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Random vibration and deterministic analyses of cable-stayed bridges to asynchronous ground motion

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Abstract. In this paper, a comparison of various random vibration and deterministic dynamic analyses of cable-stayed bridges subjected to asynchronous ground motion is presented. Different random vibration methods are included to determine the dynamic behaviour of a cable-stayed bridge for various ground motion wave velocities. As a numerical example the Jindo Bridge located in South Korea is chosen and a 413 DOF mathematical model is employed for this bridge. The results obtained from a spectral analysis approach are compared with those of two random vibration based response spectrum methods and a deterministic method. The analyses suggest that the structural responses usually show important amplifications depending on the decreasing ground motion wave velocities.

Key words: spectral analysis approach; multiple support response spectrum method; deterministic analysis; asynchronous ground motion; cable-stayed bridge.

1. Introduction

The effects of spatial variability between the supports of lifeline structures have drawn the attention of researchers, and various structural configurations subjected to either deterministic or stochastic seismic ground motions have been analysed. For structures excited by loads with random characteristics, the responses can be predicted more realistically if stochastic approaches are adopted. In the deterministic approach, a recorded accelerogram is used as the input motion at one point, and the differential motion between two points is obtained by a delay in the arrival of the seismic wave between the points. In the stochastic approach, however, a power spectral density

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function for acceleration is assigned and the spatial variation is considered to be an exponentially decaying function. When the effect of time difference between two unmodified signals is present, the motion of the support points is said to be asynchronous.

Many studies (Nazmy and Abdel-Ghaffar 1987, Garevski *et al*. 1988, Nazmy and Abdel-Ghaffar 1992, Schemman 1997) have been performed in order to investigate the effect of differential support motions on the response of long-span bridges by deterministic methods. The overall conclusion was that the differential support motions produce different responses than those of the fully correlated motions and should be incorporated in the dynamic analysis of these bridges.

The effect of spatially varying ground motions on the random vibration response of extended structures has been of concern in the last two decades (Abdel-Ghaffar and Rubin 1982, Harichandran and Wang 1988, Zerva 1991, Hao 1993, Harichandran *et al*. 1996, Monti *et al*. 1996, Zembaty 1997). These studies performed by the random vibration based spectral analysis method generally underline the significance of the spatial variability of ground motions. Literature on the application of the random vibration analysis for seismic analysis of cable-stayed bridges is meagre. Recently Allam and Datta (1999) analysed cable-stayed bridges subjected to correlated ground motions without considering the finite propagating velocity of the ground motion. Soyluk and Dumanoglu (2000) compared asynchronous deterministic analysis responses of a cable-stayed bridge with those of the stochastic analysis responses where the ground motion was assumed to be of the form of uniform ground motion. In a recently conducted study by Alkhaleefi and Ali (2002), the needed participation factors for multiple-support motion random vibration analysis was computed from modal reactions, mode shapes and natural frequencies eliminating the need to solve for the pseudo-static problem, which could become expensive and time consuming for large problems. A simple expression was also developed for the pseudo-static solution, which can be used in the expressions for the pseudo-static and covariance components of the power spectral densities.

The response spectrum method has proven to be an accurate and practical method for seismic analysis. In recent years Der Kiureghian and Neuenhofer (1991) developed a response spectrum method for the seismic analysis of structures subjected to spatially varying ground motions. Nakamura *et al*. (1993) applied this method on the Golden Gate bridge for spatially varying ground motions. It was found that the new response spectrum method offers a simple and viable alternative for seismic analysis of multiply-supported structures. Der Kiureghian *et al*. (1997) performed a comprehensive investigation of the response spectrum method for seismic analysis of three to five-span bridge structures. In this study special attention was given to the effect of site response. Allam and Datta (2000) presented a response spectrum method for the seismic analysis of cable-stayed bridges subjected to correlated stationary random ground motion. The method was based on the relationship between the power spectral density function and the response spectrum of the input ground motion. Although the analysis takes into account the partial correlation of ground motions between the supports, the finite propagation effect was ignored.

The main objective of this study is to compare the random vibration and deterministic responses of a cable-stayed bridge, namely the Jindo Bridge, subjected to asynchronous ground motion. Three different random vibration methods are utilised to determine the dynamic behaviour of the cable-stayed bridge for various wave velocities. As one of these methods is the spectral analysis approach of random vibration theory based on the power spectral density specification of the ground motion, the other two methods are random vibration based response spectrum methods. While the first response spectrum method is the multiple-support response spectrum method (MSRS) developed by

Der Kiureghian and Nueunhofer (1991), the other is based on the relationship between the power spectral density function and the response spectrum of the input ground motion, and the fundamentals of the random vibration theory (Allam and Datta 2000).

2. Spectral analysis approach

Spectral analysis approach is based on principles of stationary random vibration theory and provides an approximate estimate of the mean of the absolute maximum response of the structure. Any response quantity can be decomposed into dynamic and pseudo-static components, when there is a differential excitation at the supports. The total mean-square response can be obtained from Harichandran and Wang (1988)

$$\sigma_z^2 = \sigma_{zd}^2 + \sigma_{zs}^2 + 2\mathrm{Cov}(z_s, z_d) \tag{1}$$

where σ_{zd}^2 and σ_{zs}^2 are the dynamic and pseudo-static variances, respectively, and $\mathrm{Cov}(z_s, z_d)$ is the covariance between the dynamic and pseudo-static responses. The three components on the right-hand side of Eq. (1) are given by

$$\sigma_{z_d}^2 = \sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{r}\sum_{l=1}^{r} \psi_i \psi_j \Gamma_{ki}\Gamma_{lj} \int_{-\infty}^{\infty} H_i(-w)H_j(w)G_{\ddot{u}_k\ddot{u}_l}(w)dw \tag{2}$$

$$\sigma_{z_s}^2 = \sum_{k=1}^{r}\sum_{l=1}^{r} A_k A_l \int_{-\infty}^{\infty} \frac{1}{w^4} G_{\ddot{u}_k\ddot{u}_l}(w)dw \tag{3}$$

$$\mathrm{Cov}(z_s, z_d) = \sum_{j=1}^{n}\sum_{k=1}^{r}\sum_{l=1}^{r} \psi_j A_k \Gamma_{lj}\left(-\int_{-\infty}^{\infty} \frac{1}{w^2} H_j(w)G_{\ddot{u}_k\ddot{u}_l}(w)dw\right) \tag{4}$$

in which n is the number of modes used in the analysis, r is the number of restrained degrees of freedom, ψ_j is the response z from the jth mode, A_k is the response z due to a unit displacement of support DOF k, Γ_{ki} is the participation factor corresponding to mode i and support DOF k, $H_j(w)$ is the modal frequency response function and $G_{\ddot{u}_k\ddot{u}_l}$ is the cross spectral density function of accelerations between support DOF k and l.

3. Response spectrum method

The multiple support response spectrum method was developed by Der Kiureghian and Neuenhofer (1991) based on fundamental principles of stationary random vibration theory. The combination rule for the mean of absolute peak response is given in the form

$$E[\max|z(t)|] = \left[\sum_{k=1}^{m}\sum_{l=1}^{m} a_k a_l \rho_{u_k u_l} u_{k,\max} u_{l,\max} + 2\sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{j=1}^{n} a_k b_{lj} \rho_{u_k s_{lj}} u_{k,\max} D_l(w_j, \zeta_j) + \sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n} b_{ki} b_{lj} \rho_{s_{ki} s_{lj}} D_k(w_i, \zeta_i) D_l(w_j, \zeta_j)\right]^{1/2} \tag{5}$$

where,

$$a_k = q^T r_k \qquad k = 1, \ldots, m \tag{6}$$

$$b_{ki} = q^T \phi_i \beta_{ki} \qquad k = 1, \ldots, m; \; i = 1, \ldots, n \tag{7}$$

are the effective influence coefficients and effective modal participation factors, respectively, $u_{k,\max}$ denotes the mean peak ground displacement at support degree of freedom k, $D_k(\omega_i, \zeta_i)$ denotes the displacement response spectrum ordinate at support degree of freedom k for the frequency and mode of i, and $\rho_{u_k u_l}$, $\rho_{u_k s_{lj}}$, $\rho_{s_{ki} s_{lj}}$ are cross-correlation coefficients between the support motions and the modes of the structure. The cross-correlation coefficients are defined by

$$\rho_{u_k u_l} = \frac{1}{\sigma_{u_k}\sigma_{u_l}} \int_{-\infty}^{\infty} G_{u_k u_l}(w) dw \tag{8}$$

$$\rho_{u_k s_{lj}} = \frac{1}{\sigma_{u_k}\sigma_{s_{lj}}} \int_{-\infty}^{\infty} H_j(-w) G_{u_k \ddot{u}_l}(w) dw \tag{9}$$

$$\rho_{s_{ki} s_{lj}} = \frac{1}{\sigma_{s_{ki}}\sigma_{s_{lj}}} \int_{-\infty}^{\infty} H_i(w) H_j(-w) G_{\ddot{u}_k \ddot{u}_l}(w) dw \tag{10}$$

in which $H_i(w) = [w_i^2 - w^2 + 2i\zeta_i w_i w]^{-1}$ represents the frequency response function of mode i, and σ_{u_k} and $\sigma_{s_{ki}}$ are the root-mean-squares of the ground displacement $u_k(t)$ and the normalised modal response $s_{ki}(t)$, respectively (Nakamura *et al.* 1993).

4. Seismic excitation for random vibration analyses

In Eqs. (2-4) and Eqs. (8-10) $G_{\ddot{u}_k \ddot{u}_l}(w)$, $G_{u_k \ddot{u}_l}(w)$ and $G_{u_k u_l}(w)$ are cross-power spectral densities defined by

$$G_{\ddot{u}_k \ddot{u}_l}(w) = \gamma_{kl}(w)[G_{\ddot{u}_k \ddot{u}_k}(w) G_{\ddot{u}_l \ddot{u}_l}(w)]^{1/2} \tag{11}$$

$$G_{u_k \ddot{u}_l}(w) = -w^{-2}\gamma_{kl}(w)[G_{\ddot{u}_k \ddot{u}_k}(w) G_{\ddot{u}_l \ddot{u}_l}(w)]^{1/2} \tag{12}$$

$$G_{u_k u_l}(w) = w^{-4}\gamma_{kl}(w)[G_{\ddot{u}_k \ddot{u}_k}(w) G_{\ddot{u}_l \ddot{u}_l}(w)]^{1/2} \tag{13}$$

where $\gamma_{kl}(w)$ is the coherency function describing the variability of the ground acceleration processes for support degrees of freedom k and l as a function of frequency w. Recently Der Kiureghian (1996) proposed a general composite model of spatial seismic coherency function in the following form

$$\gamma_{kl}(w) = \gamma_{kl}(w)^{i}\exp[i(\theta_{kl}(w)^{w} + \theta_{kl}(w)^{s})] \tag{14}$$

where $\gamma_{kl}(w)^{i}$, $\gamma_{kl}(w)^{w}$ and $\gamma_{kl}(w)^{s}$ characterise the incoherence, the wave-passage and the site-response effects, respectively. In this study all the spatial effects are disregarded except the wave-passage effect and the coherency function is written as

$$\gamma_{kl}(w) = \gamma_{kl}(w)^{w} = \exp[i(\theta_{kl}(w)^{w})] \tag{15}$$

$$\theta_{kl}(w)^{w} = -\frac{wd_{kl}^{L}}{V} \tag{16}$$

where V is the apparent propagation velocity and d_{kl}^{L} is the projection of d_{kl} on the ground surface along the direction of propagation of seismic waves.

For the spectral analysis approach, the power spectral density function of the ground acceleration ($\ddot{v}_{g_1}$) characterising the earthquake process is assumed to be of the following form modified by Clough and Penzien (1993)

$$G_{\ddot{u}_k\ddot{u}_k}(w) = So\frac{w_g^4 + 4\zeta_g^2 w_g^2 w^2}{(w_g^2 - w^2)^2 + 4\zeta_g^2 w_g^2 w^2}\,\frac{w^4}{(w_f^2 - w^2)^2 + 4\zeta_f^2 w_f^2 w^2} \tag{17}$$

where S_0 is the intensity parameter, w_g and ζ_g are the resonant frequency and damping ratio of the first filter, and w_f and ζ_f are those of the second filter.

In this study the soil type is defined as homogeneous medium soil type at the support points of the chosen bridge and the filter parameters for this soil type proposed by Der Kiureghian and Neuenhofer (1991) are utilised as $w_g = 10.0$ rad/s, $\zeta_g = 0.4$, $w_f = 1.0$ rad/s, $\zeta_f = 0.6$. S_0 is obtained for

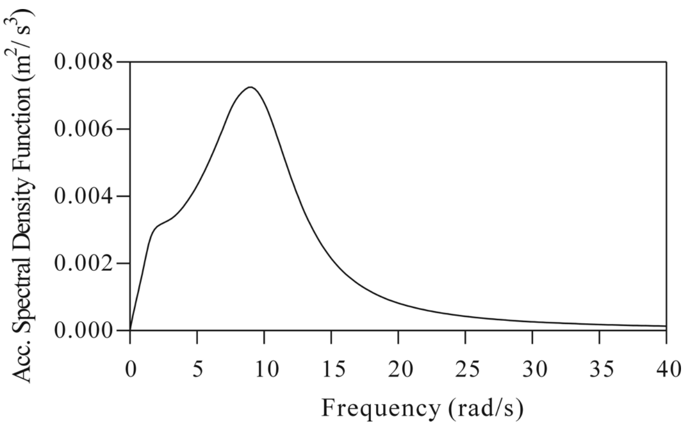

Fig. 1 Power spectral density function of ground motion for medium soil type

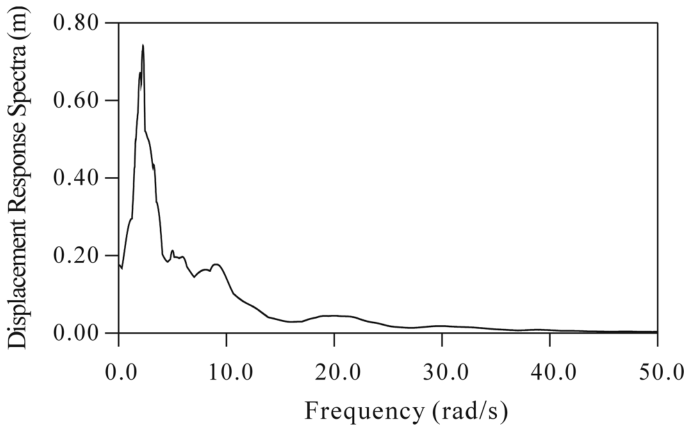

Fig. 2 Displacement response spectra for Erzincan earthquake

the medium soil type by equating the variance of the ground acceleration (Eq. 17) to the variance of east-west component of the Erzincan earthquake acceleration in 1992 in Turkey. The calculated intensity parameter for this soil type is, $S_0 = 0.00263$ m^2/s^3. The acceleration power spectral density function for the medium soil type is presented in Fig. 1.

Der Kiureghian and Neuenhofer (1991) have derived the following relation between the power spectral density of the acceleration process and the displacement response spectrum associated with each support motion for the response spectrum method

$$G_{\ddot{u}_k\ddot{u}_k}(w) = \frac{w^{p+2}}{w^p + w_f^p}\left(\frac{2\zeta w}{\pi} + \frac{4}{\pi\tau}\right)\left[\frac{D_k(w,\zeta)}{p_s(w)_0}\right]^2 \tag{18}$$

In this expression, p and w_f are parameters, τ is the duration of strong motion phase of the motion, ζ is a reference damping ratio, and $p_s(w)_0$ is the peak factor of white noise. In this study the following parameters proposed by Der Kiureghian and Neuenhofer (1991) are used: $p = 3$ and $w_f = 0.705$ rad/s. Although the peak factor for white noise excitation varies for different values of natural frequencies, an average value calculated as 2.78 is used for this study.

In the response spectrum method, the displacement response spectrum (Fig. 2) of the Erzincan earthquake is applied to the bridge model as a vertical ground motion. Although the multiple support response spectrum method is based on the principles of random vibration theory and should properly be used in conjunction with smooth response spectra that characterise an ensemble of time histories, in this study a specific non-smooth response spectra of Erzincan earthquake is used for comparison purposes.

5. Deterministic analysis

Total displacements having two components, namely, the pseudo-static, and the dynamic, can be expressed as Nazmy and Abdel-Ghaffar (1987)

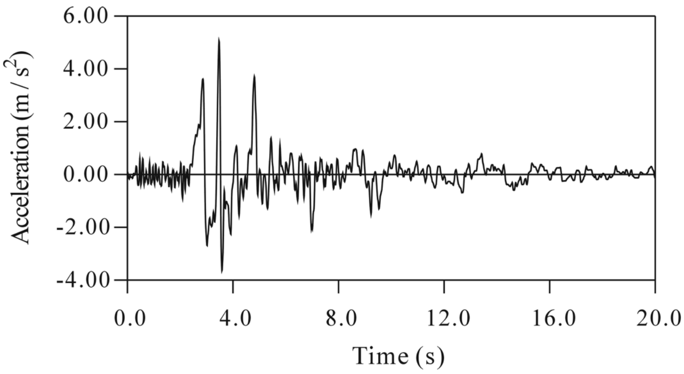

Fig. 3 Acceleration time history for east-west component of Erzincan earthquake

$$v(t) = \sum_j r_j v_{jg}(\tau_j, t) + \sum_i \phi_i Y_i(\tau_j, t) \tag{19}$$

where r_j is the jth displacement shape function due to unit displacement assigned to ground degree of freedom, v_{jg} is the jth ground displacement at the support points, τ_j is the arrival time of the jth ground motion at a specific support point, ϕ_i is the modal vector for mode i and Y_i is the modal amplitude for mode i.

In this study the time history record of the acceleration of the Erzincan earthquake (Fig. 3) is applied to the Jindo Bridge in the vertical direction for the deterministic analysis considering the finite propagation effect of the ground motion.

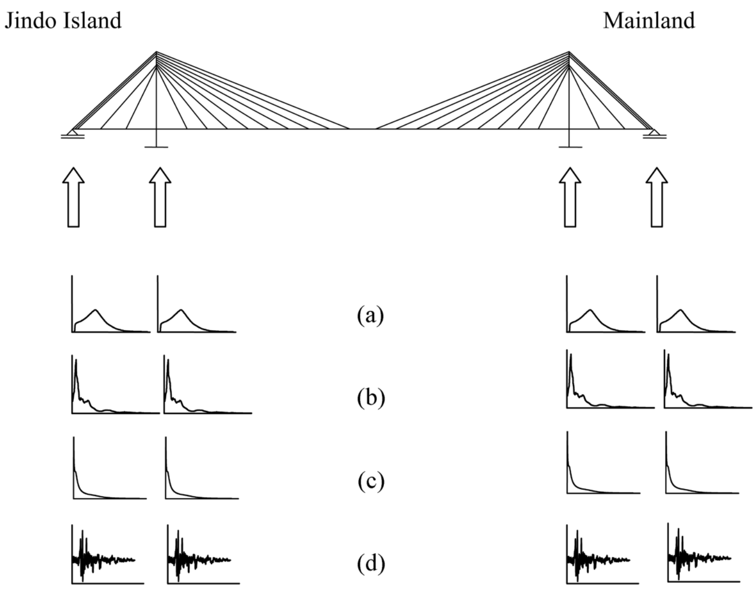

Fig. 4 Bridge model subjected to vertical ground motions (a) Spectral analysis, (b) Response spectrum method, (c) PSDF based response spectrum method, (d) Deterministic method

6. Description of the cable-stayed bridge model

In this study, the Jindo Bridge built in South Korea is chosen as a numerical example. The Jindo bridge has three spans: the main span of 344 m and two side spans of 70 m. The stays are arranged in a fan configuration and converge at the top of the A-frame towers. The stiffening girder and the towers of the Jindo bridge were made from steel. A 2% of damping coefficient is adopted for the response calculations. To investigate the stochastic response of the Jindo bridge, a two-dimensional mathematical model is used for calculations (Fig. 4). The chosen finite element model is represented by 420 degrees of freedom. The stiffening girder and towers are represented by 139 beam elements. The cable stays are modelled with 28 truss elements, and the nonlinearity of the inclined cable stays is considered with equivalent modulus of elasticity.

7. Numerical results

In this study the asynchronous dynamic response of a cable-stayed bridge is investigated by different random vibration methods and a deterministic method. The conducted random vibration methods are as follows:

1. Spectral analysis approach of random vibration theory
2. Response spectrum method based on the relationship between the power spectral density function and the response spectrum of the input ground motion. The response spectrum displacement is derived from the power spectral density function of ground acceleration by using Eq. (18). This method uses the displacement response spectrum shown in Fig. 5 based on the Erzincan earthquake and the power spectral density function (PSDF) of medium soil type. This method will be named as PSDF based response spectrum method throughout the study.
3. Multiple-support response spectrum method based on the response spectrum specification of the ground motion, named as response spectrum method.

The bridge model subjected to asynchronous ground motion in the vertical direction is shown in Fig. 4 for each analysis. It is assumed that the bridge supports are constructed on homogeneous

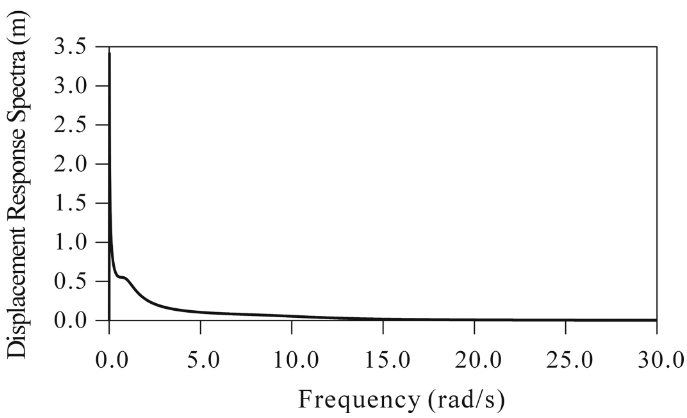

Fig. 5 Displacement response spectra obtained from Eq. (18) for medium soil type

medium soil type and the vertical ground motion is propagating across the bridge from the Jindo Island site to the mainland site with finite velocities, of 300, 600 and 1200 m/s, as well as with infinite wave velocities. The considered earthquake ground motion lasting up to 20.95 seconds is applied to the bridge in the vertical direction with two thirds of the recorded amplitude, either in power spectral density shape or in response spectrum form for random vibration analyses and in the form of time history record for deterministic analysis. In all cases the ground motion is applied to the bridge supports with a time delay as a statement of reasons of the asynchronous dynamic analysis.

7.1 Comparison of random vibration and deterministic responses

Mean of absolute maximum values of responses (Lee and Penzien 1980) at the deck and towers calculated by the spectral analysis approach, the PSDF based response spectrum method and the multiple-support response spectrum method for 600 m/s wave velocity case are compared with each other and with those of the deterministic analysis results. Vertical deck displacements obtained from the random vibration and deterministic methods are compared in Fig. 6. By comparing the displacements obtained from the spectral analysis approach and PSDF based response spectrum method, it is observed that the displacements are very close to each other. The difference of the displacements obtained by these two methods are smaller than 10% at the center span and at the level of 15% at the side spans where displacements are relatively small. While the response spectrum method based on the response spectrum specification of the input motions provide generally much larger response values than those of the former two random vibration methods at the center span, it takes the smallest values at the side spans. The displacement value calculated at the

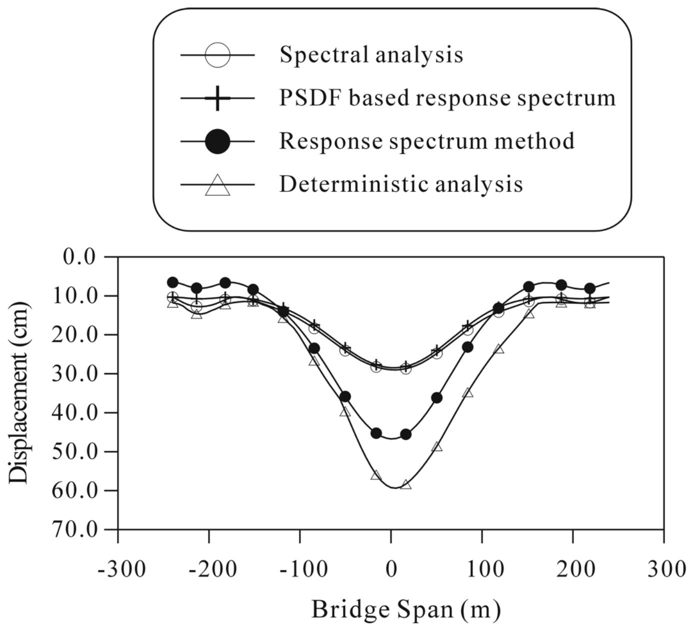

Fig. 6 Maximum vertical deck displacements for 600 m/s wave propagation velocity

middle of the deck by response spectrum method is 61% larger than those of the spectral analysis approach. The discrepancy between the spectral analysis approach and the response spectrum method is caused by the difference of the variances of ground accelerations described by Eq. (17) for spectral analysis approach and by Eq. (18) for response spectrum method. Although the same earthquake ground motion is used for both methods, the power spectral density functions of ground accelerations characterising the ground motions are different in character. In spectral analysis approach the intensity parameter is obtained by equalising the variance of Erzincan earthquake acceleration to the variance of ground motion model described by Eq. (17). But in the response spectrum method, the displacement response spectrum of Erzincan earthquake is obtained and substituted into Eq. (18). Therefore, for the considered ground motion the variance of the ground acceleration in the response spectrum method gives much larger variance (0.3215 m^2/sec^4) than those of the spectral analysis approach (0.083308 m^2/sec^4). Because mean of absolute maximum values for both methods depend largely on the intensity and frequency contents of the power spectral density function, it is natural to have much larger response values for the response spectrum method. On the other hand, the response values obtained by the deterministic method where the displacements are absolute maximum values, are the largest both at the center and side spans.

The horizontal displacements of the tower located at the Jindo Island site, are also compared for random vibration and deterministic methods (Fig. 7). The displacement values obtained by the spectral analysis approach and the PSDF based response spectrum method are close to each other. The spectral analysis approach causes 10% larger displacement value at the top of the tower compared to those of the PSDF based response spectrum method. The displacement values obtained from the response spectrum method are larger than those of the former random vibration analyses

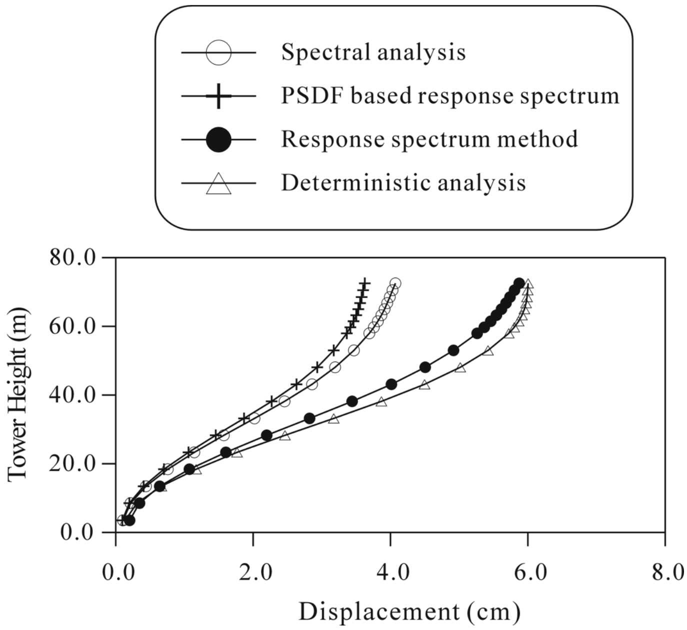

Fig. 7 Maximum horizontal displacements of the island tower for 600 m/s wave propagation velocity

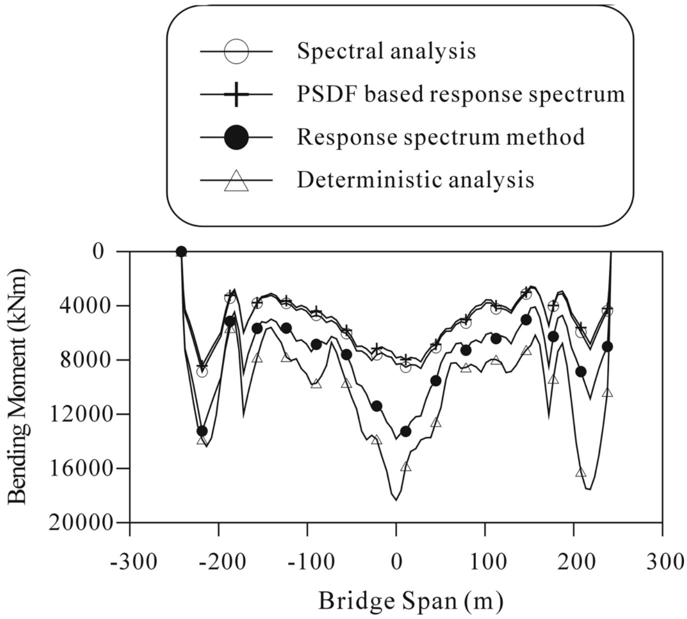

Fig. 8 Maximum deck bending moments for 600 m/s wave propagation velocity

results, and the difference between the response spectrum method and spectral analysis approach are approximately of the order of 44% at the tower top point. Although the response values obtained from the deterministic method are the largest, they are not far away from the response spectrum analysis results. The variation obtained for the center span vertical deck displacements and horizontal tower displacements can also be observed for the deck bending moments (Fig. 8). The largest difference between the bending moments obtained by the spectral analysis approach and the PSDF based response spectrum is 8%.

7.2 Effect of ground motion wave velocity

Figs. 9-11 compare the deck vertical displacements for V = 300, 600, 1200 m/s and infinite ground motion wave velocity cases calculated by the spectral analysis approach, the response spectrum method and the deterministic method, respectively. It can be observed that, as expected, the displacements generally decrease with increasing wave velocities for all analysis cases; however, this is especially true for the center span displacements. Opposed to the expectations, the displacements at the middle of the deck obtained for 300 m/s wave velocity case do not the cause largest displacement values. Therefore, although the variation of the results for random vibration and deterministic analyses depend mostly on the finite propagation effect of the ground motion, some other parameters can also effect the variation. At the side spans of the bridge where small displacements take place, the analyses produce very close displacement values.

The horizontal tower displacements obtained by the spectral analysis approach are compared in Fig. 12 for different wave velocity cases. Also in this figure the displacements decrease with increasing wave velocities. As the infinite wave velocity case induces the smallest displacement values, the 300 m/s finite wave velocity case causes the largest displacements. The displacement

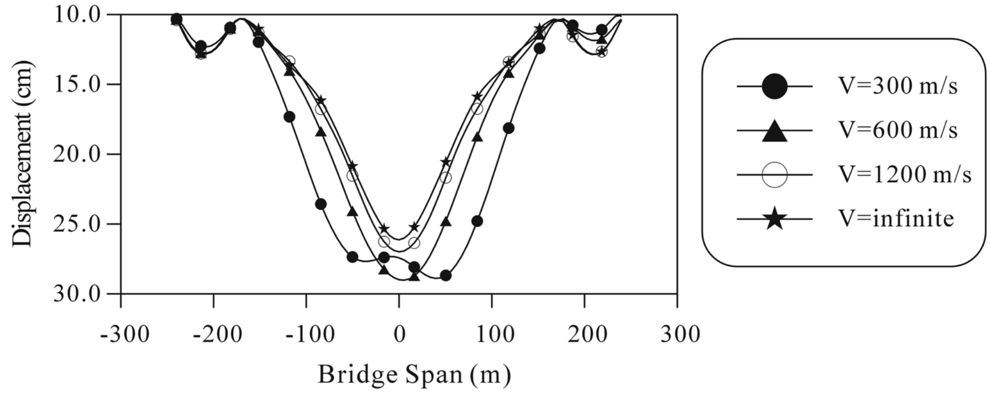

Fig. 9 Maximum vertical deck displacements by spectral analysis approach

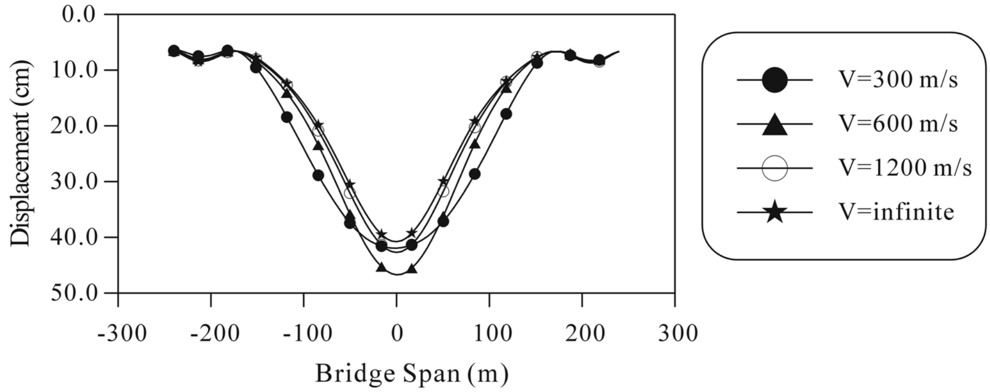

Fig. 10 Maximum vertical deck displacements by response spectrum method

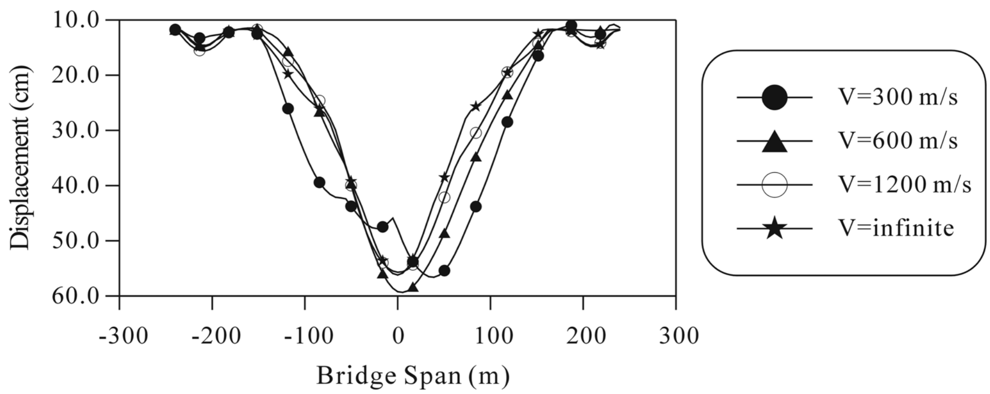

Fig. 11 Maximum vertical deck displacements by deterministic method

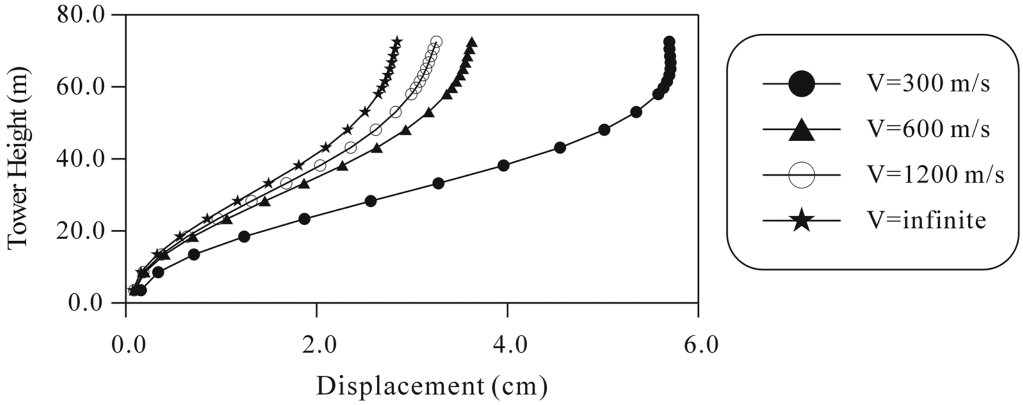

Fig. 12 Maximum horizontal displacements of the island tower by spectral analysis approach

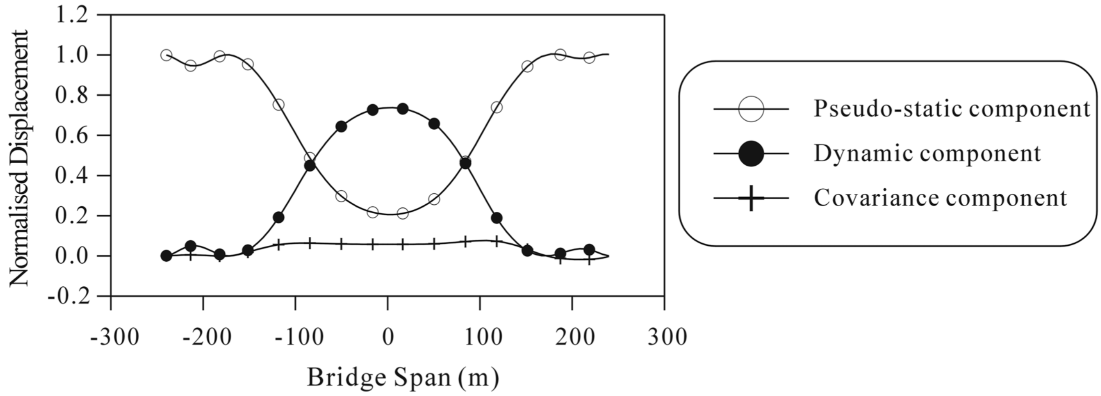

Fig. 13 Normalised displacement variances of the deck by spectral analysis approach

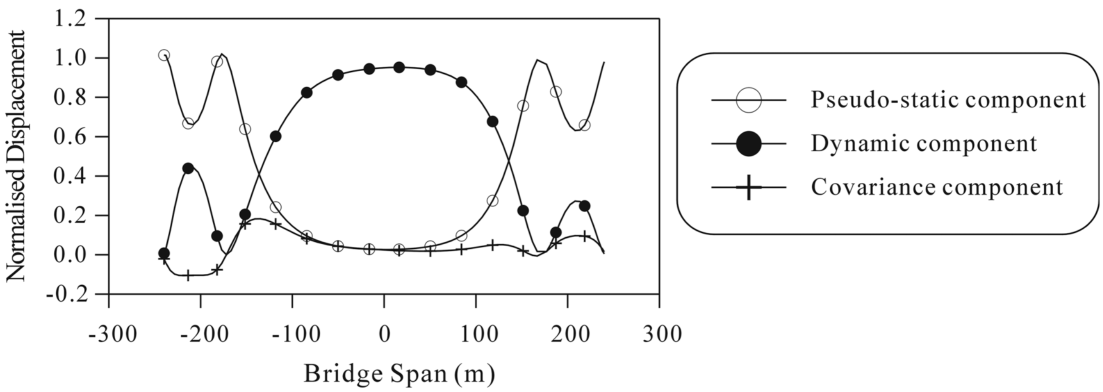

Fig. 14 Normalised displacement variances of the deck by response spectrum method

value obtained at the top of the tower for 300 m/s wave propagation velocity case is 57%, 75% and 100% larger than those of the 600 m/s, 1200 m/s and infinite wave velocity cases, respectively. This increment is mainly caused by the pseudo-static displacements which take place by the differential excitations at the support points of the bridge. As the wave velocity of the ground motion decreases, the pseudo-static component becomes more important.

7.3 Relative contributions of pseudo-static, dynamic, and cross terms

As pointed out variance of total response has three components: the pseudo-static component, the dynamic component and the covariance component between the pseudo-static and dynamic components. The contribution of each component to the total responses of the bridge is investigated in this section for the spectral analysis approach and the response spectrum method due to the 600 m/s ground motion wave velocity case.

The relative contributions of these components to the total vertical displacement responses along the bridge deck are presented in Figs. 13-14. The normalisation is performed by dividing the response components to the total response value at each structural node. While the displacements

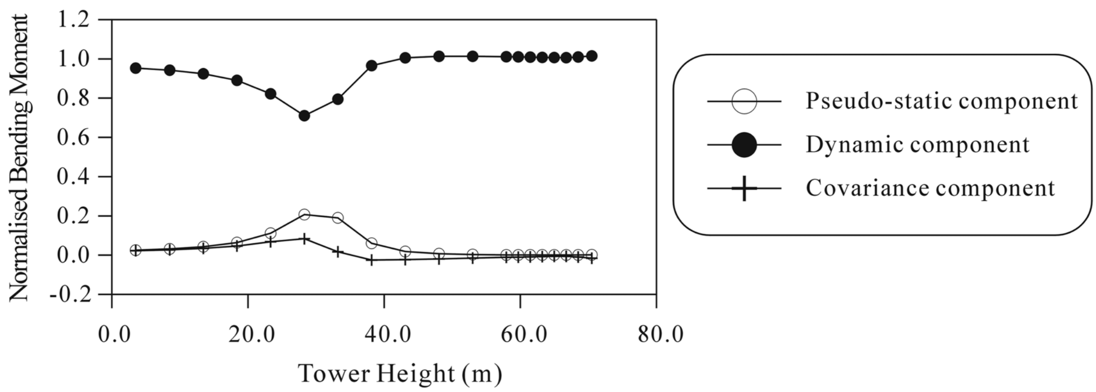

Fig. 15 Normalised bending moment variances of the island tower by spectral analysis approach

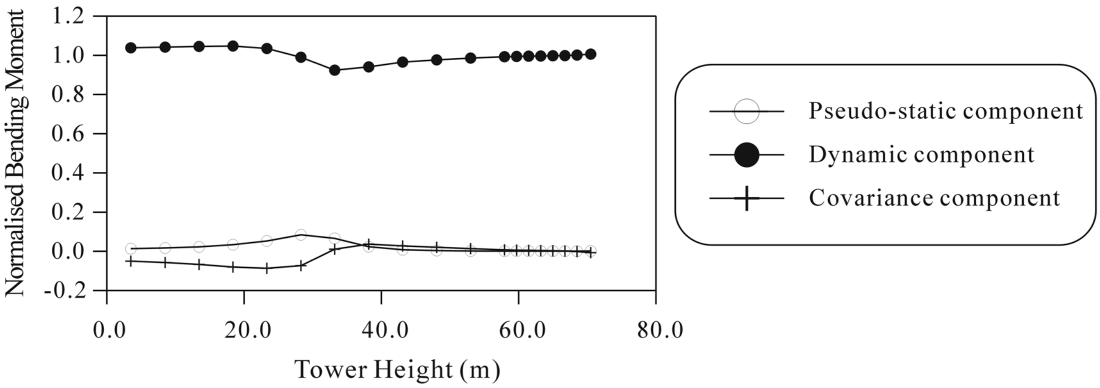

Fig. 16 Normalised bending moment variances of the island tower by response spectrum method

obtained at the side spans are dominated by the pseudo-static component, the displacements obtained at the center span are dominated by the dynamic component for both analyses. The contribution of the covariance component to the total response is very small along the bridge length. At the middle of the deck, where generally maximum total displacements take place, it can be observed that the dynamic component contributes 74%, the pseudo-static component contributes 21%, and the covariance component contributes 5% for spectral analysis approach. The ratios obtained for the response spectrum method are 95%, 3%, and 2%, respectively.

The normalised tower bending moment response components are presented in Figs. 15, 16 for both analyses. As the total bending moments are mostly dominated by the dynamic component, the pseudo-static component has also important contributions at some parts of the tower. At the tower point where the pseudo-static component has maximum contribution, the contributions of dynamic, pseudo-static and covariance components are 71%, 21% and 8%, respectively for the spectral analysis approach. The ratios obtained for the response spectrum analysis are 92%, 7% and 1%, respectively.

8. Conclusions

A comparative analysis is performed by a frequency domain spectral analysis, two response spectrum methods and a deterministic method on a cable-stayed bridge model. The main findings from this study can be categorised as follows:

Responses obtained by the frequency domain spectral analysis are very close to the results obtained by the power spectral density function based response spectrum method, mostly of the order of 10%.

As the results obtained for random vibration analyses are mean of absolute maximum values, and the deterministic method yields absolute maximum values, the results obtained by the deterministic method are generally larger than the results obtained by random vibration methods.

The multiple-support response spectrum method based on the response spectrum specification of the input motion causes generally larger response values than those of the other two random vibration methods. This discrepancy is caused by the large difference of the variances of the ground accelerations calculated for the spectral analysis approach and the response spectrum method. In spite of the fact that there is a significant discrepancy between random vibration and deterministic results, the structural responses show consistent trends.

The structural response values usually show important amplifications depending on the decreasing ground motion wave velocities.

As the responses are mostly dominated by the dynamic component, the pseudo-static component has also significant contributions at some parts of the bridge especially for displacement responses because of the flexible nature of the bridge. The contribution of the covariance component, which can also take negative values, is generally small.

References

Abdel-Ghaffar, A.M. and Rubin, L.I. (1982), “Suspension bridge response to multiple-support excitations”, *J. Eng. Mech.*, **108**, 419-435.

Alkhaleefi, A.M. and Ali, A. (2002), “An efficient multi-point support-motion random vibration analysis technique”, *Comput. Struct.*, **80**, 1689-1697.
Allam, S.M. and Datta, T.K. (1999), “Seismic behaviour of cable-stayed bridges under multi-component random ground motion”, *Eng. Struct.*, **22**, 62-74.
Allam, S.M. and Datta, T.K. (2000), “Analysis of cable-stayed bridges under multi-component random ground motion by response spectrum method”, *Eng. Struct.*, **22**, 1367-1377.
Clough, R.W. and Penzien, J. (1993), *Dynamics of Structures*, Second Edition, McGraw Hill, Inc., Singapore.
Der Kiureghian, A. and Neuenhofer, A. (1991), “A response spectrum method for multiple-support seismic excitations”, Report No. UCB/EERC-91/08, Berkeley (CA), Earthquake Engineering Research Center, College of Engineering, University of California.
Der Kiureghian, A. (1996), “A coherency model for spatially varying ground motions”, *Earthq. Eng. Struct. Dyn.*, **25**, 99-111.
Der Kiureghian, A., Keshishian, P. and Hakobian, A. (1997), “Multiple support response spectrum analysis of bridges including the site-response effect and MSRS code”, Report No. UCB/EERC-97/02, Berkeley (CA), Earthquake Engineering Research Center, College of Engineering, University of California.
Garevski, M., Dumanoglu, A.A. and Severn, R.T. (1988), “Dynamic characteristics and seismic behaviour of Jindo bridge, South Korea”, *Structural Engineering Review*, **1**, 141-149.
Hao, H. (1993), “Arch responses to correlated multiple excitations”, *Earthq. Eng. Struct. Dyn.*, **22**, 389-404.
Harichandran, R.S. and Wang, W. (1988), “Response of one- and two-span beams to spatially varying seismic excitation”, Report to the National Science Foundation MSU-ENGR-88-002, Michigan State University, Michigan.
Harichandran, R.S., Hawwari, A. and Sweiden, B.N. (1996), “Response of long-span bridges to spatially varying ground motion”, *J. Struct. Eng.*, **122**(5), 476-484.
Lee, M.C. and Penzien, J. (1980), “Stochastic seismic analysis of nuclear power plant structures and piping systems subjected to multiple support excitations”, Report No. UCB/EERC-80/19, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, CA.
Monti, G., Nuti, C. and Pinto, P.E. (1996), “Nonlinear response of bridges under multisupport excitation”, *J. Struct. Eng.*, **122**(10), 1147-1159.
Nakamura, Y., Der Kiureghian, A. and Liu, D. (1993), “Multiple-support response spectrum analysis of the Golden Gate bridge”, Report No. UCB/EERC-93/05, Berkeley (CA), Earthquake Engineering Research Center, College of Engineering, University of California.
Nazmy, A.S. and Abdel-Ghaffar, A.M. (1987), “Seismic response analysis of cable stayed bridges subjected to uniform and multiple-support excitations”, Report No. 87-SM-1, Department of Civil Engineering, Princeton University, Princeton (NJ).
Nazmy, A.S. and Abdel-Ghaffar, A.M. (1992), “Effects of ground motion spatial variability on the response of cable-stayed bridges”, *Earthq. Eng. Struct. Dyn.*, **21**, 1-20.
Schemmann, A.G. (1997), “Modeling and active control of cable-stayed bridges subject to multiple-support excitation”, Ph. D. Thesis, Stanford University, California.
Soyluk, K. and Dumanoglu, A.A. (2000), “Comparison of asynchronous and stochastic dynamic response of a cable-stayed bridge”, *Eng. Struct.*, **22**, 435-445.
Zembaty, Z. (1997), “Vibrations of bridge structure under kinematic wave excitations”, *J. Struct. Eng.*, **123**(4), 479-487.
Zerva, A. (1991), “Effect of spatial variability and propagation of seismic ground motions on the response of multiply supported structures”, *Probabilistic Engineering Mechanics*, **6**, 212-221.