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Dynamic analysis and reliability assessment of structures with uncertain-but-bounded parameters under stochastic process excitations

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ABSTRACT

This paper presents the non-deterministic dynamic analysis and reliability assessment of structures with uncertain-but-bounded parameters under stochastic process excitations. Random ground acceleration from earthquake motion is adopted to illustrate the stochastic process force. The exact change ranges of natural frequencies, random vibration displacement and stress responses of structures are investigated under the interval analysis framework. Formulations for structural reliability are developed considering the safe boundary and structural random vibration responses as interval parameters. An improved particle swarm optimization algorithm, namely randomised lower sequence initialized high-order nonlinear particle swarm optimization algorithm, is employed to capture the better bounds of structural dynamic characteristics, random vibration responses and reliability. Three numerical examples are used to demonstrate the presented method for interval random vibration analysis and reliability assessment of structures. The accuracy of the results obtained by the presented method is verified by the randomised Quasi-Monte Carlo simulation method (QMCSM) and direct Monte Carlo simulation method (MCSM).

1. Introduction

Uncertainty is inevitable as modern structure's responses are subject to many non-deterministic features such as loads, material properties, geometric dimensions, and other parameters. The concern regarding uncertainty arises from evaluation of structures based on deficiency of above information of structures. The analysis with uncertainty is to satisfy the critical cases of design of such structures as well as to make the mathematical model of structures in the realistic way, whose performances play an important role in safety criteria. The error in any phase of modelling the mathematical uncertainty can be detrimental to uncertainty estimation [1]. The traditional approach for uncertainty analysis is the probability theory in which design parameters are treated as random variables specified by probability density function [2]. However, to satisfy precise condition for modelling this function as using probability theory, it is required to construct a great amount of information which is not always available for all applications. In such situations, another approach for uncertainty modelling should be considered. The case in which random distribution of uncertainty feature is not available, but its lower and upper bounds exist, can lead to the representation of interval method. This method was first developed by Moore [3]. Then the application of this method to several systems has been presented by Moore and Bierbaum [4]. Many studies have shown the advantages for this method in structural analysis for static and dynamic problems [5–14]. The important tool to evaluate uncertainty analysis including interval analysis is the Monte Carlo Simulation which allows investigating the possible change range of the system outputs. However, it requires more computational effort and time consuming to attain the sufficient solutions for large scale structures in accordance with the increasing number of iterations. Therefore, the endeavour to address these issues in uncertainty analysis is considered by using another algorithm. The interval analysis is to capture the lower and upper bounds of system outputs which can be converted to optimization problems. Although there are many algorithms to solve optimization problems, PSO proposed by Kennedy and Eberhart [15] is more effective compared to other optimization algorithms [16,17] as this method requires fewer number of iteration to attain the same or better results. The efficiency of PSO has been recently verified and developed on many engineering problems including structural optimization [18–20]. Improvements have been made by many researchers to strengthen the performance of PSO. A noticed improvement of PSO is that particles are initialized by using randomized low discrepancy sequences instead of common

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uniform initialization (pseudo random number generator) [21–23] as low discrepancy sequences are deterministic but more uniform and their optimal discrepancy can be achieved. However, these studies only focus on mathematical expressions with one variable. Liu et al. [24] employed the low-discrepancy sequences initialized high-order nonlinear particle swarm optimization algorithm (LHNPSO) for interval dynamic response analysis of vehicle–bridge interaction system with uncertainty but simple model of structure. Thus, there are still latent challenges regarding application of this method with multi-dimension particles and various kinds of structures especially large scale structures. In this paper, PSO with randomized low-discrepancy sequences initialized particles and high-order nonlinear inertia weight, denoted as RLHNPSO with multi-dimension parameters is employed to investigate the structural analysis for benchmark examples comprising large scale structures under stationary stochastic process excitation.

Taking the element of uncertainty to the system model, evaluation for the hazard posed by system operation is a great of interest termed as reliability analysis. Reliability is a fundamental attribute for the safe operation of any modern system [25]. The investigation of reliability analysis is discussed in many fields of engineering [26–30]. For structure analysis, reliability is a fundamental feature concerning safe performance of structures over a life time [31,32]. In real practical design, it is really necessary to take dynamic reliability into account because structures are commonly analysed with the time-dependent loads or random excitation such as wind, earthquake, and explosion. The so-called first-passage failure probability emerged when Rice [33] introduced the level crossing theory defined to obtain time-dependent reliability. Then, Coleman [34] introduced formulation for the first-passage considering Poisson process. Although there are some studies on dynamic reliability [35–38], most models of reliability analysis are based on the probability theory. However, this technique is not always available for applications as probabilistic information is not defined for some cases. Consequently, fuzzy theory or imprecise probabilistic theory [39–41] is employed to assess the structural dynamic reliability.

In this paper, random vibration analysis and reliability assessment are implemented while the structural parameters are defined as uncertain-but-bounded variables. Mathematical expressions considering interval safe boundary are developed to calculate the change ranges of dynamic reliability of structures. The RLHNPSO is incorporated into the analysis process to find out the better enclosure of dynamic reliability. This paper is organized as follows. Section 2 presents the interval dynamic analysis of structures under stochastic process excitations. Section 3 discusses the dynamic reliability assessment of structures with interval parameters. Section 4 introduces the application of the low-discrepancy sequences initialized high-order nonlinear particle swarm optimization algorithm, followed by the flow chart for interval structural random vibration analysis and reliability assessment illustrated in Section 5. Numerical examples are given in Section 6 and conclusions are drawn in Section 7.

2. Interval analysis for structures under stationary process excitation

The equation of motion of structures with interval parameters under stationary random process excitation can be expressed as the following:

$$[M^I]\{\ddot{u}^I(t)\}+[C^I]\{\dot{u}^I(t)\}+[K^I]\{u^I(t)\}=-[M^I]\{1\}\ddot{x}_g(t) \tag{1}$$

where $[M^I]$, $[C^I]$ and $[K^I]$ are the interval matrices with respect to mass, damping and stiffness of structures, respectively. $\{u^I(t)\}$, $\{\dot{u}^I(t)\}$ and $\{\ddot{u}^I(t)\}$ are interval vectors defining structural displacement, velocity and acceleration. $\{1\}$ is a column vector with all components 1 and $\ddot{x}_g(t)$ is the random ground acceleration to represent stochastic process excitations.

The global matrix of a structure can be simplified as an addition of the element contributions:

$$[K^I]=\sum_{k=1}^{N}K_k^I \tag{2}$$

$$[M^I]=\sum_{k=1}^{N}M_k^I \tag{3}$$

where K_k^I and M_k^I are interval global matrices of stiffness and mass of each element corresponding to dimension of freedom degree of whole structures. N is the number of elements of the structure.

By employing Rayleigh's quotient in the form of modal analysis and spectral matrices, frequency equation is expressed as:

$$diag[(\omega_j^I)^2]=\frac{[\phi^I]^T[K^I][\phi^I]}{[\phi^I]^T[M^I][\phi^I]} \tag{4}$$

where $[\phi^I]$ and diag$[(\omega_j^I)^2]$ are interval matrices of natural modes and natural frequencies, respectively. ω_j^I is the jth interval natural frequency.

Normalised modal matrix has the following orthogonal properties:

$$[\phi^I]^T[M^I][\phi^I]=[I] \tag{5}$$

$$[\phi^I]^T[K^I][\phi^I]=diag[(\omega_j^I)^2] \tag{6}$$

The solution of coupled equations presented in Eq. (1) can be achieved by using Duhamel integral:

$$\{u(t)\}=\int_0^t[\phi^I][h^I(t-\tau)][\phi^I]^T[M^I]\{1\}\ddot{x}_g(t)d\tau \tag{7}$$

The interval matrix of impulse response function is determined by:

$$\{h^I(t)\}=diag\{h_j^I(t)\}$$

$$h_j^I(t)=\begin{cases}\frac{1}{\omega_{jD}^I}\exp(-\xi_j\omega_j^I t)\sin\ \omega_{jD}^I t & t\le 0\\ 0 & t<0\end{cases} \tag{8}$$

where ξ_j is the jth modal damping of the structure, and $\omega_{jD}^I=\omega_j(1-\xi_j^2)^{1/2}$. Since $\xi_j\ll 1$, $\omega_{jD}^I\approx\omega_j^I$ can be used. By performing a Fourier transformation for the correlation function matrix and integrating it step by step, the mean square value matrix of the structural displacement $(\psi_u^I(t))^2$can be obtained as [38,42]:

$$(\psi_u^I(t))^2=\int_0^\infty[\phi^I][H^I(\omega)][\phi^I]^T[M^I]$$
$$\{1\}\{1\}^T[M^I]^T[S_p(\omega)][\phi^I][H_*^I(\omega)][\phi^I]^T d\omega \tag{9}$$

where ω the computational frequency variable in frequency domain. $[S_p(\omega)]$ is the equivalent one-side power spectral density matrix of $\ddot{x}_g(t)$. In this study, model of Kanai–Tajimi designated in reference [43] is employed to define the random ground level accelerations:

$$[S_p(\omega)]=diag\left\{\frac{(1+4(\xi_g\omega/\omega_g)^2)\times S_0}{(1-\omega^2/\omega_g^2)^2+4(\xi_g\omega/\omega_g)^2}\right\} \tag{10}$$

$H^I(\omega)$ is the interval matrix with respect to the frequency response function matrix of the structures and can be expressed as:

$$H^I(\omega)=diag\left\{H_j^I(\omega)\right\}$$

$$H_j^I(\omega)=\frac{1}{(\omega_j^I)^2-\omega^2+i2\xi_j\omega_j^I\omega},\quad j=1,2,\ldots,n \tag{11}$$

where $i=\sqrt{-1}$ is the complex number. $[H_*^I(\omega)]$ is an interval matrix termed as complex conjugate matrix of $H^I(\omega)$.