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Nonstationary seismic response analysis of long-span structures by frequency domain method considering wave passage effect

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ABSTRACT

In this paper, a frequency domain method is proposed for the nonstationary seismic analysis of long-span structures subjected to random ground motions considering the wave passage effect. Based on the correlation analysis theory and fast Fourier transform (FFT), a semi-analytical solution is derived for the evolutionary power spectral density of the random response of long-span structures in the frequency domain. The expression of this solution indicates that the evolutionary property of nonstationary random responses can be determined completely by the modulation function of random ground motions, and hence the solution has clear physical interpretations. For slowly varying modulation functions, the FFT can be implemented with a small sampling frequency, so the present method is very efficient within a given accuracy. In numerical examples, nonstationary random responses of a long-span cable stayed bridge to random ground motions with the wave passage effect are studied by the present method, and comparisons are made with those of the pseudo excitation method (PEM) to verify the present method. Then the accuracy and efficiency of the present method with different sampling frequencies are compared and discussed. Finally, the influences of the apparent velocity of the seismic waves on nonstationary random responses are investigated.

1. Introduction

During an earthquake, the energy released at the epicenter transfers to the ground surface in the form of seismic waves. Since the waves travel along different paths and through a complex medium, ground motions caused by the earthquake at different locations will have significant differences. Even if the propagation medium is exactly uniform, there is still a difference in the arrival times of seismic waves at different locations due to their different distances to the epicenter. This phenomenon is known as the "wave passage effect". Long-span structures are generally important facilities, e.g. long-span bridges, dams, or nuclear power plants. Therefore, their aseismatic capabilities are highly relevant to public safety. In seismic analysis, long-span structures have their own special features compared to general building structures. A major feature is that these structures extend over long distances parallel to the ground, so their supports undergo different motions during an earthquake. Hence, the dynamic behaviors of long-span structures with and without consideration of the wave passage effect have significant differences [1,2].

The time-history method is widely applied for the random analysis of long-span structures subjected to an earthquake with spatial variation [3]. This method is based on stochastic simulation, and response parameters (mainly mean values and variances) are obtained through statistical analysis of samples of the random responses. Its main drawback, however, is that it has a huge computational cost. Over three decades, some more efficient methods have been developed. One of them is an extension of the conventional response spectrum method, which was initially only feasible for uniform seismic excitation. Der Kiureghian and Neuenhofer [4] developed a special response spectrum method for the response of structures to a random earthquake considering the wave passage effect, incoherence effect and site-response effect. Yamamura and Tanaka [5] presented an analysis of a suspension bridge to multi-support seismic excitations. In their work, ground motions within a group of adjacent supports on continuous soil or rock were assumed to be uniform and synchronized, while those of different groups were treated as non-uniform and uncorrelated. Berrah and Kausel [6] proposed a modified response spectrum method to address the problem of long-span structures subjected to imperfectly correlated seismic excitations. However, they did not consider the influence of quasi-static displacement. Due to the naturally random properties of the

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earthquake, it is more rational to study the seismic response of long-span structures using random vibration theory. Heredia-Zavoni and Vanmarcke [7] developed a random vibration method for the seismic analysis of linear multi-support systems. This method reduced the response evaluation to that of a series of linear one degree systems in a way that fully accounts for the space-time correlation structure of the ground motion. Lee and Penzien [8] studied random responses of piping systems under multi-support excitations, obtaining mean and extreme values of the systems in either the time or the frequency domain. Lin et al. [9] simplified a surface-mounted pipeline as an infinitely long Bernoulli-Euler beam attached to evenly spaced ground supports, and solved its random seismic responses. Zanardo et al. [10] carried out a parametric study of the pounding phenomenon associated with the seismic response of multi-span simply supported bridges with base isolation devices. Tubino et al. [11] investigated the influence of the partial correlation of the seismic ground motion on long-span structures by introducing suitable equivalent spectra. Lupoi et al. [12] studied the effects of the spatial variation of ground motion on the response of bridge structures. The results showed that the spatial variation affects the random response considerably. Lin et al. [13,14] proposed a random vibration method known as the pseudo-excitation method (PEM). In the framework of the PEM, the random vibration analysis was reduced to relatively simple harmonic or transient analysis, and hence its computation was of high efficiency. The PEM was also used for seismic responses of long-span structures to ground motion with spatial variations.

In the research mentioned above, ground motions were always assumed to be stationary random processes. However, some practical observation results showed that the intensity of the ground motion had three obvious stages, i.e. increasing, steady and decreasing, during the duration of the earthquake. Hence it is more rational to assume the ground motion as a nonstationary random process. Spectral methods, such as Wigner-Ville spectrum [15], physical spectrum [16], evolutionary spectrum [17,18] etc., can provide a general description of the energy-frequency properties of nonstationary processes, and thus have been a focal point of study. The evolutionary power spectral density (PSD) was widely used in the earthquake engineering for its clear physical interpretation and relatively simple mathematical derivation [19,20]. An evolutionary PSD is always defined as the product of a deterministic uniform or nonuniform modulation function and a stationary PSD. Based on a spectral representation based simulation algorithm, Deodatis [21] introduced an iterative scheme to generate seismic ground motion samples at several locations on the ground surface that were compatible with prescribed response spectra, correlated according to a given coherence function, include the wave passage effect. Alderucci and Muscolino [22] presented a random vibration analysis of linear classically damped structural systems subjected to fully nonstationary multicorrelated excitations and gave a closed-form solution of the evolutionary PSD of the response. Combining the experimental data of a multi-support seismic shaking table test and structural health monitoring findings, Ozer et al. [23] developed a framework to evaluate random seismic response and estimate reliability of bridges under multi-support excitations. In the authors' previous works [13,24], the PEM and a highly accurate step-by-step integration method named the Precise Integration Method (PIM) were combined to solve nonstationary random responses of long-span structures under the earthquake with consideration of the wave passage effect. Generally, a time-frequency domain analysis is required to obtain the solution of the evolutionary PSD when structures are excited by a nonstationary random excitation. During the time-frequency domain analysis, the time domain integration is performed at each frequency point. To achieve accurate results, small time steps are required in the time domain integration, especially for a wide band random excitation with high frequency components. Hence, there will inevitably be a huge computational cost.

Combining the evolutionary PSD and correlation analysis theory, this paper develops a frequency domain method for the random vibration analysis of long-span structures subjected to ground motions with the wave passage effect. This method can be used to obtain the semi-analytical solution of the evolutionary PSD of random responses and its computation is very efficient. This paper is structured as follows. In Section 2, governing equations of long-span structures subjected to nonuniform earthquake excitation are given. Section 3 presents the evolutionary PSD model with consideration of the wave passage effect. By separating the deterministic modulation function from the evolutionary PSD, Section 4 establishes a frequency domain method to obtain the semi-analytical solution of random responses. In Section 5, a long-span cable-stayed bridge is adopted as an example structure. The present method is applied to random vibration analysis of the bridge and the results are compared to those of the PEM to verify the present method. The influences of the wave velocity on random responses are compared and discussed. Section 6 gives some conclusions.

2. Governing equations of structures under nonuniform seismic excitation

The governing equations of a long-span structure with N supports and n degrees of freedom (DOF) subjected to nonuniform seismic excitation can be written as [25]

$$\begin{bmatrix}\mathbf{M}_{aa} & \mathbf{M}_{ab}\\ \mathbf{M}_{ab}^{\mathrm{T}} & \mathbf{M}_{bb}\end{bmatrix}\begin{Bmatrix}\ddot{\mathbf{y}}_a(t)\\ \ddot{\mathbf{y}}_b(t)\end{Bmatrix}+\begin{bmatrix}\mathbf{C}_{aa} & \mathbf{C}_{ab}\\ \mathbf{C}_{ab}^{\mathrm{T}} & \mathbf{C}_{bb}\end{bmatrix}\begin{Bmatrix}\dot{\mathbf{y}}_a(t)\\ \dot{\mathbf{y}}_b(t)\end{Bmatrix}+\begin{bmatrix}\mathbf{K}_{aa} & \mathbf{K}_{ab}\\ \mathbf{K}_{ab}^{\mathrm{T}} & \mathbf{K}_{bb}\end{bmatrix}\begin{Bmatrix}\mathbf{y}_a(t)\\ \mathbf{y}_b(t)\end{Bmatrix} = \begin{Bmatrix}\mathbf{0}\\ \mathbf{p}_b(t)\end{Bmatrix} \tag{1}$$

where the subscripts "a" and "b" indicate the non-support and support DOF, respectively; $\mathbf{y}_a(t)$ is an n-dimensional vector containing all non-support displacements; m-dimensional vectors $\mathbf{y}_b(t)$ and $\mathbf{p}_b(t)$ represent the enforced support displacements and forces at all supports, respectively; the $n \times n$ matrices $\mathbf{M}_{aa}$, $\mathbf{C}_{aa}$ and $\mathbf{K}_{aa}$ [$\mathbf{M}_{bb}$, $\mathbf{C}_{bb}$ and $\mathbf{K}_{bb}$] are the mass, damping and stiffness matrices associated with $\mathbf{y}_a(t)$ [$\mathbf{y}_b(t)$]; the superscript "T" denotes transposition. Note that when the lumped mass matrix approximation is adopted, $\mathbf{M}_{ab}$ is null.

In order to solve Eq. (1), the absolute displacement $\mathbf{y}_a(t)$ can be decomposed into the following two parts [25]:

$$\begin{Bmatrix}\mathbf{y}_a(t)\\ \mathbf{y}_b(t)\end{Bmatrix}=\begin{Bmatrix}\mathbf{y}_s(t)\\ \mathbf{y}_b(t)\end{Bmatrix}+\begin{Bmatrix}\mathbf{y}_d(t)\\ \mathbf{0}\end{Bmatrix} \tag{2}$$

in which $\mathbf{y}_s(t)$ and $\mathbf{y}_d(t)$ are the quasi-static and dynamic displacement vectors, respectively, which satisfy the following equations:

$$\begin{bmatrix}\mathbf{K}_{aa} & \mathbf{K}_{ab}\\ \mathbf{K}_{ab}^{\mathrm{T}} & \mathbf{K}_{bb}\end{bmatrix}\begin{Bmatrix}\mathbf{y}_s(t)\\ \mathbf{y}_b(t)\end{Bmatrix}=\begin{Bmatrix}\mathbf{0}\\ \mathbf{p}_b(t)\end{Bmatrix} \tag{3}$$

Expanding the first row of Eq. (3) gives

$$\mathbf{y}_s(t) = -\mathbf{K}_{aa}^{-1}\mathbf{K}_{ab}\mathbf{y}_b(t) \tag{4}$$

Assuming that the damping force is proportional to the dynamic relative velocity $\dot{\mathbf{y}}_d(t)$ instead of $\dot{\mathbf{y}}_a(t)$, the first row of Eq. (1) can be rewritten as

$$\mathbf{M}_{aa}\ddot{\mathbf{y}}_d(t)+\mathbf{C}_{aa}\dot{\mathbf{y}}_d(t)+\mathbf{K}_{aa}\mathbf{y}_d(t) = \mathbf{M}_{aa}\mathbf{K}_{aa}^{-1}\mathbf{K}_{ab}\ddot{\mathbf{y}}_b(t) \tag{5}$$

In the random vibration analysis of long-span structures under nonuniform seismic excitation, seismic waves are always assumed to travel along a certain direction. For long-span structures with N supports, the accelerations of ground motions at supports in the travelling direction can be expressed as the following N-dimensional vector

$$\ddot{\mathbf{u}}_b(t) = \{\ddot{u}_1(t),\, \ddot{u}_2(t), \ldots,\, \ddot{u}_N(t)\}^{\mathrm{T}} \tag{6}$$

At the same time, $\ddot{\mathbf{y}}_b(t)$ in Eq. (5) can also be expressed as the following m-dimensional ground acceleration vector