\sigma_n(t) = T_n r_n(t) \tag{11}$$

where T_n is the stress-displacement transformation matrix.

3.2 Analysis of Reservoir Substructure

To compute the frequency response of the hydrodynamic forces on the upstream face of the dam, the reservoir sub-structure is idealized by vertical upstream face of the dam and horizontal reservoir bottom extending to infinity on the upstream side. These assumptions do not have any significant effect if the upstream face of the dam is vertical even for half the height and depth of water is approximately constant for reservoir length up to three times or more the water depth. The wave equation to be solved for the reservoir domain to get the frequency response function, $\bar{p}(x, y, \omega)$, of the hydrodynamic pressure, $p(x, y, t)$, becomes the Helmholtz's equation

$$\frac{\partial^2 \bar{p}}{\partial x^2} + \frac{\partial^2 \bar{p}}{\partial y^2} + \frac{\omega^2}{C_w^2} \bar{p} = 0 \tag{12}$$

where C_w is the velocity of pressure waves in water. This equation needs to be solved for three different sets of boundary conditions on the upstream face of the dam, the reservoir bottom, the free-surface of water and at infinitely large distance on the upstream of the dam as illustrated in Figure 7.

The boundary conditions in the top and middle diagrams correspond to the frequency-response functions $\bar{p}^x(x, y, \omega)$ and $\bar{p}^y(x, y, \omega)$ for the horizontal and vertical ground accelerations of a rigid dam, respectively. The bottom diagram corresponds to the frequency-response function $\bar{p}_j^f(x, y, \omega)$ due to horizontal acceleration of the upstream face of the dam, which is represented by the j^{th} Ritz vector for the dam that contains only the elements corresponding to the *x*-DOF of the nodal points at the upstream face of the dam. The total hydrodynamic pressure is obtained simply by adding the solutions of Equation (12) obtained independently for the three sets of boundary conditions.

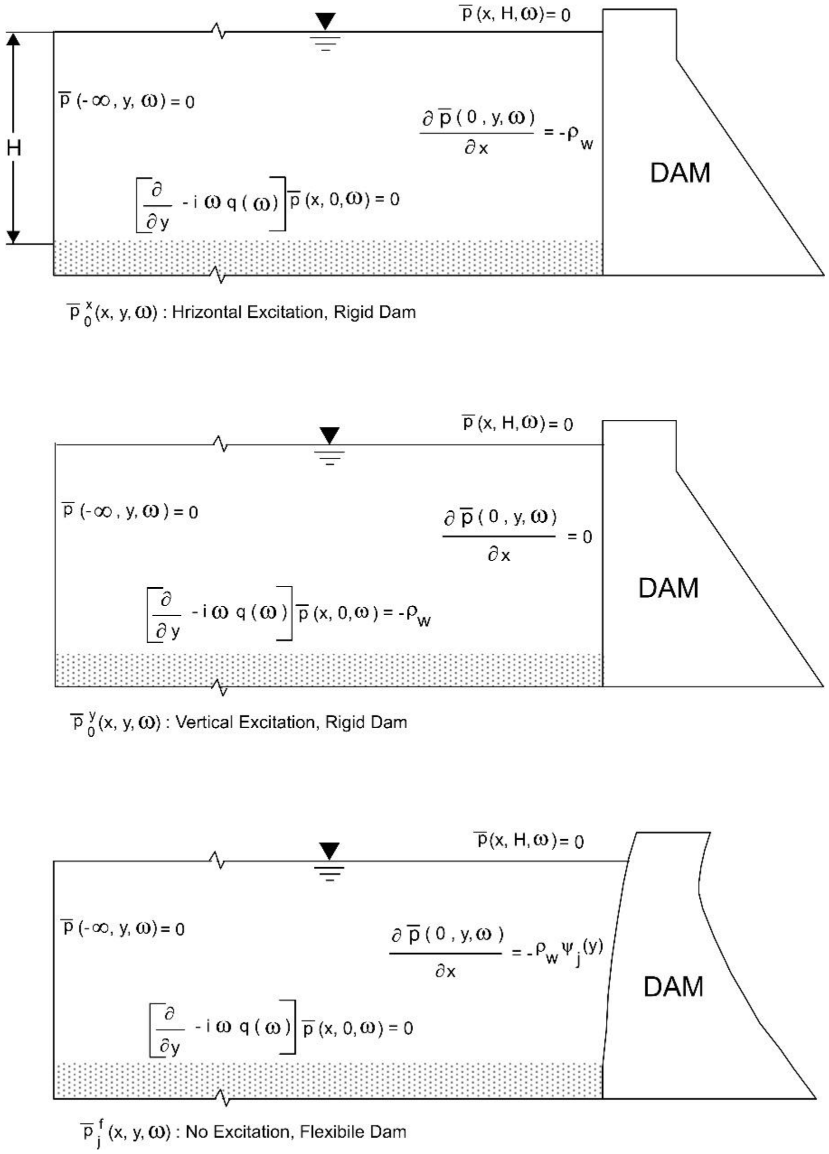

Fig. 7 Illustration of the boundary conditions used for the estimation of three different contributions to the hydrodynamic forces

The pressure waves impinging on the reservoir bottom are partially absorbed by the layer of sediments as well as the underlying foundation rock. These absorption effects are accounted by introducing a damping coefficient $q(\omega)$ in the boundary conditions at the reservoir bottom, which is defined in terms of the wave reflection coefficient $\alpha(\omega)$ as

$$q(\omega)C_w = \frac{1-\alpha(\omega)}{1+\alpha(\omega)} \tag{13}$$

To investigate the effect of the reflection coefficient $\alpha(\omega)$ on the response of the dam in details, Gupta and Pattanur [30] have derived an expression for $\alpha(\omega)$ in terms of the properties of the sedimentary layer and the underlying foundation rock. Typical examples of the frequency-dependent reflection coefficient $\alpha(\omega)$ for different values of the depth, d_s, of sediments as a fraction of the depth, H, of water are given in Figure 8(a); whereas the results for different values of the ratio of the elastic modulus, E_f, of the foundation rock to the modulus, E_s, of the sedimentary materials are given in Figure 8(b).

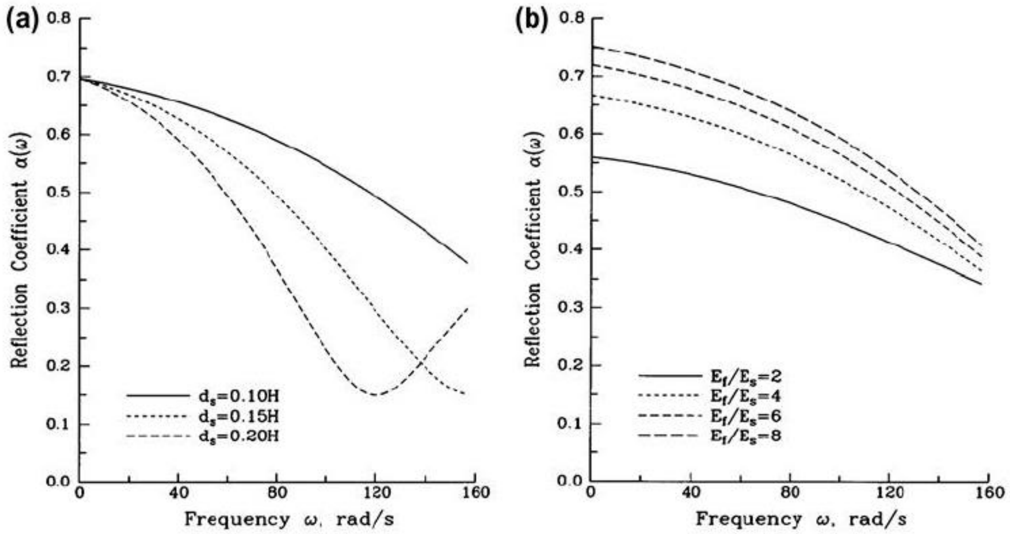

Fig. 8 Dependence of the wave reflection coefficient $\alpha(\omega)$ on: (a) the thickness of the sedimentary layer at reservoir bottom; and (b) the modulus of elasticity of the rock below

From the results in Figure 8(a), it is seen that the reservoir bottom sediments cause the reflection coefficient to decrease with increase in frequency, because the high-frequency waves get attenuated and absorbed faster in the sediments. As the depth of sediments increases, the reflection coefficient is seen to decrease faster. Also, the reflection coefficient is seen to be characterized by a valley at around 120 rad/s for the case of d_s / H = 0.20. This is due to the resonance of the sedimentary layer around this frequency, in which case it absorbs more of the wave energy. From the results in Figure 8(b), the reflection coefficient is seen to increase significantly with increase in the stiffness of the foundation rock. Foundation rock with higher stiffness causes stronger reflections of the wave energy at the sediments and foundation rock interface. From a comprehensive investigation of the effect of the reservoir bottom absorption on the dynamic response of the dam, Gupta and Pattanur [31] have illustrated that the stresses in the dam depend mainly on the modulus of elasticity of the rock at the reservoir bottom and the effect of the sedimentary layer may be neglected. Thus, the use of the following frequency-independent wave reflection constant can be considered sufficient in practical applications

$$\alpha = \frac{1-\beta_{wf}}{1+\beta_{wf}} \tag{14}$$

where $\beta_{wf} = \rho_w C_w / \rho_f C_f$ is the impedance ratio of water to the foundation rock.

3.3 Analysis of Foundation Substructure

The stiffness matrix $S_f(\omega)$ connecting the interaction forces with the deformations of the base nodes is obtained using the flexibility influence coefficients F_m^{ij}, which represent the base displacement in direction *i* at nodal point m due to uniformly distributed harmonic element force of unit amplitude in direction *j* (*i*, *j* = 1 for *x*-direction and 2 for *y*-direction), as illustrated in Figure 9. The values of F_m^{ij} depend on the Poisson's ratio υ, the hysteretic damping coefficient η_f, and the shear wave velocity C_s in the foundation rock. These are computed by solving the wave equation for the foundation domain assumed to be a homogeneous, isotropic, linearly viscoelastic half-space with two stress boundary conditions prescribed at the free surface. The first boundary condition of uniformly distributed unit harmonic stress in the *y*-direction gives the compliance functions F_m^{22} and F_m^{21}; whereas the second boundary condition of unit harmonic stress in the *x*-direction gives the compliance functions F_m^{11} and F_m^{12}. The details on computation of the influence coefficients F_m^{ij} and how to use them for construction of the foundation stiffness matrix $S_f(\omega)$ are given in Dasgupta and Chopra [32].

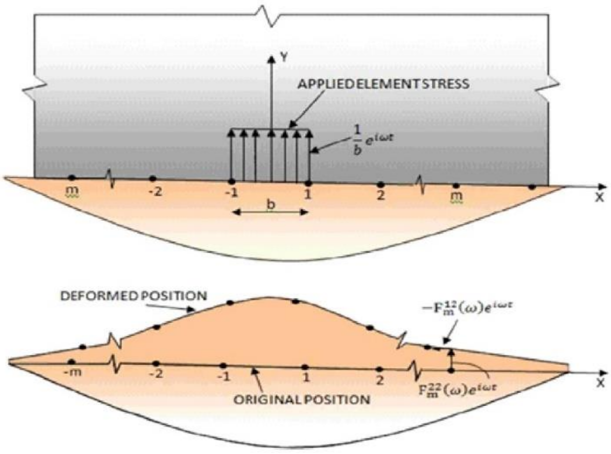

Fig. 9 Schematic illustration of the definition of flexibility influence (or compliance) coefficients F_m^{22} and F_m^{12} for vertically applied element load

3.4 Illustrative Numerical Results

To illustrate that the substructure approach is able to account for the effects of dam-foundation rock and dam-reservoir water interaction in a realistic way, example results are again computed for the tallest non-overflow monolith (M-17) of the Koyna dam. Two dimensional finite element idealization of the dam section along with the average values of Young's modulus of elasticity E_d, density ρ_d, Poisson's ratio ν_d, and hysteretic damping factor η_d for the dam and the corresponding values E_f, ρ_f, ν_f, and η_f for the foundation rock are given in Figure 10. The water is assumed to be compressible with velocity of pressure waves C_w = 1438 m/s and density ρ_w = 1.0 t/m^3. Figure 11 shows the longitudinal horizontal and vertical components of the Koyna accelerogram of the 1967 earthquake used as input excitation.

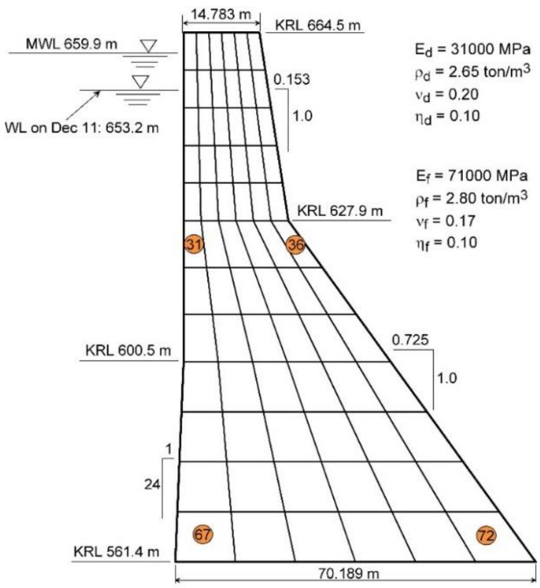

Fig. 10 FE idealization of the tallest non-overflow monolith of the Koyna dam used to compute the illustrative results by substructure method

To illustrate the effect of the dam - foundation rock interaction on the response, maximum principal stresses are computed for three values of the modulus of foundation rock as 310,000 MPa (termed as rigid foundation), 71,000 MPa and 31,000 MPa with empty reservoir condition. These three values of the foundation modulus are selected to have increasing effect of the dam-foundation interaction on the response of the dam. Similarly, to illustrate the effect of the dam - reservoir water interaction, stresses are computed for three cases of reservoir water level as 25 m, 50 m and at MWL (termed as reservoir full) with modulus of the foundation rock fixed at 71,000 MPa. These three cases are also selected to have increasing effect of the dam-reservoir interaction on the response of the dam.

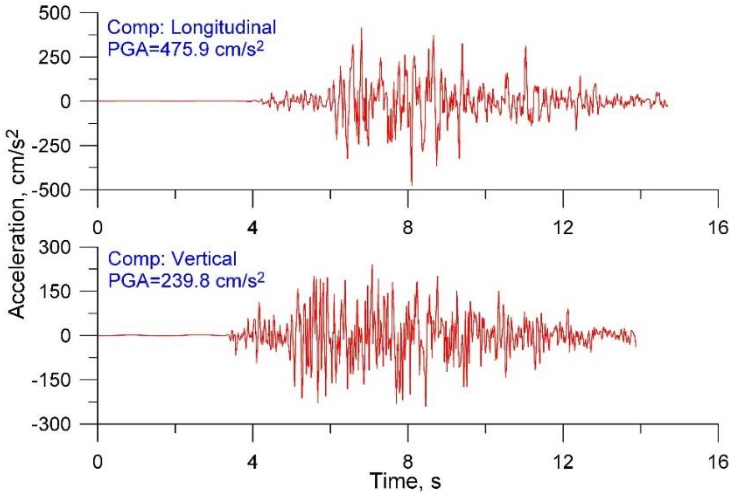

Fig. 11 Longitudinal and vertical components of the accelerograms recorded at the foundation gallery of the Koyna dam from the earthquake of 11 December 1967 with magnitude 6.5

Figure 12 shows the distribution of the highest values of the maximum principal stresses in the dam monolith for the three values of the modulus of the foundation rock. Lower values of the modulus indicates more flexible foundation, which causes the natural period and the damping ratio of the dam-foundation rock system to increase compared to the case of rigid foundation. Depending upon the distribution of the energy of input earthquake excitation among various frequencies, the increase in natural period may result in an increase or a decrease in the response. But the increase in effective damping always results in a decrease in the response, which mostly dominates. Thus the net effect of dam-foundation rock interaction is to reduce the maximum principal stresses in the dam, as indicated by decrease in the area enclosed by the highest stress contours and broadening of the area enclosed by the lowest stress contour from Case-1 to 3 in Figure 12.

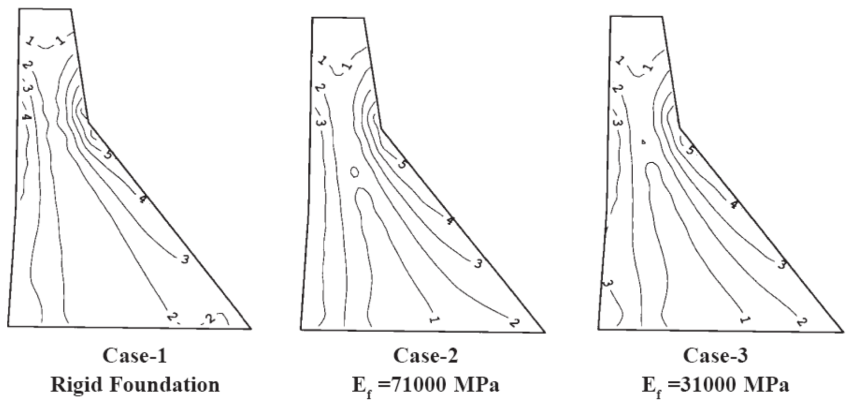

Fig. 12 Example results on the maximum principal stresses obtained by substructure method to illustrate the dam-foundation rock interaction effects

Figure 13 shows the distribution of the highest values of the maximum principal stresses in the dam monolith for the three values of the depth of water with a constant value of the elastic modulus of foundation rock as 71,000 MPa, so that the variation in the response can be attributed solely to the dam-water interaction effects. The interaction between the dam and the impounded water also lengthens the natural period and enhances slightly the damping of the dam-reservoir system. Increase in natural period may have increasing or decreasing effect on the response, whereas the increase in damping will slightly decrease the response. However, dam-reservoir interaction also generates significant hydrodynamic forces on the upstream face of the dam, which generally overcomes the decrease in response due to the enhanced damping value. The results for the three water levels, termed as Case-4 to 6, indicate increase in stresses with increase in the water level of the reservoir. The contours for higher stress values are seen to enclose larger areas as one goes from Case-4 to 6, indicating that tensile stresses exceed the value corresponding to that contour over a larger portion of the monolith because of dam-water interaction effects.

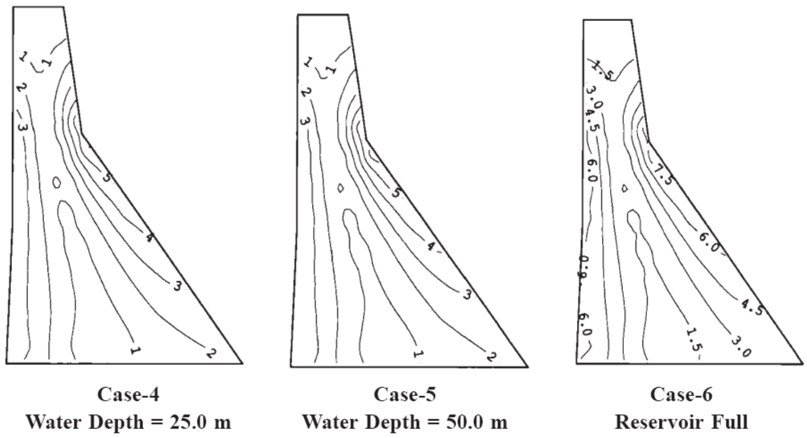

Fig. 13 Example results on the maximum principal stresses obtained by substructure method to illustrate the dam-reservoir water interaction effects

EFFECTS OF THE STOCHASTIC NATURE OF GROUND MOTION

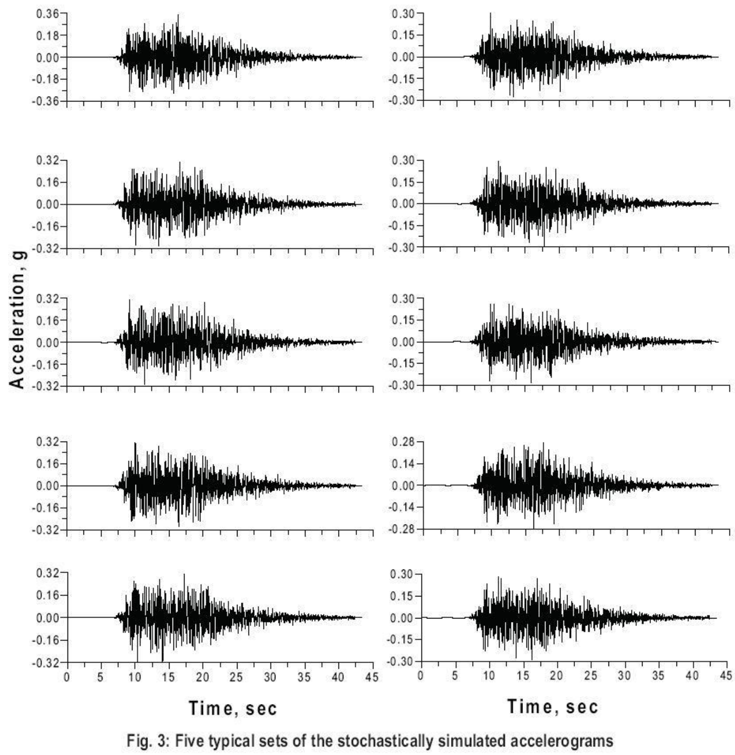

Fig. 14 Examples of typical accelerograms of horizontal (left side plots) and vertical (right side plots) compatible to a given set of horizontal and vertical target response spectra

Due to stochastic nature of earthquake ground motion, no two acceleration time histories are expected to be identical even if they are recorded for the same source, path and site parameters. It is necessary that the corresponding variability in the computed response is quantified and accounted realistically in the safety evaluation of gravity dams. To account for this variability, various codes and guidelines recommend using the maximum value of the responses for 3-5 simulated accelerograms [33]. Design accelerograms are commonly obtained to be compatible with a target response spectrum [34] obtained by deterministic and probabilistic seismic hazard analysis methods [10, 35, 36]. Figure 14 gives the typical examples of the accelerograms generated from the same set of the horizontal and vertical response spectra. In reality, it is possible to generate a very large ensemble of such accelerograms, and thus the use of a limited number of accelerograms cannot be considered adequate to capture the effect of the stochastic nature on the computed response. On the other hand, the computation of the response for very large number of accelerograms is inconvenient in practical design applications [37].

To obviate the need of computing the response for a large number of accelerograms, Gupta [38] and Gupta and Joshi [39] have utilized the substructure formulation to propose a statistical method based on the random vibration theory. In this method, expressions of Equations (9) and (11) for the complex Fourier transform of the displacement and stress responses have been used to define the power spectral density functions (PSDFs) of the corresponding response quantities in terms of the PSDFs of the *x*- and *y*-components of ground motion. The PSDFs of ground motion are obtained from the target design response spectra using a method due to Gupta and Trifunac [40]. Using the probability distributions developed by Gupta and Trifunac [41, 42] to obtain the statistical estimates of various order of peaks when all the peaks of a stationary stochastic process are arranged in decreasing amplitudes, the PSDF of a response quantity of the dam can be used to estimate the expected (mean), most probable (mode), or with any desired confidence level the amplitudes of several largest amplitude peaks of the response. The parameters involved in the probability distribution of the ordered peaks are defined in terms of the first few moments of the PSDF of the response. As the dam-foundation and dam-reservoir interaction effects are already included into the sub-structure formulation, the same are automatically represented in the proposed statistical method also.

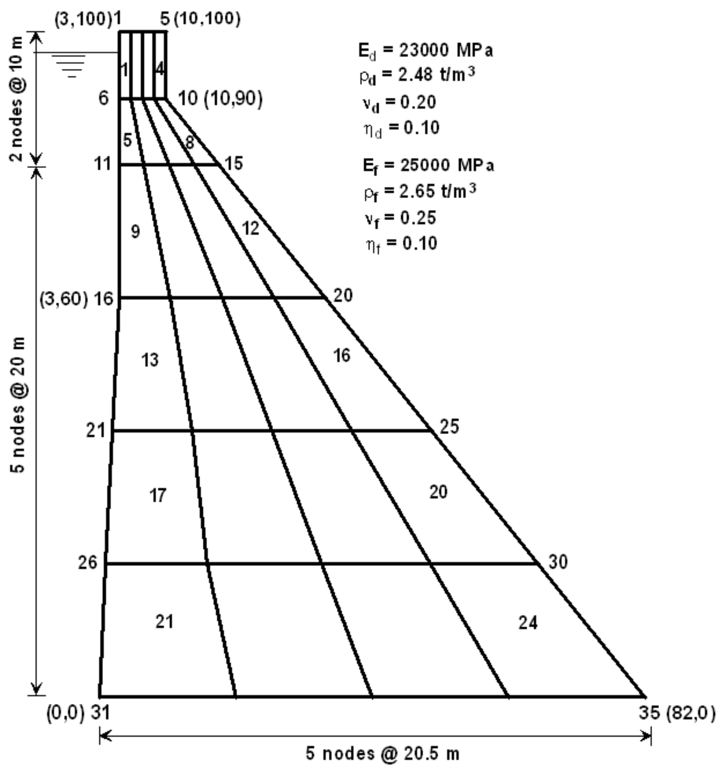

Fig. 15 FE idealization of the typical dam section used to compute the example results for illustrating the effects of stochastic nature of earthquake excitation

To illustrate the effect of the stochastic viability in the ground motion on the response, the statistical method has been used to compute the numerical results for a typical dam section with the finite element idealization shown in Figure 15. The height and base width of the dam are taken as 100 m and 82 m, respectively. The properties of the dam material and the foundation rock used are also indicated in Figure 15. To represent a wide variety of input excitations, the example results are computed for four different sets of horizontal and vertical acceleration records of real earthquakes as shown in Figure 16, along with the details of the contributing earthquakes and the name of the recording site. The response of the dam is computed in terms of the expected (mean), most probable (mode), and with confidence levels of P = 0.50, 0.05 and 0.95 of the first ten highest peaks of the maximum (tensile) and minimum (compressive) principal stresses, σ_{max} and σ_{min}, at the centroids of all the 24 elements.

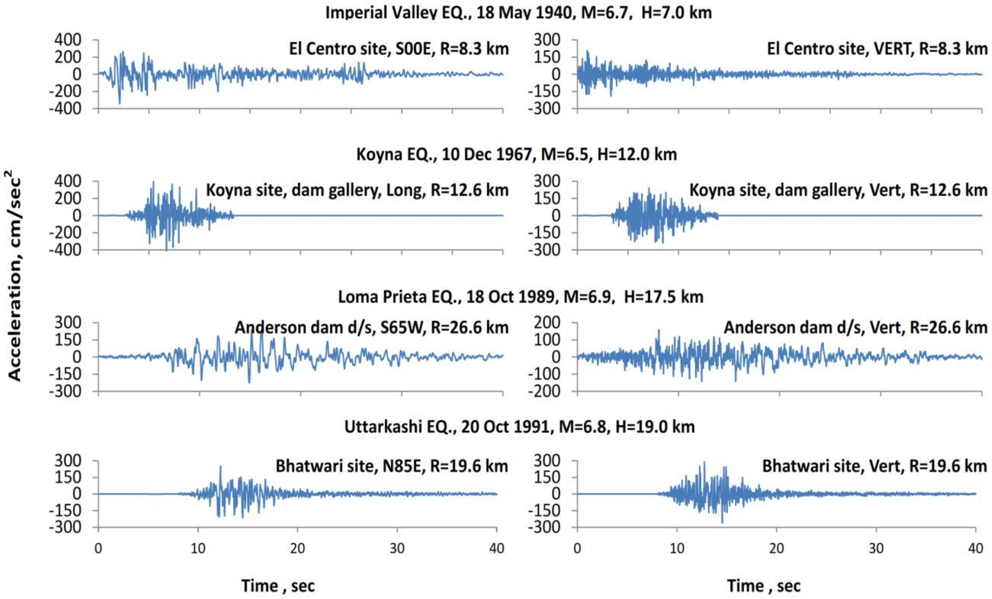

Fig. 16 Four sets of horizontal and vertical components of accelerograms used to compute the example results for illustrating the effects of stochastic nature of earthquake excitation

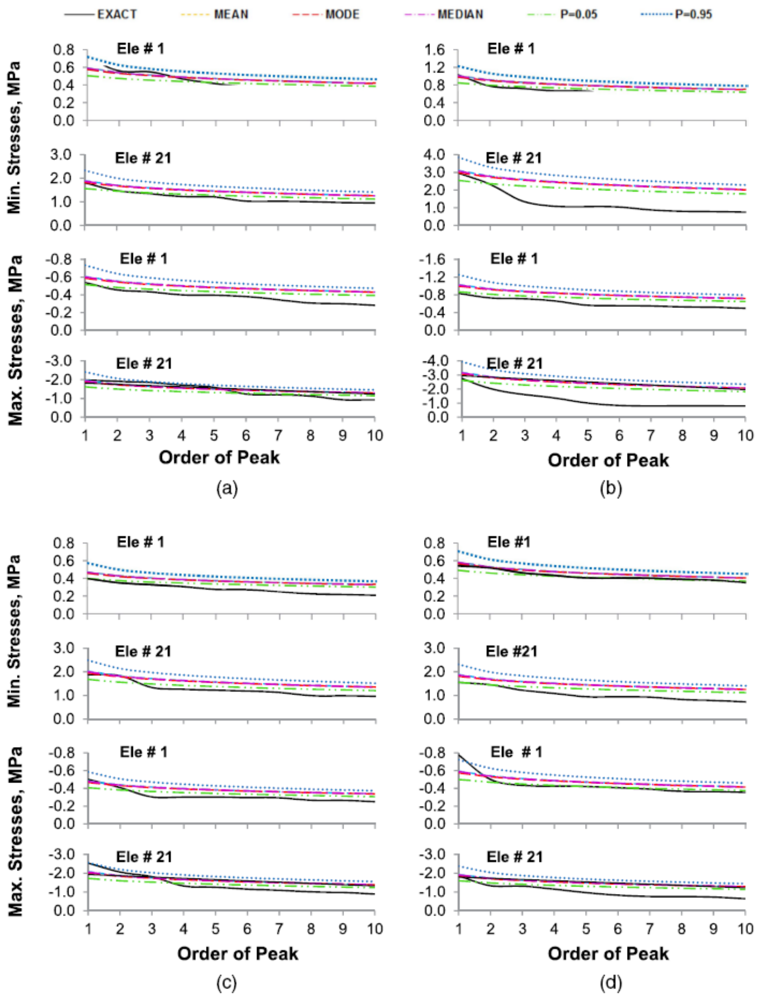

Fig. 17 Comparison of the statistical estimates with the exact time-history results for the peak amplitudes of principal stresses in two selected elements of the typical dam section of Figure 15 with input excitations as the accelerograms of (a) Imperial Valley earthquake; (b) Koyna earthquake; (c) Loma Prieta earthquake; and (d) Uttarkashi earthquake as given in Figure 16

Figure 17 shows the comparisons between the statistical estimates and the exact time-history results obtained from the substructure method described in the previous section for the principal stresses in Element no. 1 (top upstream) and Element no. 21 (heel) for all the four input excitations. The expected, most probable, and median ($P = 0.50$) estimates of the peak amplitudes of the principal stresses are all seen to be very close to each other and any one of them can be used to describe the central trend. Further, all these are seen to be in close agreement or slightly conservative compared to the exact time-history results for the highest 10 peaks for different input excitations. The exact time-history results for both σ_{max} and σ_{min} are seen to lie well within the 5 % to 95 % confidence interval for the highest 2-3 peaks. Thus, assuming the expected peak amplitudes to represent the central trend, the amplitudes for higher confidence levels can be used to quantify the effect of the stochastic variability of ground motion on the response.

To get an idea about the magnitude of the stochastic effects on the principal stresses, Table 1 lists the percentage increase in the mean values of the first-order (highest) peak to the values with 0.95 confidence level. The increase is very significant in the range of 21 to 24 %. As the stochastic variability is over and above the effects of the aleatory and epistemic uncertainties accounted in the estimation of the design response spectra, a confidence level of 0.84 (representing mean plus sigma estimate) may be considered adequate to quantify the stochastic variability in the response of gravity dams. The 84th percentile estimates are found to be higher than the mean estimates by about 15 %.

Table 1: Percentage increase in the mean estimates of principal stresses to get the estimates with confidence level of 0.95 for input excitations as the accelerograms of (a) Imperial Valley earthquake; (b) Koyna earthquake; (c) Loma Prieta earthquake; and (d) Uttarkashi earthquake

Input	**Mean Estimate in MPa**		***P* = 0.95 Estimate in MPa**		**% Stochastic Variability**	
	σ_{max}	σ_{min}	σ_{max}	σ_{min}	σ_{max}	σ_{min}
Element no. 1						
(a)	0.59208	0.60563	0.71676	0.73314	21.01	21.05
(b)	1.00681	1.02943	1.22451	1.25199	21.62	21.62
(c)	0.47119	0.48213	0.57219	0.58547	21.44	21.43
(d)	0.58121	0.59428	0.70641	0.72228	21.54	21.54
Element no. 21						
(a)	1.89587	1.95066	2.33226	2.40059	23.02	23.66
(b)	3.11108	3.18085	3.84664	3.93324	23.64	23.65
(c)	2.02774	2.08012	2.49172	2.55659	22.88	22.91
(d)	1.88376	1.92775	2.31504	2.36962	22.89	22.92

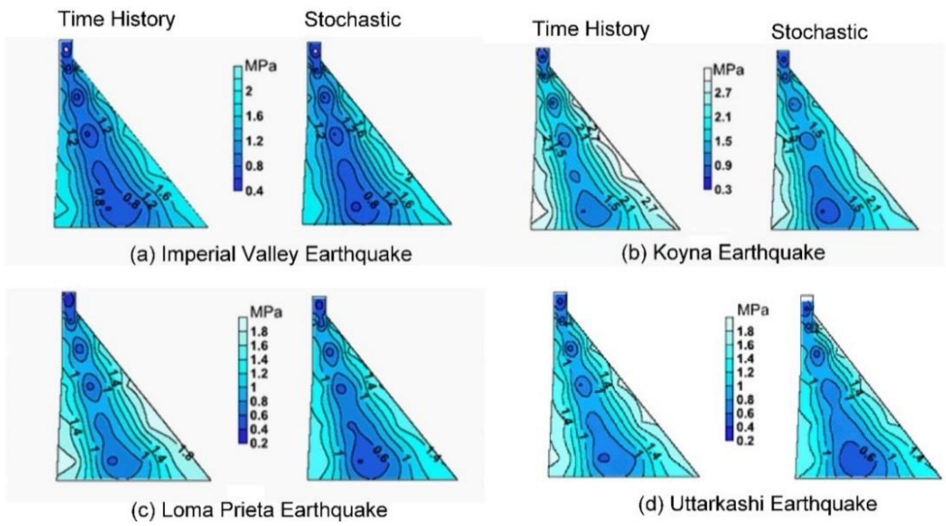

Fig. 18 Comparison of the maximum principal stress contours based on statistical expected values of the highest peak with those based on the highest peak of the exact time history results with input excitations as the accelerograms of (a) Imperial Valley earthquake; (b) Koyna earthquake; (c) Loma Prieta earthquake; and (d) Uttarkashi earthquake

Finally, to illustrate that the response spectrum based statistical method [39] is able to accurately predict the distribution of principal stresses throughout the dam section, Figure 18 compares the contours of the statistical expected values of the highest peak of the maximum principal stresses in all the elements with the contours of the highest peaks of the exact time-history solutions. The distribution of the stress values in both the cases is seen to be very similar and in good numerical agreement for all the four input excitations with widely varying characteristics. Thus the conclusion made from the results in Figure 17 regarding the ability of the statistical method to predict accurately the amplitudes of several orders of highest amplitude peaks of principal stresses is validated for the entire dam cross section also.

MATERIAL PROPERTIES

In addition to the rigorousness of the method of analysis, the reliability of the results on dynamic response analysis depends also on the material properties of the dam and the foundation rock. As the material properties generally show very wide scattering, it is necessary that the properties used in a particular case are defined with reasonable conservatism. Site specific material properties should be obtained by testing of core samples taken from a test fill placement for new dams and from different locations for existing dams. The test fill concrete should comprise the same type of fine and coarse aggregates as to be used in the actual construction and be cured for 90 days. Core samples of the foundation rock from the actual site have to be tested to obtain the properties of the foundation rock.

The primary material properties of interest for concrete are those that affect the prediction of the structural response and those that are required for the evaluation of performance. For the linear elastic analysis of interest in this paper, Young's modulus of elasticity, Poisson's ratio, and density are the basic material properties required for the computation of response. To assess the safety of the proposed design of a new dam or the design of an existing dam, the computed compressive and tensile stresses due to combined dynamic and static forces are required to be compared with the strength of the dam material. Thus accurate and realistic estimation of the compressive and tensile strengths also attains great importance. This section describes briefly the various tests needed to estimate the site-specific material properties for the final design and evaluation of concrete gravity dams. However, the preliminary design may be based on the approximate material properties defined using the general guidelines presented in the next subsection.

1 Approximate Material Properties for Preliminary Design

The properties of mass concrete increase with age and attain their ultimate values at the age of about one year. The modulus of elasticity for one year old dam concrete is reported by Jansen [43] to vary over a wide range of 28,000 to 48,000 MPa. As per the Indian code IS: 456-2000 [44], the elasticity modulus can be approximated by $E_s = 5000\sqrt{f_{c;s}}$ with $f_{c;s}$ as the static compressive strength in MPa. Similar relations between modulus of elasticity and compressive strength are proposed in many other building codes, such as American Concrete Institute code ACI 318 [45], British Code of Practice CP 110 [46], and Eurocode 2 [47]. A significant feature of the modulus of elasticity is that it may be higher under rapidly varying earthquake loading compared to its static value. Raphael [48] and Canadian Electrical Association [49] have reported 33 % and 25 % increase, respectively. However, testing of 103 cores samples from 10 different dams by U.S. Bureau of Reclamation has indicated the ratio of the dynamic to the static moduli of elasticity to vary widely from 0.70 to 1.10 with a mean value of 0.89 [50]. Thus, a higher value for the modulus of elasticity under dynamic condition cannot be used with much confidence. It is recommended that the static modulus, E_s, obtained from the compressive strength using the correlation given in the code IS: 456-2000 [44] may be used for the preliminary design.

The Poisson's ratio of the concrete may also vary over a wide range of 0.10 to 0.30. The German Standard DIN 4227 [51] relates the Poisson's ratio to the static elasticity modulus as: 0.15-0.18 for 24,000 MPa, 0.17-0.20 for 30,000 MPa, and 0.20-0.25 for 35,000 MPa. The American Concrete Institute code ACI 207.1R-05 [52] has reported the modulus of elasticity and the corresponding Poisson's ratio for several concrete dams in USA and Brazil, which are in good conformity with the German Specifications. The Poisson's ratio may also increase under dynamic loading. Harris [53] has reported an average value of 1.07 for the ratio of dynamic to static Poisson's ratio from the testing of 16 dam core samples, whereas Mohorovic et al. [50] have report a wide range of values from 0.69 to 1.69 with mean value of 1.09. In view of such large scattering in the estimates, it is proposed that no dynamic enhancement be used in the Poisson's ratio in dynamic response analyses [54]. The static Poisson's ratio, ν_s, can be obtained approximately from the elastic modulus using the German Specifications DIN 4227 [51].

The density of concrete is used in defining the mass matrix in the dynamic equations of motion. Higher values of the density would induce more vigorous dynamic response by increase in the inertial forces. It is thus necessary that the density is defined accurately. The density of concrete can vary between 2.3 t/m^3 and 2.5 t/m^3, and 2.4 t/m^3 may be considered a good representative value in the absence of the exact value [54].

The foundation rock is a highly inhomogeneous medium and the properties of the same type of rock may vary widely due to variations in the in situ condition. For preliminary design of gravity dams, the modulus of elasticity, Poisson's ratio, and density of the foundation rock may be taken from other nearby dam site with similar type of foundation rock. If such information is lacking, the properties summarized in Bradon [21] for 43 projects investigated by US Bureau of Reclamation the world over for different rock types may provide a good basis for selecting the properties of the foundation rock.

The safety of gravity dams is assessed by comparing the compressive and tensile stresses computed by linear elastic analysis with the compressive and tensile strengths of the concrete, respectively. The key property that controls the performance of a dam is its tensile strength, because the tensile stresses under actual ground excitations are generally comparable to the compressive stresses, whereas the tensile strength is much smaller. Several investigators have correlated the static tensile strength $f_{t;s}$ in MPa to the static compressive strength $f_{c;s}$ in MPa as given in Table 2. Tensile strength can be determined from three different types of tests, viz.; direct tension, flexural tension and splitting tension tests. As these tests give quite different values, the type of the tensile strength to be used depends on the anticipated mode of failure. For example, the direct tensile strength can be considered more appropriate to assess the safety against the earthquake induced stresses. Thus, the correlation for the direct tensile strength by Raphael [56] in Table 2 can be used to define the static tensile strength for the preliminary stage design, where the static compressive strength may be approximate by the intended design strength.

Table 2: Some selected relations between tensile and compressive strengths of concrete

Sr. No.	Type of Test	Type of Specimen	Factor c in $f_{t;s} = cf_{c;s}^{2/3}$ MPa	Reference
1.	Flexural	Prisms 10 × 10 × 40 cm	0.399	Khan et al. [55]
2.	Flexural	Cylinders 15 × 30 cm	0.441	Raphael [56]
3.	Splitting	Cylinders 15 × 30 cm	0.232	Hellmann [57]
4.	Pure tension	Cylinders 15 × 30 cm and 15 × 30 cm	0.325	Raphael [56]
5.	Pure tension	Prisms 5 × 20 × 20 cm and 9 ×1 5 × 60 cm	0.297	Kupfer and Gerstle [58]

Similar to the modulus of elasticity and the Poisson's ratio, the strength of the mass concrete also attains its maximum value at the age of about one year. Also, both the compressive and tensile strengths increase under dynamic loading. Mohorovic et al. [50] have reported the ratio of dynamic to static compressive strength to vary over a wide range of 0.73 to 1.45 with mean value of 1.07, whereas they have indicated the ratio for split tensile strength to vary from 0.98 to 1.73 with mean value of 1.44. From a rigorous study with some 12,000 published test results, Raphael [56] has proposed a magnification factor of 1.5 for the dynamic tensile strength, which is in good agreement with the mean factor of 1.44 by Mohorovic et al. [50]. Thus, no dynamic enhancement is recommended in the static compressive strength, but 50 % dynamic enhancement in the static tensile strength has received wide recognition [59-61].

2 Estimating Exact Material Properties by Testing of Samples

For reliable estimation of stresses in a gravity dam by rigorous time-history response analysis, it is necessary to use the exact material properties determined by testing of core samples taken from different locations in case of an existing dam or from a test fill placement in case of a new dam. The static values of the Young's modulus of elasticity (E_s), Poisson's ratio (ν_s) and compressive strength $f_{c;s}$ of the dam concrete can be obtained as per the procedures given in Indian standard IS: 516-1959 [75] or American standard ASTM C 39/C 39M-04a [62]. The core diameter is required to be about three times the maximum nominal size of the coarse aggregate and the length has to be at least twice the diameter. To determine the compressive strength, the core sample is loaded axially at a slow rate of approximately 140 kg/cm^2/min on a universal testing machine (UTM) or a compression testing machine (CTM) until the resistance to the increasing load breaks down. The Young's modulus is determined from the slope of the stress-strain plot by accurately measuring the axial deformations by fitting an extensometer to the specimen for each load step up to a maximum of 1/3 of the design compressive strength. The lateral deformation has to be also measured for each load step by fitting another extensometer or a combination extensometer for determination of the Poisson's ratio, which is defined as the ratio of the lateral to the longitudinal strain. Typical laboratory setup for the estimation of compressive strength is shown in Figure 19(a) and that for the estimation of the modulus of elasticity and Poisson's ratio is shown in Figure 19(b).

As mentioned before, the tensile strength can be determined by three different methods, leading to widely differing estimates for the same type of concrete. The splitting tension test is the easiest to perform, in which a core sample is loaded till failure along a vertical plane passing through a pre-marked diameter on the circular cross section at a nominal rate within the range of 1.2 MPa/min to 2.4 MPa/min as shown in Figure 19(c). As per Indian code IS: 5816-1999 [63] and American Standard ASTM C 496/C 496M-04 [64], the split tensile strength (f_{sp}) is given by $2P/\pi Ld$, where P is the failure load, and L and d are the length and diameter of the core sample, respectively. The direct tension test can be performed using the procedures given in USBR [65] and CRD-C164-92 [66] by subjecting a cylindrical core sample to pure tension on a UTM as shown in Figure 19(d) [67]. The flexural strength (modulus of rupture) is another measure of the tensile strength, which can be estimated conveniently by loading the concrete prisms of size 15 × 15 × 70 cm till failure in bending as per the procedure given in IS: 516-1959 [75] or ASTM C78-02 [68]. The major limitation of this test is that it can be performed only on the concrete prisms cast in the laboratory.

Fig. 19 Typical test setups for determining the (a) compressive strength; (b) Young's modulus and Poisson's ratio; (c) split tensile strength; and (d) direct tensile strength of dam concrete

The core samples of the foundation rock can also be tested in an identical manner to the concrete samples for determining the Young's modulus, Poisson's ratio, unconfined compressive strength, and split tensile strength as per the procedures given in the Indian codes IS: 9221 [69] and IS: 10082 [70] or the American standards ASTM D7012-14e1 [71] and ASTM D3967-16 [72]. The foundation rock in situ is characterized by joints, discontinuities, and weak planes, the effects of which is not accounted in the properties obtained by testing of small size cores in the laboratory. The modulus of elasticity and the strength values obtained by testing of core samples may have to be thus reduced and the Poisson's ratio enhanced judiciously to arrive at more realistic estimates for use in the dynamic response analyses of gravity dams. The density values of the concrete and the foundation rock can be obtained simply as the mass per unit volume by taking the weight of the core and calculating its volume from the dimensions.

Under the high-frequency dynamic loading during earthquakes, the various properties of the concrete and the foundation rock are expected to be higher than their static estimates. However, until confirmed by the experimental results, no dynamic enhancement is recommended in the modulus of elasticity and the compressive strength values. The experimental facilities for estimation of dynamic tensile strength are generally lacking, but a dynamic increase of 50 % in the static tensile strength is acceptable to get the dynamic tensile strength of the concrete.

PERFORMANCE CRITERIA

The seismic performance of gravity dams is governed mainly by the tensile strength of the concrete. The tensile stresses in the traditional pseudo-static method are limited within 1 to 4 % of the design compressive strength, which is satisfied by using unrealistically small lateral earthquake forces compared to the actual forces. Realistic performance evaluation of gravity dams has to be based on the tensile stresses due to design basis earthquake (DBE) and maximum credible earthquake (MCE) plus the initial stresses due to various static loads. A dam is required to withstand DBE with little or no damage and remain within elastic range, whereas the dam should be capable of surviving the MCE without a catastrophic failure that may result in uncontrolled release of water. Considerable damage due to inelastic deformations may occur during MCE, but the performance in common design applications is required to be evaluated from the results of a linear elastic analysis only. This requires formulating appropriate performance criteria vis-à-vis the tensile strength of the concrete.

The safety evaluation by linear elastic analysis is based on the concept of the apparent tensile strength of concrete, which is the strength that would be realized if the stress-strain curve would have been linear up to the failure strain. Figure 20 gives four curves for the tensile strength as a function of compressive strength. The curves from bottom to top represent the actual static, apparent static, actual dynamic and apparent dynamic strengths, respectively. The static tensile strength, $f_{t;s}$, is defined from the static compressive strength, $f_{c;s}$, by the relation $f_{t;s} = 0.325 f_{c;s}^{2/3}$, and the dynamic tensile strength, $f_{t;d}$, is taken 50 % higher as mentioned before. Experimental evidences support an apparent static tensile strength, $f_{t;sa}$, of about 30 % higher than the true value, which is the basis for the apparent static tensile strength curve in Figure 20. Raphael [56] has proposed to use 30 % increase to define the apparent dynamic tensile strength also. However, Bhattacharjee and Leger [73] have proposed the apparent dynamic tensile strength, $f_{t;da}$, to be only 8 % higher, because some investigators [59, 74] have reported the near peak stress-strain non-linearity to decrease substantially under dynamic loading.

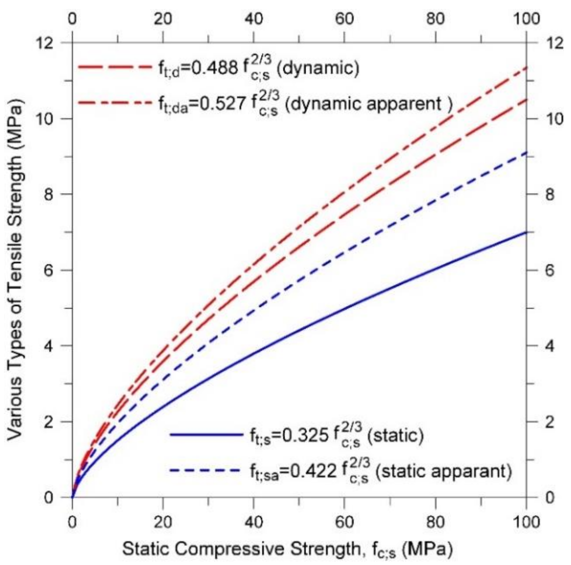

Fig. 20 Various types of tensile strength of dam concrete as a function of the static compressive strength (modified after Raphael [56])

The seismic demands imposed on a gravity dam by DBE and MCE ground motions may first be determined by simplified pseudo-dynamic response spectrum analysis. The total dynamic plus static stresses should be used for evaluation of performance, which requires the tensile stresses to be within the actual dynamic tensile strength of the concrete during DBE and within the apparent dynamic tensile strength during MCE. No rigorous time-history analysis is necessary, if both these criteria are satisfied. Rigorous time-history analysis should be carried if the pseudo-dynamic method indicates unsafe design. The performance under DBE requires that the highest values of the total dynamic plus static values of the maximum principal stresses over the complete cross section of the dam are in general less than the actual dynamic tensile strength. However, single excursions beyond the tensile strength may be permitted at isolated locations near heel and the downstream face where abrupt change in slope takes place.

The results of the linear elastic time-history analysis under MCE may be used to evaluate the performance using the guidelines given in the USACE Engineering Manual EM 1110-2-6051 [6], which are based on the demand to capacity ratio (DCR) and the associated cumulative duration. The DCR is defined as the ratio of the maximum principal stresses under combined static and dynamic loads to the static tensile strength obtained by testing of core samples or from static compressive strength and the results in Figure 20. The cumulative duration represents the total duration of stress excursions beyond a specified level of DCR, which can be obtained by multiplying the number of stress values exceeding that stress level by the time-step used in the time-history analysis. The dam will exhibit nonlinear response in the form of cracking of concrete and/ or opening of construction joints if the DCR values exceed 1.0, but the damage may be considered acceptable if the DCR values are less than 2.0 and limited to 15 percent of the dam cross-sectional area, and the cumulative duration of the stress excursions beyond the tensile strength of the concrete falls below the performance curve given in Figure 21.

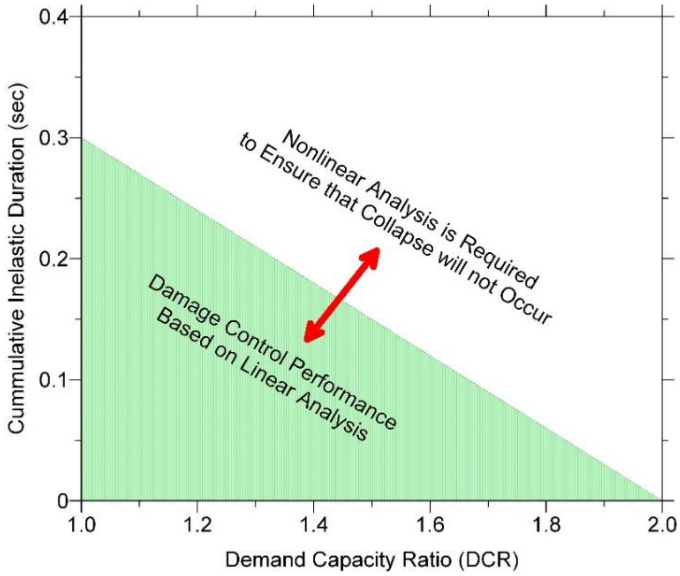

Fig. 21 Performance curve for concrete gravity dams analyzed by linear elastic time-history method (after [6])

To arrive at an appropriate decision on whether the nonlinear time-history analysis should be carried to get more accurate idea about the damage, further examination and interpretation of the results of the linear time-history analyses can be performed as follows:

(i) ***Natural Periods:*** Depending on whether the natural periods of the lowest few modes of the dam fall on the ascending or descending part of the design response spectrum, it should be possible to infer that the lengthening of the periods of vibration due to nonlinear behavior would increase or decrease the seismic demand. A decrease in the seismic demand indicates that the nonlinear response would improve the situation.

(ii) ***Displacement Histories:*** Time-histories of the nodal displacements at critical locations such as the crest should be plotted and examined to ensure that the displacements are not large enough to endanger the overall stability of the dam.

(iii) ***Distribution of Stresses:*** Vector plots of the maximum (tensile) and minimum (compressive) principal stresses due to static plus dynamic loads should be examined for the dam cross section. This will provide an idea about overstressed zones of the dam and the magnitudes and directions of the principal stresses over these zones, which can be interpreted to predict the probable directions of the tensile cracking.

(iv) ***Concurrent Principal Stresses:*** The largest tensile and compressive stresses in (iii) above generally occur at different times during the earthquake excitation. Therefore, vector plots should also be presented for the concurrent principal stresses for the time steps at which the critical principal stresses reach their highest values. This will provide a more realistic idea about the extent, location, and direction of the probable tensile cracking.

A nonlinear analysis under MCE condition is recommended only if the interpretations of the results of the linear elastic time-history analysis indicate extensive tensile cracking that is judged to impair the ability to quickly and safely draw down the reservoir. However, before embarking upon the nonlinear analysis, the feasibility of modifying the design to satisfy the performance criteria for linear analysis should be explored in case of new dams, which may include increasing the slope of the downstream face, increasing the thickness of the cross section, and/ or increasing the strength of concrete. In case of existing dams, a retrofit scheme should be designed so that the retrofitted dam meets the performance criteria of the linear elastic analysis.

ADDITIONAL ISSUES

Three additional issues which are beyond the scope of detailed description in this paper may be mentioned as the material damping for the dam and the foundation, three-dimensional response analysis, and the nonlinear dynamic analysis. An overview these issues is given below.

1. Material Damping for the Dam and the Foundation

The numerical models for the dynamic response analysis of gravity dams need to specify explicitly the material damping for the dam and the foundation rock, whereas the effects of the radiation damping in the foundation and the energy loss at the reservoir boundaries are introduced inherently in the solution through the boundary conditions used. The experimentally determined damping values from forced vibration tests, measurement of ambient vibrations, or recording of low magnitude earthquakes represent the overall damping in the system, from which the individual components of damping cannot be separated out. On the other hand, the overall damping value of the dam-reservoir-foundation system in the numerical analysis cannot be estimated without completion of the solution. An overall viscous damping of 5-7 % can be considered a good choice for gravity dams, but what values of the material damping for the dam and the foundation rock have to be used in the numerical analysis to achieve a desired overall damping in this range cannot be decided straight away. Viscous damping ratio in the range of 1-3 % for the dam and hysteretic damping ratio in the range of 0.02-0.06 (corresponding to viscous damping ratio of 1-3 %) for the foundation rock may be used to achieve a specified overall viscous damping ratio in the range of 5-7 % by some trial and errors.

As the dynamic response amplitudes depend significantly on the damping values, it is necessary that extensive experimental estimates are obtained for the existing dams in India to specify the damping ratios for the new dams more accurately. Instrumenting major dams to measure their responses during future earthquakes will also be useful for estimating the damping value and calibrating the mathematical models.

2. Three Dimensional Dynamic Analysis

The two-dimensional linear elastic analysis by substructure method is based on several simplifying assumptions, which may be violated significantly in many cases. The foundation rock is assumed to be a homogeneous half-space, but it may in reality be characterized by large scale heterogeneities in the form of stratifications. The upstream face of the dam is assumed to be vertical and the reservoir bottom horizontal, which may also deviate significantly in many cases. Further, it is assumed that the individual dam monoliths do not have any component of vibration along the dam axis and the reservoir is wide enough to assume the motion of water to be the same for any vertical plane perpendicular to the axis of the dam. For dams in narrow canyons it is thus necessary to perform a three-dimensional dynamic analysis, but the substructure approach has not been developed to carry out the three-dimensional analyses. It thus becomes necessary to use a rigorous FE model for the purpose, which requires expertise to perform such analyses using commercial software.

3. Nonlinear Response Analysis

Although a judicious interpretation of the results of the linear elastic time-history analysis can be useful to identify the zones of a dam which may undergo damage due to inelastic behavior, a reliable estimation of the locations and extent of the damage is possible by a nonlinear analysis only. The high stresses predicted by the linear analyses do not take into account the redistribution of stresses when cracks form or when contraction joints open and close. But the nonlinear analysis presents a number of challenges in developing the numerical model, defining nonlinear constitutive behavior of the material, and dealing with the sensitivity of the results to uncertainty in ground motion and the material properties. Also, the performance criteria in terms of the permissible damage and amount of sliding at lift joints or at cracked interface are open to personal judgment [54]. The nonlinear analysis should be followed by a post-earthquake stability analysis of the damaged dam to ascertain its water retaining capability.

SUMMARY AND CONCLUSIONS

The gravity dams in India are commonly designed by pseudo-static seismic coefficient method, which is primarily concerned with the static stability against sliding under different combinations of various static loads and highly reduced seismic loads including the hydrodynamic forces. This method cannot be considered suitable to ensure the seismic safety of dams, because the design is not governed by the stresses responsible for structural damage. Earthquake resistant design of gravity dams has to be based on the dynamic stresses under DBE and MCE excitations plus the initial stresses due to various static loads. The dynamic analysis of gravity dams poses a very complex problem due to semi-unbounded nature of the reservoir and the foundation domains and the dam-water-foundation interaction effects. Realistic response estimation has to consider all these aspects rigorously, yet in a practically simple manner. This paper has described some of the crucial issues involved in the dynamic response analysis of gravity dams, which can be summarized as below:

(i) ***Dam-Foundation Rock Interaction:*** The vibration of a dam during strong earthquakes causes considerable dynamic deformations in the foundation rock at the base of the dam, due to which the effective earthquake motion at the base is modified compared to the free-field motion. Also, the stress waves travelling from the dam into the foundation are radiated to infinity in the unbounded foundation, causing additional damping into the dam-foundation system. For realistic and accurate estimation of the dynamic response of gravity dams, it is necessary that the method of analysis is able to account for these effects in a realistic manner.

(ii) ***Dam-Water Interaction:*** Pressure waves in the impounded water due to the vibrations of the dam and the reservoir bottom during an earthquake generate significant hydrodynamic forces on the upstream face of the dam. The common practice of approximating these forces by the inertial forces of an added mass of water to the upstream face of the dam is based on unrealistic assumptions, which may under or overestimate the stresses in the dam. For realistic and accurate estimation of the dynamic response of gravity dams, it is necessary that the method of analysis is able to model the hydrodynamic forces accurately by considering the effects of the water compressibility and absorption of pressure waves at the reservoir bottom.

(iii) ***Methods of Analyses:*** The methods for dynamic response analyses of gravity dams are required to model the dam-water-foundation interaction in a realistic way. The commonly used standard finite element (FE) method based on massless foundation block to model the dam-foundation interaction and the added mass formulation to model the dam-water interaction is based on physically unrealistic assumptions, which result in crossly unrealistic estimates of the dynamic stresses in the dam. On the other hand, the commercial FE codes cannot be used directly in a user friendly manner to solve the rigorous FE problem, in which the truncated boundaries of the foundation and the reservoir have to be modeled by viscous dampers and the effective earthquake forces have to be computed outside and applied at these boundaries. However, at least for the two-dimensional linear elastic behavior, the substructure approach provides a convenient method for the rigorous time-history analysis of gravity dams. This method can be implemented straight away using the special purpose computer program EAGD-84 developed by Fenves and Chopra [9]. Using the results of the substructure method, Fenves and Chopra [7] have also proposed a simplified empirical method, which can be used to obtain fairly realistic estimates of the peak values of the principal stresses on the upstream and downstream faces of a dam. This method, termed as pseudo-dynamic method, is proposed to be used for preliminary design of dams as a replacement of the conventional pseudo-static method.

(iv) ***Stochastic Nature of Earthquake Excitation:*** The earthquake excitation is characterized by significant stochastic uncertainties, which requires that the response of a dam be computed for a large number of spectrum compatible acceleration time-histories and the response values with a suitable higher percentile be used for the purpose of design. However, the effect of the stochastic uncertainties in ground motion on the response of dams can be quantified more conveniently by the response spectrum based statistical method due to Gupta and Joshi [39]. Typical results computed by this method indicate about 15 % increase in the mean estimates of the principal stresses to get the estimates with confidence level of 0.84.

(v) ***Material Properties:*** The dynamic response of gravity dams depends significantly on the Young's modulus of elasticity, Poisson's ratio, and the density of the dam and the foundation rock. Reliable estimation of stresses for final design of a dam requires using exact values of these material properties determined by testing of core samples. However, when experimentally determined properties are lacking, the preliminary design may be based on the approximate material properties decided from the guidelines provided in this paper. No dynamic increase is recommended in any of these material properties needed for computation of the dynamic response.

(vi) ***Performance Criteria:*** For satisfactory performance evaluation of gravity dams, the combined dynamic and static tensile stresses during DBE and MCE have to satisfy appropriate safety criteria. Minor damage that can be repaired easily and economically is permitted under DBE and significant cracking that would not lead to sudden and uncontrolled release of water is permitted under MCE. Single excursions of the stresses beyond the dynamic tensile strength at some isolated critical locations of the dam cross section are not considered unsafe for the DBE excitation. However, the safety evaluation under MCE excitation requires more systematic examination and interpretation of the results of the linear elastic analysis. By identifying the extent of the over-stressed zones at different point of time and by finding the number of stress cycles beyond the specified limits, an appropriate decision about the need for the nonlinear analysis may also be arrived at.

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