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ABSTRACT: In this article, the time domain madal analysis technique for evaluating the nonstationary responses of linear systems is combined with the equivalent linearization approach for determining the stationary responses of nonlinear soil strustures. A new analysis method is formed and the nonstationary responses of the earth and rockfill dam are calculated. The results from nonstationary analysis are compared with the ones from the stationary computations. The effectiveness of nonstationarity of input and output is investigated and the relevent conclusions are given.

NTRODUCTION

The dynamic reliability analysis of structure has been fast developed in recent years and fascinated a lot of researcher in civil engineering. The first reason is that the methods of seismic risk analysis of the site have been improved greatly, and then it becomes more and more possible to describle the earthquake ground motion from the point of view of probabilistic sense. The second reason is that for many important engineering projects, such as unclear powers, sea platformes, large dams and so on, the demands of aseismic design are more and more higher, this makes the researchers have to consider the influences of all kinds of unceratin factors. As the one of important engineering, the aseismic design of earth and rockfill dam is also a relative outstanding subject.

Due to the dynamic nonlinearity of soils, the earthquake response analysis of earth dams is restricted in some degree. The deterministic response analyses of history curves based on the equivalent linearization method were widely performed in the past research. One or several different earthquake waves were selected as the input motions and the dynamic response curves of every points on the dam were calculated. Then, the check on the earthquake safety was conducted according to the different damage forms and failure standards. However, a large number of computation results show that to a same dam, even though the input seismic waves have the equivalent control parameters, the dynamic responses excited by these input motions and the evaluations of the aseismic safety are noticeable difference. The main reason is owing to the dynamic nonlinearity of soils and the accumulating effectiveness of earthquake damage of earth dams. Therefor, it is very necessary to develop stochastic seismic response calculations and the fatigue failure analyses for the nonlinaer earth and rockfill dams subjected to the random earthquake excitations.

But as we know, it is very difficulty to consider the nonlinaerity and the randomness at the same time, and litter works has been done. In the references [1−2], a equivalent linearization approach was presented by us to handle the nonlinearity of soils in the stationary response analysis. In order to take into account the nonstationary features of earthquake motion and dynamic responses, Gasparini proposed a scattered time domain analysis technique for linear structures in the reference [3]. The object of this paper is to combine with above two methods and establish a new procedure. Then, this method is used in the nonstationary response analysis of an earth and rockfill dam. The effectivenesses of nonstationarity are initially investigated. Some conclusions are given.

NONSTATIONARY SEISMIC MOTION MODEL

A evolutionary filtered Gaussian white noise $\ddot{z}_g(t)$ is taken as the base seismic acceleration input model

$$\ddot{z}_g(t) = \ddot{x}_g(t) + \ddot{y}_g(t) \tag{1}$$

in which $\ddot{x}_g(t)$ is a nonstationary white noise process

$$\ddot{x}_g(t) = F(t)\ddot{x}_0(t) \tag{2}$$

and $\ddot{y}_g(t)$ is the reletive acceleration response of the filter

$$\ddot{y}_g(t) + 2\xi_g \omega_g \dot{y}(t) + \omega_g^2 y_g(t) = -\ddot{x}_g(t) \tag{3}$$

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In Eq.(2), F(t) is a deterministic modulating function and the maximum equals 1.0 ; $\ddot{x}_0(t)$ is a Gaussian white noise and the spectral intensity is G_0. When F(t) equals constant in the whole earthquake interval, $\ddot{x}_g(t)$, and then, $\ddot{z}_g(t)$, are all the stationary process. According to reference [3], Q(t) is used to represent the nonstationary strength function and $Q(t)=\pi G_0F_2(t)$, the maximum, $Q_m=\pi G_0$. In this paper, Q(t) is assumed as the piece–wise linear function and shown in Fig.1. The stationary power spectral density of $\ddot{z}_g(t)$ is described by Kanai–Tajimi spectral density function

$$G_{\ddot{z}_g}(\omega)=\frac{G_0[1+4\xi_g^2(\omega/\omega_g)^2]}{\{[1-(\omega/\omega_g)^2]^2+4\xi_g^2(\omega/\omega_g)^2\}} \tag{4}$$

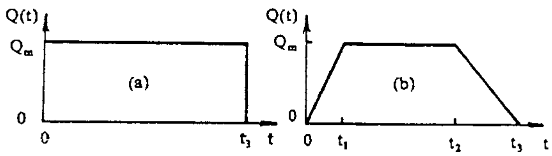

Fig.1 Nonstationary Time Function

After the average maximum of imput acceleration, $\bar{a}_m$, and peak factor R_p are selected, Q_m an be derived

$$Q_m=4\xi_g\bar{a}_m^2/[\omega_gR_p^2(1+4\xi_g^2)] \tag{5}$$

NONSTATIONARY RESPONSE ANALYSIS TO LINEAR PROPORTIONAL DAMPING SYSTEM

For a MDOF linear system excited by a horizontal earthquake acceleration process, $\ddot{z}_g(t)$, the equations of motion are

$$[M]\{\ddot{X}\}+[C]\{\dot{X}\}+[K]\{X\}=-[M]\{J_x\}\ddot{z}_g(t) \tag{6}$$

Under the assumption of proportional damping, arbitrary i–th modal vibration equation is of the form

$$\ddot{y}_i+2\xi_i\omega_i\dot{y}_i+\omega_i^2y_i=-\eta_i\ddot{z}_g(t) \tag{7}$$

in which y_i is i–th modal displacement; ξ_i and ω_i are respectively i–th modal damping ratio and frequency; η_i is i–th modal participation factor. In order to solve Eq.(7), the filter of Eq.(3) is introduced and modal equation is augmented as follows

$$\left.\begin{array}{l}\ddot{y}_g+2\xi_g\omega_g\dot{y}_g+\omega_g^2y_g=-\ddot{x}_g(t)\\ \ddot{y}_i+2\xi_i\omega_i\dot{y}_i+\omega_i^2y_i=-\eta_i[\ddot{y}_g(t)+\ddot{x}_g(t)]\end{array}\right\} \tag{8}$$

The state formulation for the augmented modal system is obtained by introducing state vector

$$\{a_i\}=[y_i\ \dot{y}_i\ y_g\ \dot{y}_g]^T$$

Then Eq.(8) becomes

$$\{\dot{a}_i\}=[A]\{a_i\}+\{B\}x_g(t) \qquad ($$

in which, matrix [A] and vector {B} are respectively

$$[A]=\begin{bmatrix}0 & 1 & 0 & 0\\ -\omega_i^2 & -2\xi_i\omega_i & \eta_i\omega_g^2 & 2\eta_i\xi_g\omega_g\\ 0 & 0 & 0 & 1\\ 0 & 0 & -\omega_g^2 & -2\xi_g\omega_g^2\end{bmatrix};\quad \{B\}=\begin{Bmatrix}0\\0\\0\\1\end{Bmatrix} \qquad ($$

Eq.(10) is a state matrix equation excited by nonstationa Gaussian white noise, $\ddot{x}_g(t)$. Through the solution of Eq.(1(we can indirectly obtain the modal response, y_i, excited nonstationary Gaussian filered white noise, $\ddot{z}_g(t)$.

When $\ddot{x}_g(t)$ in Eq.(10) is a zero mean Gaussian white noise, tl response $\{a_i\}$ is a fourth–order Gauss–Markov random pr cess and expected value is also zero. The method presented t Gasparini [3] is used to calculate the covariances of respon. in Eq.(10). At first, nonstationary time function, Q(t), is c vided into a number of small time intervals, each interv equals Δt. For arbitrary two modal state vectors $\{a_i\}$, $\{a_j\}$, u der obtained the covariance matrix at t_{k-1}, $[K_{ij}(t_{k-1})]$, the valu at t_k is given by

$$[K_{ij}(t_k)]=[\Phi_i(t_k)][K_{ij}(t_{k-1})][\Phi_j(t_k)]^T+\int_0^{\Delta t}[\Phi_i((t_k-\tau\ )]\{B\}Q(\tau)\{B\}^T[\Phi_j(t_k-\tau)]^Td\tau \qquad (1$$

here, $[K_{ij}(t_k)]=E[\{a_i(t_k)\}\{a_j(t_k)\}^T]$ is a (4×4) order matrix an contain the covariances of modal displacement y_i and mod velocity $\dot{y}_i$, when i=j, the variances of modal responses ar obtained; $[\Phi_i(t_k)]$ is the transition matrix for the augmente modal state vector and the formulation of each element is gi en by Gasparini [3]. The analytical expressions of the integr tion in Eq.(12) are also listed in reference [3]. For arbitrar two–two modals, Eq.(12) is calculated from first to final ir terval. Then, the n–order covariance matrix of mod dislacements or modal velocity may be combained fro $[K_{ij}(t_k)]$. These matrixes are further multiplied by releven mode shape transition matrixes, finially, the covariance ma trixes of the relative displacement {X}, relative velocity $\{\dot{X}\}$ absolute acceleration $\{\ddot{Z}\}$, etc., at every time interval, can als be obtained. Samilarly, other statistical parameters and th probabilities of exceeding response threshold can be derived For example, when the variances at t moment, $\sigma_x^2(t),\sigma_{\dot{x}}^2(t),\sigma_{\ddot{x}}^2$ (t) have been determined, the mean rate of upcrossing zero o relative displacement x(t) is given by

$$v_0^+(t)=(1/2\pi)[\sigma_{\dot{x}}(t)/\sigma_x(t)] \qquad (13$$

'he mean rate of upcrossing threshold x = b is

$$v_b^+(t) = (1/2\pi)[\sigma_{\dot{x}}(t)/\sigma_x(t)]exp[-b^2/2\sigma_x^2(t)] \qquad (14)$$

'he probability of exceeding response threshold |x| = b in in-
erval [0,t] is

$$P_b = 1 - exp[-2\int_0^t v_b^+(t)dt] \qquad (15)$$

'he band width parameter at this moment is

$$\alpha(t) = [\sigma_{\dot{x}}^2(t)/\sigma_x(t)\sigma_{\ddot{x}}(t)] \qquad (16)$$

NONSTATIONARY RESPONSE ANALYSIS TO NONLINEAR EARTH AND ROCKFILL DAMC

The computations above presented are only the linear system response analyses. Practically, the dynamic shear modulus and damping ratio of soils are dependent on the cyclic shear amplitude. In this paper, the dynamic nonlinear properties of soils are described by Hardin–Drnevich [4] hyperbolic model, i.e., the relationships among shear modulus G, damping ratio ξ and equivalent amplitude of cyclic shear strain, γ_e, are repersented by

$$\left.\begin{aligned} G = G(\gamma_e) &= \frac{1}{1+|\gamma_e/\gamma_R|}G_m \\ \xi = \xi(\gamma_e) &= \frac{|\gamma_e/\gamma_R|}{1+|\gamma_e/\gamma_R|}\xi_m \end{aligned}\right\} \qquad (17)$$

where $\gamma_R$denotes the reference shear strain; G_m, $\xi_m$are respectively maximum shear modulus and maximum damping ratio. In order to treat the nonlinearuty of soils, the modiffied equivalent linearization method [2] for evaluating stationary responses of soil structures is introduced to each time interval in the nonstationary response analysis process. At first, a typical calculating profile is selected in the dam and is discrted by the two–dimensional plane strain finite element networks. At the begining of the first interval, a group of initial shear modulus and damping ratio are assumed for each element, the system is considered as a linear one. The nonstationary random responses of the dam are calculated by using the time domain modal analysis technique above persented. Because the modulus and damping ratio of soils are related to shear strain amplitude, the iteration procedure should be performed. Based on the modified equivalent linearization method, the equivalent shear strain amplitude can be obtained from the following formula

$$\gamma_e = E[\gamma_p]_1 + \alpha(E[\gamma_p]_2 - E[\gamma_p]_1) \qquad (18)$$

in which, $E[\gamma_p]_1$ and $E[\gamma_p]_2$ are the mathematical expectancies when the cyclic shear strain amplitudes, γ_p, follow Gaussian and Reyleigh distrbution respectively. α is the band width parameter and determined by Eq.(16). Further, the equivalent shear strain γ_e is substituded into the Eq.(17), the modified element shear modulus and damping ratio can be directly interpolated out. Then, these new element shear modulus and damping ratio are used to replace the initial ones. The linear system are formed and nonstationary response calculations are conducted again in the first time step. This iteration pro–cedure is carried out repeatly until the strain compatial results, including modulus, damping ratio, acceleration, shear stress, etc., are obtained. These results are taken as the final response values at the end of first time step. Successively, the convergent element shear modulus and damping ratio are selected as the initial values in the second interval, same iteration process go on. For all of the time steps, the random responses of the earth dams are computed in the same way. Finally, by linking up the responses at the end of every time interval, the nonstationary stochastic seismic responses are obtained in the numerical form of scattered.

Another problem is that in reference [3] the damping matrix [C] in Eq.(6) was assumed to be orthogonal with respect to the modal vector . But in this paper, we utilized the variable damping finite element approach [5] to formed the damping matrix [C]. That is, element damping matrix $[c]^e$ is of the form

$$[c]^e = a[m]^e + b[k]^e \qquad (19)$$

here $[m]^e$and $[k]^e$are respectively the element mass matrix and stiffness matrix; a, b are the parameters and taken as $a = \xi_e\omega_1$, $b = \xi_e/\omega_1$, $\xi_e$is the element damping ratio and the values for different element are generally different, $\omega_1$is the fundamental natural frequency of the system. Then, the total damping matrix, [C], is combained form the all of the element damping matrixes. Because the element damping ratio $\xi_e$in the different elements is generally not same valus, the total damping matrix, [C], is not proportion to the total mass matrix [M] and stiffness matrix [K]. By using the classical modal decomposition method, the lower few undamped modes of the system are selected and Eq.(6) are transformed as lower–order principal coorainate equation

$$[M]^*\{\ddot{Y}\} + [C]^*\{\dot{Y}\} + [K]^*\{Y\} = -\{\Gamma\}\ddot{x}_g(t) \qquad (20)$$

in which $[M]^*$ and $[K]^*$ are the diagonal matrixes, but $[C]^*$ isn't. So, Eq.(20) will be coupled by non–zero off–diagonal terms in the $[C]^*$. Up to the present, nonstationary response analysis for Eq.(20) is still very difficult work. In order to utilize the method in reference [3], the non–zero off–diagonal elements in the $[C]^*$ are neglected in this paper. At the calculations of stationary response, it is found that the effects of neglecting off–diagonal terms in $[C]^*$ are small when the kinds of soils in the dam are similar. Thus, Eq.(20) may be uncoupled and the method above mentioned can be used, and then, nonstationary responses of the earth dams can also obtained easily.

NUMERICAL EXAMPLE

A homogeneous earth and rockfill dam with 60 meters in height is chosen to verify the effectiveness of the method proposed here. To compare, three different cases are studied for this dam, i.e., stationary input and stationary output (Case S.S.), stationary input and nonstationary output (Case S.N.), nonstationary input and nonstationary output (Case N.N.). The unit weight of soil is 1.6 ton / m and the Possion ratio equals 0.3. The finite element discretization of the dam is shown in Fig. 2. Hardin–Drnevich soil model parameters are $\xi_m=0.28$, $\gamma_R=2.5\times10^{-4}$, $G_m=69.9(K_m)\sqrt{\sigma_m'}$, $K_m=80$. The base acceleration input is a nonstationary nonwhite Gaussian process. The frequency and damping ratio of the filter are taken as $\omega_g=6\pi1$ / sec., $\xi_g=0.6$. The piece–wise linear function Q(t) is selected in two forms shown in Fig.1. In Fig.1 (a) $t_3=15$sec. and in [0, 15]sec. $Q(t)=Q_m$. This practically repersents a stationary function. In Fig.1 (b), $t_1=2$sec., $t_2=10$sec., $t_3=15$sec. Here, Q(t) contains a finite buildup time, a peroid of uniform intensity and a period of decay. In Eq.(5), the average maximum input acceleration and peak factor are respectively selected as $\bar{a}_m$ = 150 gal, $R_p=3.0$.

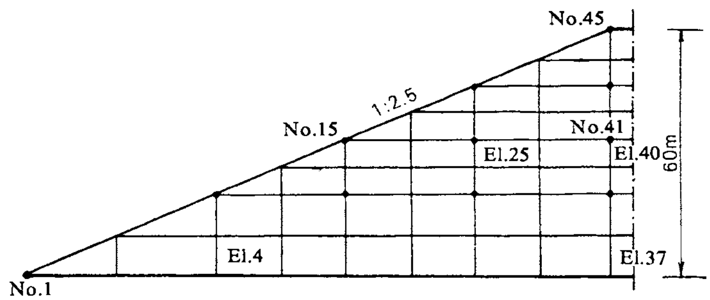

Fig.2 Finite Element Network of Dam Profile

In three calculations, Case S.S. is completed by using tl method suggested in reference [2]. The total vibration time a taken as 20sec., the time step $\Delta t=0.5$sec. In each interval, tl iteration numbers of dynamic characteristics of soils are take as 4 or 5 times. The lower five modes are selected in the cor putation.

Fig.3 shows the evolution of the mean square absolu acceleration responses at the different four nodes in thr cases. Similarly, Fig.4 gives the evolution of the mean squa shear stress responses in the different four elements. It is note that the results from Case S.S. are constent values in interv [0, 15]sec., while the ones from Case S.N. and Case N.N. ap pear nonstationary changes with nonstationarity of the inpu motions. The results of Case S.N. show that after the end c input motions, the free vibration responses of the dam wi attenuate and last in due course of time. But since Case S.S neglected the nonstationary responses, it can not predict th attenuation free vibrations, and therefor it will assess the fai ure on the lower side. The comparisons between Case S.N and Case N.N. show that the responses of the dam are seriou ly affected by the nonstationary characteristic of the input mc tion. The responses of Case S.N. are much larger than ones o Case N.N. in the buildup time and the decay peroid becaus Case S.N. don't consider the change of intensity of the inpu motion. Obviously, the failure predicted by Case S.N. may b the most intensive while the practical failure probably isn't s serious. From above Figs., it can also be seen that althougl the rcsults from three different cases exist some errors, th three values are almost equal at the uniform strength period o the input motion, which indirectly verifies the effectiveness o the persented new method.

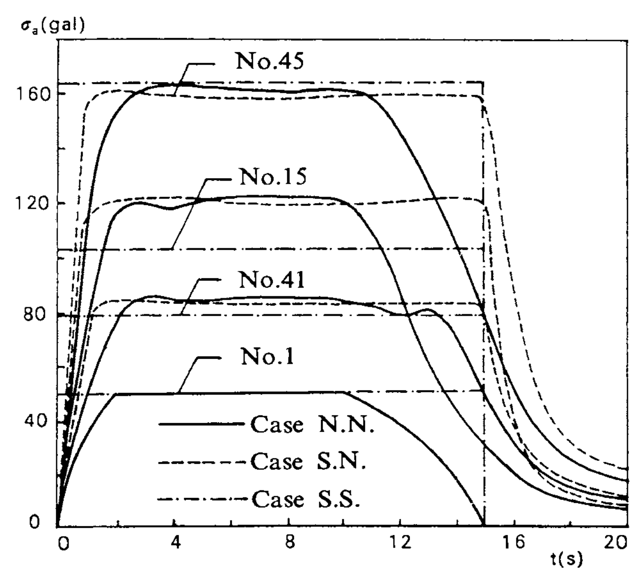

Fig.3 Mean Square Acceleration Evolution Curves

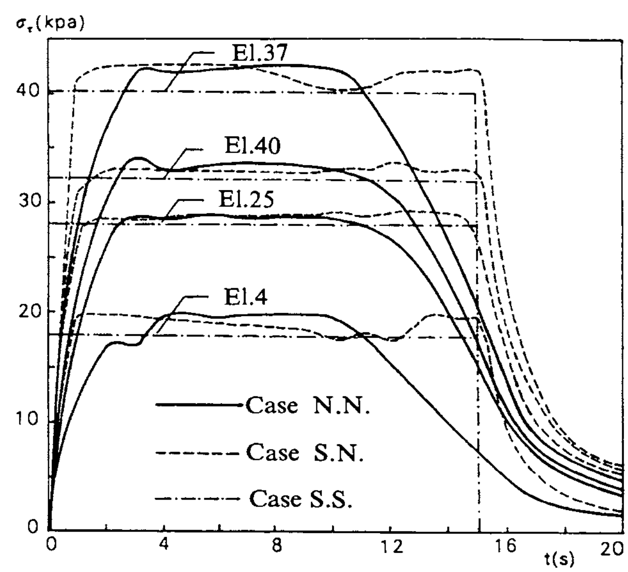

Fig.4 Mean Square Shear Stress Evolution Curves

ig.5 and Fig.6 respectively give the distributions of the mean quare acceleration and shear stress in the cases of S.S and I.N. at t = 6 sec. Obviously, after the input motion and output responses reach basically stationary, no matter which is ase S.S or case N.N., the values of dynamic responses are ery close.

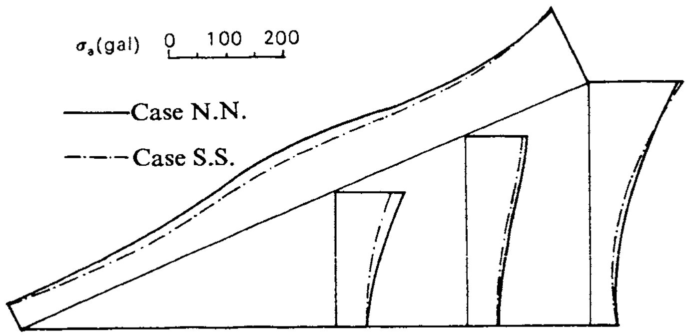

Fig.5 Mean Square Acceleration Distributions

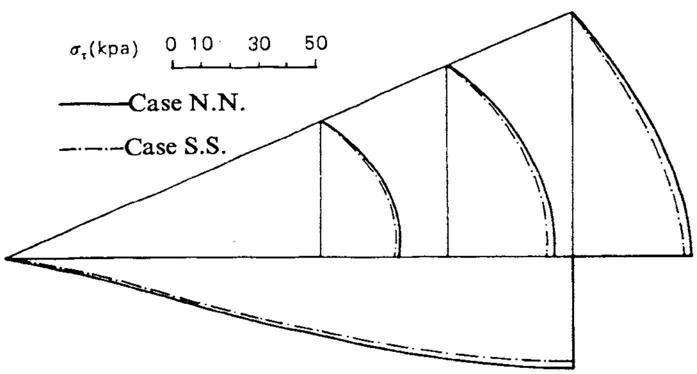

Fig.6 Mean Square Shear Stress Distributions

SUMMARIZATIONS AND CONCLUSIONS

Stochastic seismic response analysis of earth and rockfill dams is a very significant work. On the basis of the time domain modal analysis technique and the equivalent linear procedure, a new method is suggested in this paper and the nonstationary random seismic responses of an earth dam are evaluated. The calculating results show that persented method is practicable. The comparisons of three cases illustrate that the nonstationarity of input motion is a important factor influencing the responses of the dam. The stationary response analysis may estimate the dynamic damage on the low side due to neglecting the attenuation free vibration. On the contrary, if only was the nonstationarity of dynamic responses considered but the one of input motion was neglected, the failure would be assessed on the high side. Some further investigations will be continued, such as effective stress stochastic response calculations of soil structures, accumulated damage analysis of sand liquefaction, etc.

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