Steel and Composite Structures, *Vol. 22*, *No. 5* (*2016*) *1001-1018*
DOI: http://dx.doi.org/10.12989/scs.2016.22.5.1001

Stochastic response of suspension bridges for various spatial variability models

Süleyman Adanur [1], Ahmet C. Altunışık [*1], Kurtuluş Soyluk [2] and A. Aydın Dumanoğlu [3a]

[1] *Department of Civil Engineering*, *Karadeniz Technical University*, *Trabzon*, *Turkey*
[2] *Department of Civil Engineering*, *Gazi University*, *Ankara*, *Turkey*
[3] *Karadeniz Technical University*, *Trabzon*, *Turkey*

(*Received September 02*, *2016*, *Revised October 07*, *2016*, *Accepted November 03*, *2016*)

Abstract. The purpose of this paper is to compare the structural responses obtained from the stochastic analysis of a suspension bridge subjected to uniform and partially correlated seismic ground motions, using different spatial correlation functions commonly used in the earthquake engineering. The spatial correlation function employed in this study consists of a term that characterizes the loss of coherency. To account for the spatial variability of ground motions, the widely used four loss of coherency models in the literature has been taken into account in this study. Because each of these models has its own characteristics, it is intended to determine the sensitivity of a suspension bridge due to these losses of coherency models which represent the spatial variability of ground motions. Bosporus Suspension Bridge connects Europe to Asia in Istanbul is selected as a numerical example. The bridge has steel towers that are flexible, inclined hangers and a steel box-deck of 1074 m main span, with side spans of 231 and 255 m on the European and Asian sides, respectively. For the ground motion the filtered white noise model is considered and applied in the vertical direction, the intensity parameter of this model is obtained by using the S16E component of Pacoima Dam record of 1971 San Fernando earthquake. An analytically simple model called as filtered white noise ground motion model is chosen to represent the earthquake ground motion. When compared with the uniform ground motion case, the results obtained from the spatial variability models with partial correlation outline the necessity to include the spatial variability of ground motions in the stochastic dynamic analysis of suspension bridges. It is observed that while the largest response values are obtained for the model proposed by Harichandran and Vanmarcke, the model proposed by Uscinski produces the smallest responses among the considered partially correlated ground motion models. The response values obtained from the uniform ground motion case are usually smaller than those of the responses obtained from the partially correlated ground motion cases. While the response values at the flexible parts of the bridge are totally dominated by the dynamic component, the pseudo-static component also has significant contributions for the response values at the rigid parts of the bridge. The results also show the consistency of the spatial variability models, which have different characteristics, considered in this study.

Keywords: coherency; spatial variability; geometric nonlinearity; stochastic response; suspension bridge

*Corresponding author, Associate Professor, E-mail: ahmetcan8284@hotmail.com
[a] Emeritus Professor

http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

1. Introduction

Because dynamic loads like earthquake motions are random, there is a need to a process, which takes into account the uncertainty of the dynamic loading in the analysis. The analysis due to the random loading is defined as stochastic analysis and should be considered in the earthquake response analysis of structural systems if a realistic analysis is desired.

Suspension bridges consist of elements like tower, cable, hanger and deck; the behaviour of each one is different. Under the effect of external forces, especially cables and hangers are subjected to large tensional forces and these forces have important effects on the element stiffness matrices. This characteristic, called as geometric nonlinearity of the structural elements, should be taken into account in the analysis of suspension bridges.

Support points of long-span suspension bridges will be subjected to differential ground motions during the spatially varying earthquake ground motions, because of their extended lengths. One component considering the spatial variability of the ground motion is the loss of coherency (incoherence effect), which results from the reflections and refractions of seismic waves through the soil layers during their propagation.

The effects of spatial variability of ground motions between the supports of lifeline structures have drawn the attention of researchers, and various structural configurations subjected to stochastic seismic ground motions have been analysed. In a previously conducted study, Button *et al*. (1981) developed a computer programme called STOCAL for the stochastic seismic analysis of structural systems. This program is capable of analysing structural systems subjected to uniform ground motions. Adanur *et al*. (2003) investigated the stochastic analysis of suspension bridges for different correlation functions. One of the leading studies considering the effect of the multiple-support excitation on long span bridges was performed by Soyluk (2004). Dumanoglu and Severn (1990), Bryja (2009) analysed the stochastic response of suspension bridges to earthquake ground motions. These studies ignored the spatial variability of ground motions at the support points and uniform ground motion model were assumed. The effect of the spatially varying ground motions on the random vibration responses of lifeline structures has been of considerable concern in the last decade (Chen and Harichandran 2001, Soyluk and Dumanoglu 2004, Ateş *et al*. 2006, Zhang *et al*. 2009, Bi and Hao 2013, Li and Chouw 2014, Shrestha *et al*. 2014, Dong *et al*. 2015). All these studies mostly performed on long span bridges underline the importance of the spatial variability of earthquake ground motions. The nonlinear deterministic analyses of suspension bridges to uniform dynamic effects were also carried out over the past two decades (Brownjohn 1994). Also, some paper recently published about the construction stage, probabilistic sensitivity, reliability analysis and cable tension force estimation of suspension bridges (Atmaca and Ateş 2012, Cheng and Liu 2012, Cavdar 2013, Günaydin *et al*. 2014, Wang *et al*. 2015, Adanur *et al*. 2016).

In this study, stochastic response of a suspension bridge subjected to the partially correlated seismic ground motions is carried out considering the geometric nonlinearity. The spatial correlation function is represented by a term that characterizes the loss of coherency. To account for the spatial variability of ground motions, the widely used four loss of coherency models in the literature has been considered in this study. These models are proposed by Harichandran and Vanmarcke (1986), Abrahamson (1993), Hindy and Novak (1980) and Uscinski (1977). Each of these models has its own characteristics. As Harichandran and Vanmarcke's model is based on the recordings of SMART-1 array, Hindy and Novak as well as Uscinski have used loss of coherency models of the form of exponential functions. Some other loss of coherency models like

Abrahamson's model seems to be more complex. Due to remarkable differences of these models, it is intended to determine the sensitivity of a suspension bridge due to this loss of coherency models representing the spatial variability of ground motions. As a numerical example Bosporus Suspension Bridge built in Istanbul is chosen. Filtered white noise ground motion model modified by Clough and Penzien (1993) is used in this study as ground motion model. The intensity parameter for filtered white noise model is obtained by equating the variance of this model to the variance of the two thirds of the S16E component of Pacoima Dam record of 1971 San Fernando earthquake.

2. Formulation

2.1 Stochastic analysis

2.1.1 Pseudo-static response component

Pseudo-static component of spectral density function can be expressed as

$$S_z^{qs}(\omega) = \frac{1}{\omega^4}\sum_{l=1}^{r}\sum_{m=1}^{r} A_l A_m S_{lm}(\omega) \tag{1}$$

where r is the number of support degrees-of-freedom, A_l and A_m are static displacements for unit displacements assigned to each support point and $S_{lm}(\omega)$ is the cross spectral density function of accelerations between supports l and m.

The variance of the pseudo-static response component is obtained in the following form

$$\sigma^2{}_z^{qs} = \int_{-\infty}^{\infty} S_z^{qs}(\omega)d\omega = \sum_{l=1}^{r}\sum_{m=1}^{r} A_l A_m \int_{-\infty}^{\infty} \frac{1}{\omega^4} S_{lm}(\omega)d\omega \tag{2}$$

2.1.2 Dynamic response component

Dynamic component of spectral density function in the case of spatially varying ground motion can be written as

$$S_z^{d}(\omega) = \sum_{j=1}^{n}\sum_{k=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_j \psi_k \Gamma_{lj}\Gamma_{mk} H_j(-\omega) H_k(\omega) S_{lm}(\omega) \tag{3}$$

where n is the number of modes, $[\psi]$ is the eigenvectors, $[\Gamma]$ is the modal participation factor and $H(\omega)$ is the frequency response function.

The variance of the dynamic response component will be

$$\sigma^2{}_z^{d} = \int_{-\infty}^{\infty} S_z^{d}(\omega)d\omega = \sum_{j=1}^{n}\sum_{k=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_j \psi_k \Gamma_{lj}\Gamma_{mk} \int_{-\infty}^{\infty} H_j(-\omega) H_k(\omega) S_{lm}(\omega)d\omega \tag{4}$$

2.1.3 Covariance of pseudo-static and dynamic response component

Cross-spectral density function of pseudo-static and dynamic components is defined as

$$S_{z\;z}^{s\;d}(\omega) = -\frac{1}{\omega^2}\sum_{j=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r}\psi_j A_l \Gamma_{mj} H_j(\omega) S_{lm}(\omega) \tag{5}$$

The covariance between the pseudo-static and dynamic components is therefore

$$Cov(z^{qs}, z^d) = \mathrm{Re}\left[\int_{-\infty}^{\infty} S_{z\;z}^{qs\;d}(\omega)d\omega\right] = \sum_{j=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r}\psi_j A_l \Gamma_{mj}\left(-\int_{-\infty}^{\infty}\frac{1}{\omega^2}H_j(\omega)S_{lm}(\omega)d\omega\right) \tag{6}$$

2.1.4 Total response

The variance of total response can be computed by the addition of the pseudo-static, dynamic and covariance response components as

$$\sigma_z^2 = \sigma^{2\,qs}_{\;z} + \sigma^{2\,d}_{\;z} + 2Cov(z^{qs}, z^d) = \int_{-\infty}^{\infty} S_z^{qs}(\omega)d\omega + \int_{-\infty}^{\infty} S_z^d(\omega)d\omega + 2\,\mathrm{Re}\left[\int_{-\infty}^{\infty} S_{z\;z}^{qs\;d}(\omega)d\omega\right] \tag{7}$$

2.1.5 Mean of maximum value

The mean of maximum value in stochastic analysis is given by

$$\mu = p\sigma_z \tag{8}$$

where p is a peak factor and σ_z is standard deviation of the total response.

2.2 Stochastic analysis

The cross spectral density function of the accelerations at two support points l and m is written as

$$S_{lm}(\omega) = S(\omega)\left|\gamma_{lm}(\omega)\right| \tag{9}$$

where ω is the circular frequency, $S(\omega)$ is the power spectral density function of ground acceleration and $\gamma_{lm}(\omega)$ is the coherency function (Zerva 1992).

The power spectral density function of ground acceleration related to the filtered white noise ground motion model modified by Clough and Penzien is in the following form

$$S(\omega) = S_0\left[\frac{\omega_g^4 + 4\xi_g^2\omega_g^2\omega^2}{(\omega_g^2-\omega^2)^2 + 4\xi_g^2\omega_g^2\omega^2}\right]\left[\frac{\omega^4}{(\omega_f^2-\omega^2)^2 + 4\xi_f^2\omega_f^2\omega^2}\right] \tag{10}$$

where S_0 is an intensity factor; ω_g, ξ_g and ω_f, ξ_f are the first and second filter natural frequencies and damping ratios, respectively. In this study, firm soil type is used and as filter parameters the values proposed by Der Kiureghian and Neuenhofer (1991) are utilized: ω_g = 15.0 rad/s, ξ_g = 0.6, ω_f = 1.5 rad/s, ξ_f = 0.6. S_0 is obtained for the firm soil type by equating the variance of the filtered white noise ground motion model to the variance of S16E component of Pacoima Dam record of

1971 San Fernando earthquake acceleration. The calculated value of the intensity parameter for firm soil type is S_0(firm) = 0.009715 m²/s³. The S16E component of Pacoima Dam record of 1971 San Fernando earthquake acceleration, the corresponding power spectral density function of the earthquake motion and the acceleration power spectral density function of the filtered white noise ground motion model for firm soil type are presented in Fig. 1.

The analyses included in this study are performed for four different spatial correlation functions (loss of coherency) as described below (Chen and Harichandran 2001).

- Harichandran and Vanmarcke (1986) proposed a model, called as Model 1 in this study, based on the analysis of recordings made by the SMART-1 seismograph array in Lotung, Taiwan and defined as

$$\left|\gamma_{lm}(\omega)\right| = A\exp\left[-\frac{2d_{lm}}{\alpha\theta(\omega)}(1-A+\alpha A)\right]+(1-A)\exp\left[-\frac{2d_{lm}}{\theta(\omega)}(1-A+\alpha A)\right] \tag{11}$$

where d_{lm} is the distance between support points l and m, $\theta(\omega)$ is frequency-dependent spatial scale of fluctuation

$$\theta(\omega)=k\left[1+\left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-\frac{1}{2}} \tag{12}$$

where A, α, k, f_0 and b are model parameters. In this study, the values obtained by Harichandran *et al*. (1996) are used (A = 0.636, α = 0.0186, k = 31200, f_0 = 1.51 Hz and b = 2.95)

- Abrahamson (1993) proposed a relatively complex model, defined as Model 2, based on the earthquake events recorded in California and Taiwan, and given by

$$\left|\gamma_{lm}(\omega)\right| = \left|\gamma_{lm}(\omega)\right|^1 h_{lm}(\omega) \tag{13}$$

$$\left|\gamma_{lm}(\omega)\right|^1=\tanh\left\{\frac{c_3(d_{lm})}{1+c_4(d_{lm})\left(\frac{\omega}{2\pi}\right)}+\left[4.65-c_3(d_{lm})\right]\exp\left[c_6(d_{lm})\left(\frac{\omega}{2\pi}\right)\right]+0.35\right\} \tag{14}$$

$$c_3(d_{lm})=3.5-0.37\ln(d_{lm}+0.04) \tag{15}$$

$$c_4(d_{lm})=0.65\left[1-\frac{1}{1+\frac{d_{lm}}{4}}\right] \tag{16}$$

$$c_6(d_{lm}) = 3[\exp(-0.05d_{lm}) - 1] - 0.0018d_{lm} \tag{17}$$

where $h_{lm}(\omega)$ is the plane wave factor

$$h_{lm}(\omega) = \frac{1}{1 + \left[\dfrac{\left(\dfrac{\omega}{2\pi}\right)}{c_8(d_{lm})} \right]^3} \tag{18}$$

$$c_8(d_{lm}) = \exp[5.20 - 0.634\ln(d_{lm} + 0.1)] \tag{19}$$

- Hindy and Novak (1980) proposed a model, called as Model 3, is expressed as

$$|\gamma_{lm}(\omega)| = \exp\left[-\alpha(\omega d_{lm})^{\beta}\right] \tag{20}$$

where α and β are model parameters and considered as $\alpha = 3.007 \times 10^{-4}$ and $\beta = 0.9$.

- Uscinski (1977) proposed a model, defined as Model 4, based on wave propagation in random media and expressed as

$$|\gamma_{lm}(\omega)| = \exp\left[-\left(\frac{\alpha d_{lm}\omega}{v_s}\right)^2\right] \tag{21}$$

where α is a spatial incoherence parameter and v_s is the shear wave velocity. Luco and Wong (1986) used this model in a civil engineering context and in this study the value of $(\alpha/v_s) = 2 \times 10^{-4}$ s/m is employed.

The variations of spatial variability (loss of coherency) models depending on the frequency are shown in Figs. 2 through 5. While full correlation is observed for Model 2 through Model 4 at low frequencies, Model 1 produces partial correlation even at zero frequency. It can also be observed that while Model 4 rapidly approaches to zero at relatively small frequencies and large separation distances, the coherency still exists for Model 1 even at higher frequencies. From these figures, it is obvious that all the considered loss of coherency models tend to decrease with increasing frequencies and separation distances between the support points.

2.3 Geometric nonlinear behaviour

In geometric nonlinear behaviour, total stiffness matrix of the system is obtained as

$$K = K_E + K_G \tag{22}$$

where K_E and K_G are the elastic and the geometric stiffness matrices, respectively.

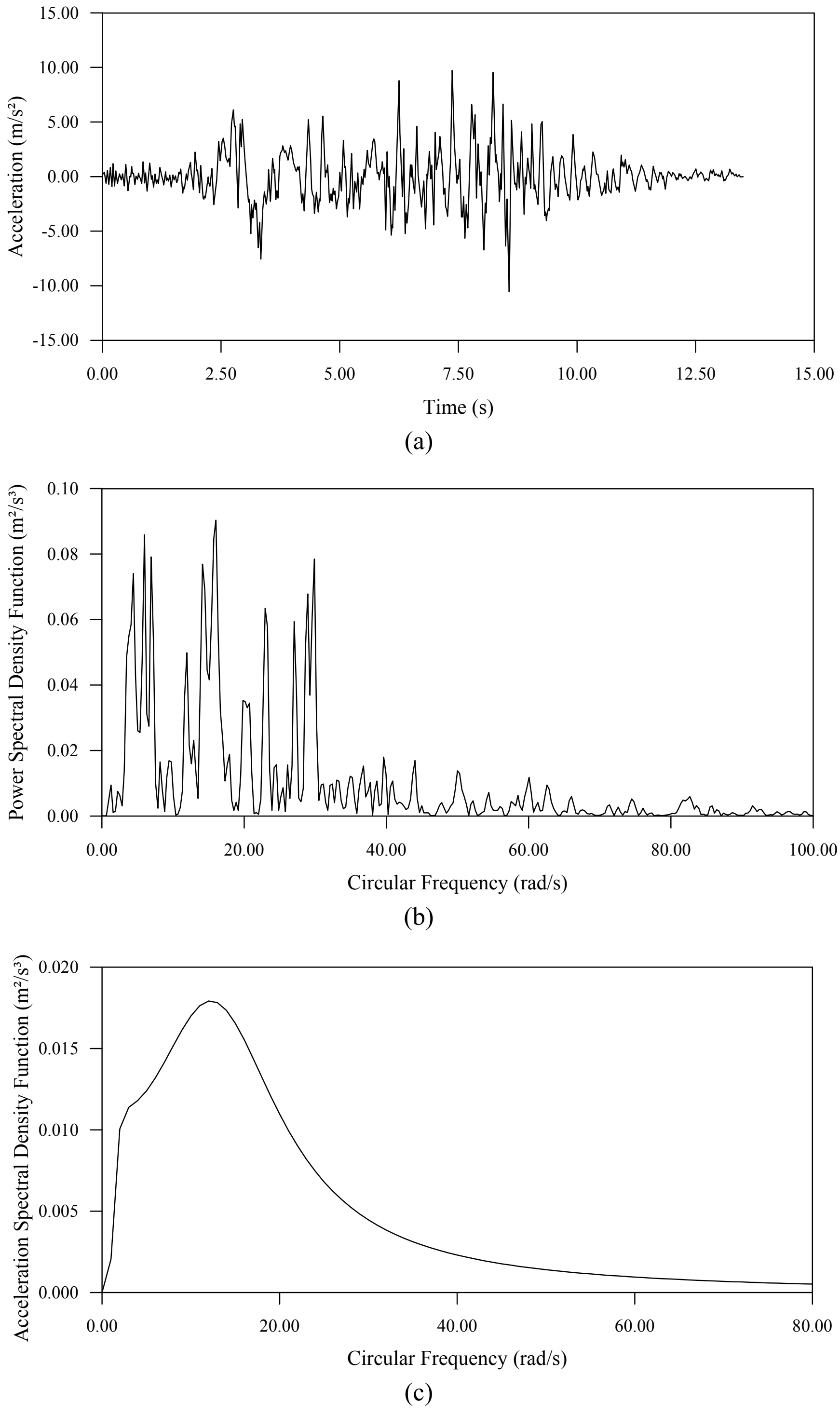

Fig. 1 (a) Acceleration time history of S16E component of Pacoima Dam record; (b) Acceleration power spectral density function of Pacoima Dam record; (c) Acceleration power spectral density function of filtered white noise model

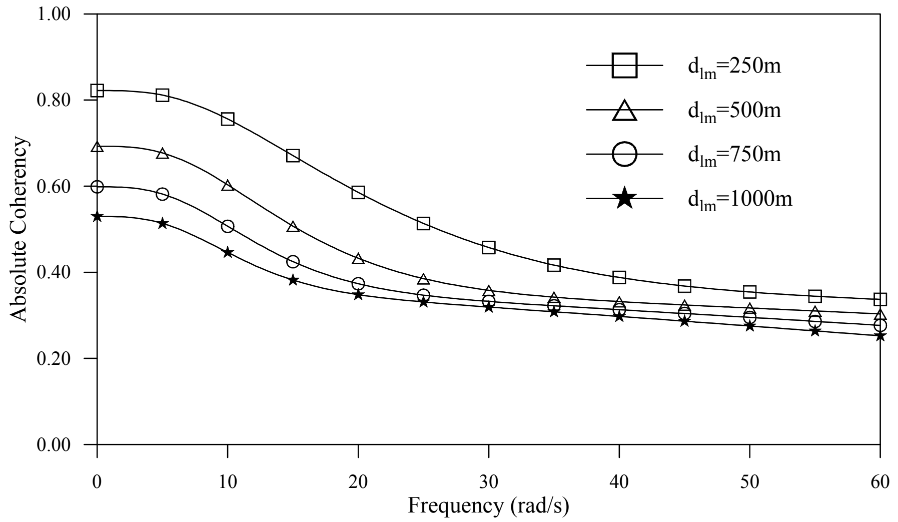

Fig. 2 Harichandran and Vanmarcke's coherency model (Model 1)

3. Bosporus suspension bridge

In this study, the Bosporus Suspension Bridge built in Turkey and connects Europe to Asia in Istanbul is chosen as an example (Fig. 6). The bridge opened to the traffic in 1973 has steel towers that are flexible, inclined hangers and a steel box-deck of 1074 m main span, with side spans of 231 and 255 m on the European and Asian sides, respectively. The horizontal distance between the cables is 28 m and the roadway is 21 m wide, accommodating three lanes each way. The roadway at the mid-span of the bridge is approximately 64 m above the sea level. Schematic representation of Bosphorus Suspension Bridge including the dimensions is given in Fig. 7.

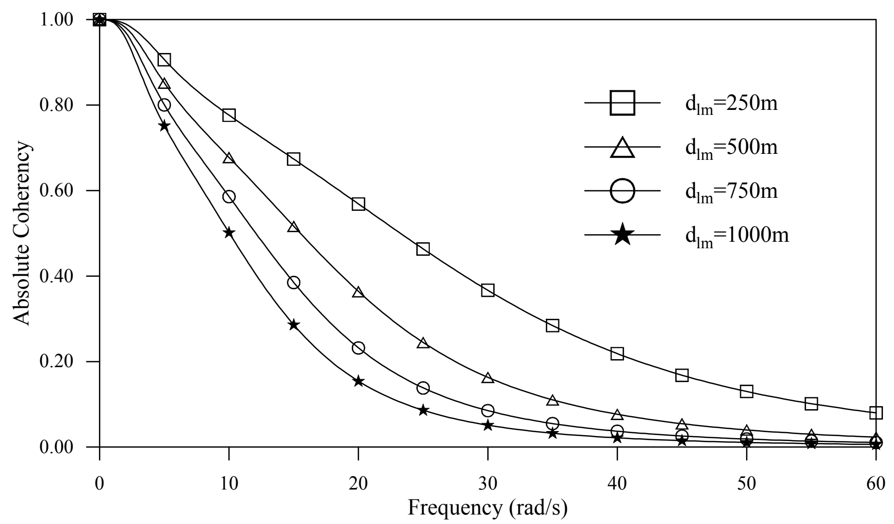

Fig. 3 Abrahamson's coherency model (Model 2)

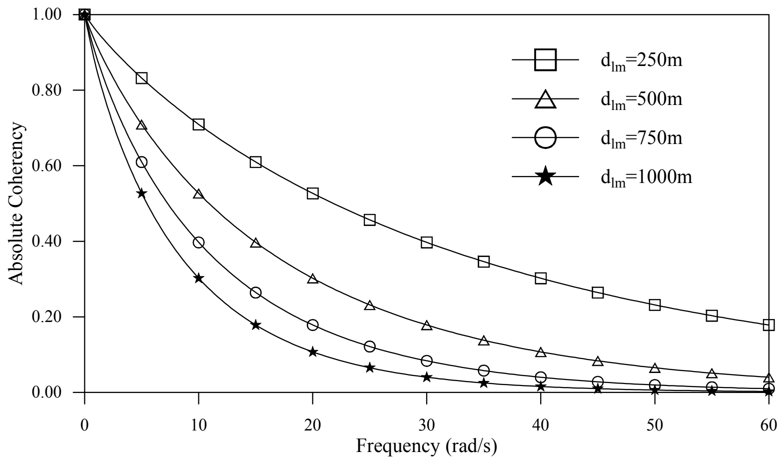

Fig. 4 Hindy and Novak's coherency model (Model 3)

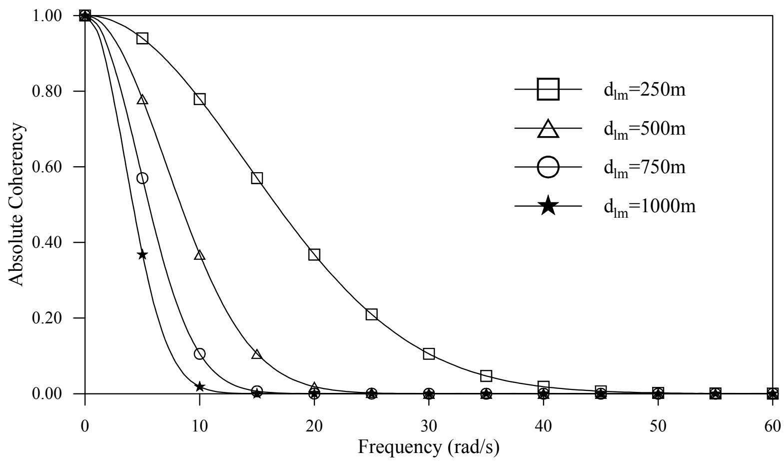

Fig. 5 Uscinski's coherency model (Model 4)

Fig. 6 Bosphorus Suspension Bridge

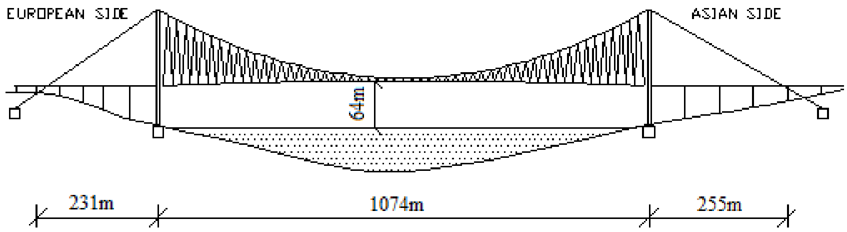

Fig. 7 Schematic representation including dimension

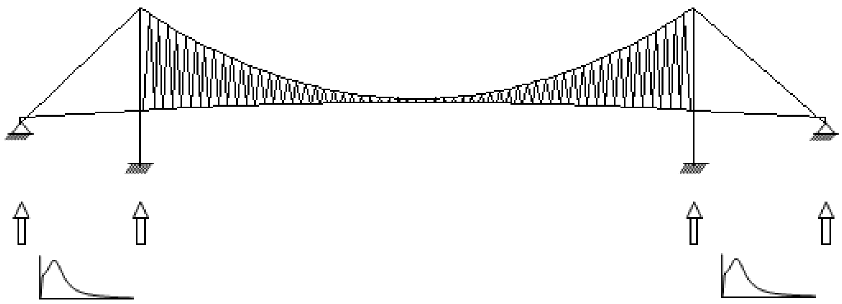

Fig. 8 Two-dimensional finite element model of Bosporus Suspension Bridge subjected to vertical ground motion

The deck was constituted considering aerodynamic form to reduce of the wind affect along the bridge deck. The aerodynamic steel box girder deck of the bridge consist of 60 box girder deck pieces of 17.9 m long 3 m deep prefabricated sections 33 m wide. The top of each box section constitutes an orthotropic plate on which 35 mm thickness mastic asphalt surfacing is laid. The bridge has slender steel towers of 165 m high. The tower legs are 5.20 × 7.00 m at the bottom and they become 3.00 × 7.00 m at the top. Vertical tower legs are connected by tree horizontal portal beams. Main cables of the bridge are built up parallel wire, 5mm in diameter over the hot dipped galvanizing. Each main cable consists of 19 strands extending between towers and contains 548 parallel wires, with other four stands each of which contains 192 wires in the backstays.

It was shown that a two-dimensional analysis of suspension bridges provides natural frequencies and mode shapes which are in close agreement with those obtained by the three-dimensional analysis. So, two-dimensional finite element model is used for the present calculations. The fact that this two-dimensional model has a relatively small number of degrees of freedom makes it more attractive by saving on computer time. Obviously, if actual design values for the responses are desired three-dimensional models should be taken into account. The selected finite element model of the bridge is represented by 202 nodal points, 195 beam elements and 118 truss elements as shown in Fig. 8.

This model has three degrees of freedom at each nodal point, namely, two translational degrees of freedom in vertical and longitudinal axes and one rotational degree of freedom. So, the finite element model of the bridge is decreased to 475 degrees of freedom and therefore a two-dimensional analysis is adopted in the vertical plane of the Bosporus Suspension Bridge for the stochastic response analysis when subjected to uniform and partially correlated seismic ground motions.

The filtered white noise ground motion model modified by Clough and Penzien (1993) is used as a ground motion model in which the spectral density function intensity parameter is determined according to the S16E component of the Pacoima dam record of the San Fernando earthquake in 1971. The analyses performed in this study are carried out with a newly developed computer code SVEM (Dumanoglu and Soyluk 2002) based on STOCAL (Button *et al*. 1981). The analyses are obtained for 2.5% damping ratio and first 15 modes.

Since the primary objective of this study is to perform a parametrical study associated with the stochastic response of a suspension bridge subjected to uniform and partially correlated seismic ground motions; the soil-structure interaction is not considered. Although the soil-structure interaction is important for bridge structures (Callisto *et al*. 2013, Erhan and Dicleli 2014, Ghotbi 2016), the non-consideration of the soil-structure interaction in this study is caused by the avoidance to model absorbing boundaries in the dynamic analysis.

4. Numerical results

Because the numerically chosen bridge is approximately symmetric, results obtained from stochastic analyses are presented only for the bridge deck and for the Asian side tower.

4.1 Mean of maximum responses

Mean of maximum values of vertical total deck displacements and total deck bending moments calculated for uniform and correlated ground motion cases are compared in Figs. 9 and 10, respectively. As can be observed from both figures the uniform ground motion case causes closer response values to those of the correlated ground motion cases except Model 1.

Figs. 11 and 12 illustrate the mean of maximum values of horizontal total tower displacements and total tower bending moments, respectively. At the tower the correlated ground motion cases

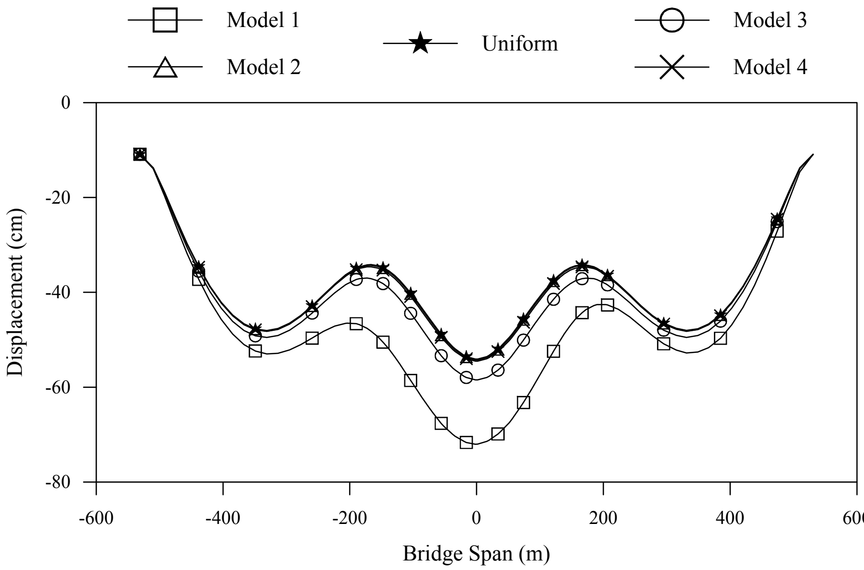

Fig. 9 Mean of maximum vertical total deck displacements

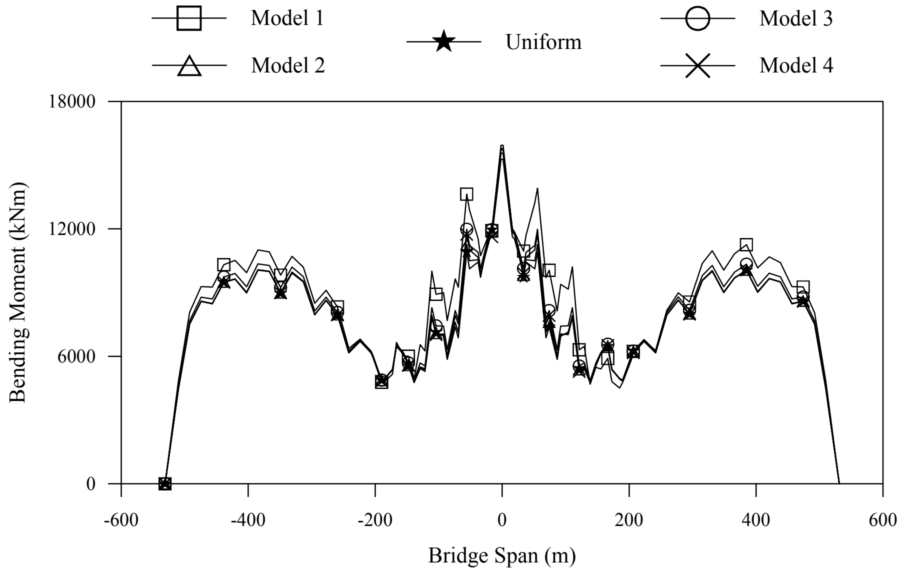

Fig. 10 Mean of maximum total deck bending moments

yield remarkably larger response values when compared with the uniform ground motion case. The responses obtained for Model 1 are larger than the response values obtained from other spatial variability models both at the tower and deck. This difference can be attributed to the frequency contents of the spatial variability models. As can be observed only Model 1 shows a coherency loss even at zero frequency. Model 4 seems to provide the smallest response values among the spatial variability models considered in this study. If the variation of Model 4 over frequency is investigated, it can be observed that this spatial variability model approaches rapidly to zero especially for large separation distances. Because the fundamental frequency w_g = 15.0 rad/s of the considered ground motion model is largely beyond the frequency range where the spatial variability model has some contributions, this model causes usually smaller response values. So, the behaviour of each spatial variability model especially at the low-frequency range seems to control the structural responses. The response values obtained from Model 2 and Model 3 are usually in between the results obtained from Model 1 and Model 4.

It is also observed that the response values obtained from the correlated ground motion cases are generally higher than those of the uniform ground motion case which assumes full correlation between the ground motions applied to the support points.

4.2 Variances of response components

The relative contributions of pseudo-static, dynamic and covariance components to the total vertical displacement responses along the bridge deck are presented in Figs. 13 and 14 for uniform and correlated (Model 1) ground motion cases, respectively. The normalization is performed by dividing the variance values by the maximum total response. For uniform ground motion model acting in the vertical direction, the pseudo-static displacement component is constant because of the rigid body motion as presented in Fig. 13. At the bridge deck where maximum total displacement takes place it can be observed that the dynamic component contributes 101%, the

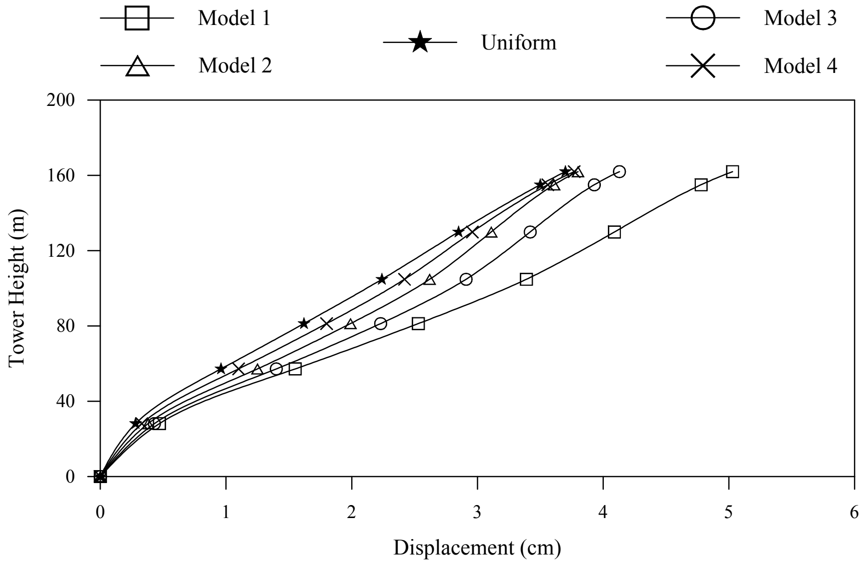

Fig. 11 Mean of maximum horizontal total tower displacements

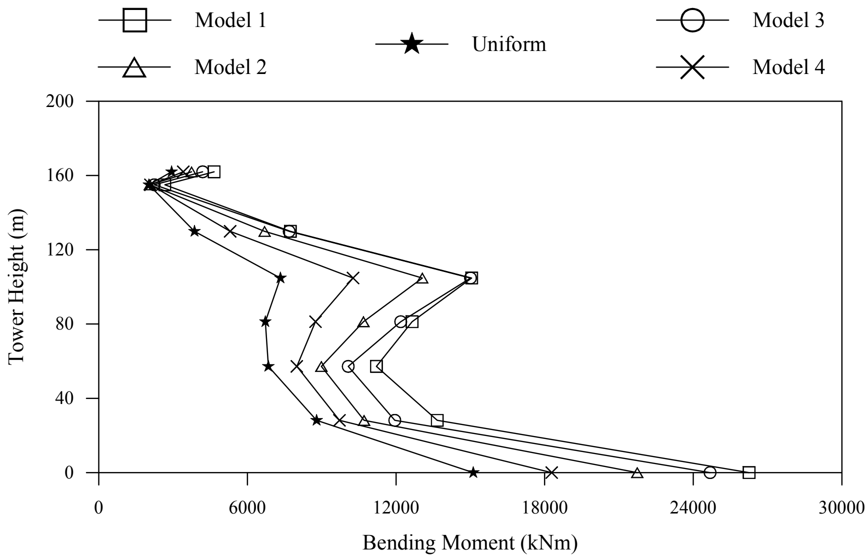

Fig. 12 Mean of maximum total tower bending moments

pseudo-static component contributes 3%, and the covariance component contributes -4% for both uniform and correlated (Model 1) excitation cases. The contribution of the pseudo-static and the covariance components to the total deck displacement response can be neglected for both uniform and correlated ground motion cases. This fact is caused by the flexible nature of the considered suspension bridge deck.

Figs. 15 and 16 show the relative contributions of the response components to the horizontal tower displacements for uniform and correlated (Model 1) ground motion cases. In the case of uniform ground motion model, the total tower displacements are completely dominated by the dynamic component.

For the correlated (Model 1) ground motion case the dynamic component contributes 74%, the pseudo-static component contributes 31%, and the covariance component contributes -5% at the top of the tower. As the correlated ground motion and uniform ground motion cases cause very close vertical displacements at the deck which are totally dominated by the dynamic component, tower displacements have important contributions from the pseudo-static component for the correlated ground motion case.

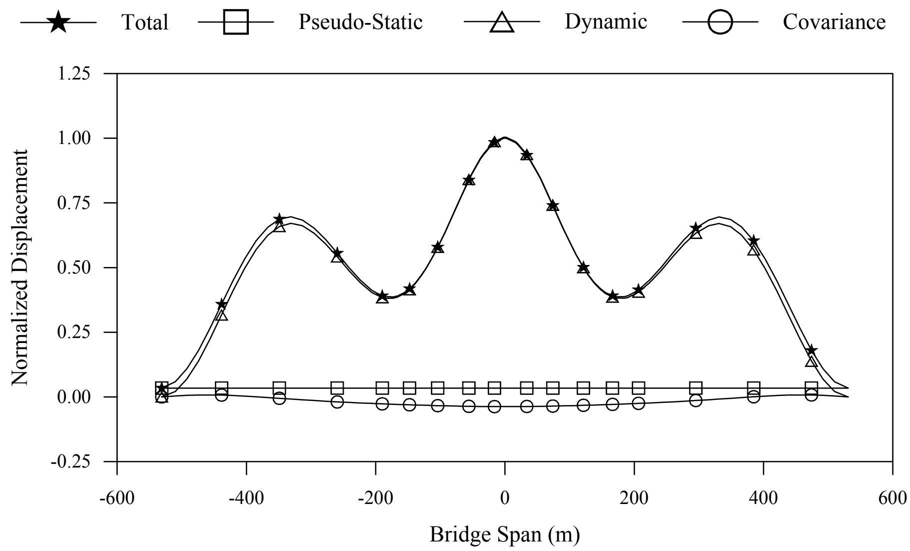

Fig. 13 Normalized displacement variances of the deck for uniform ground motion

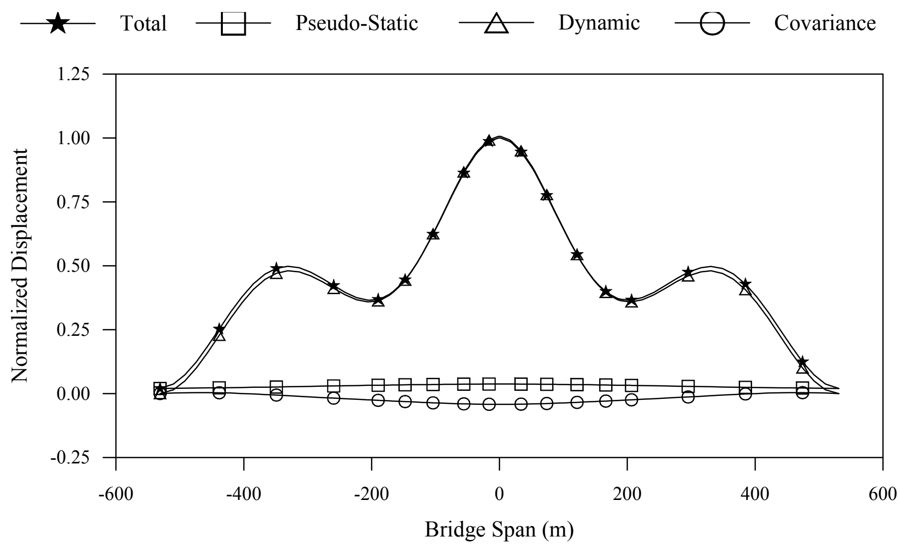

Fig. 14 Normalized displacement variances of the deck for correlated (Model 1) ground motion

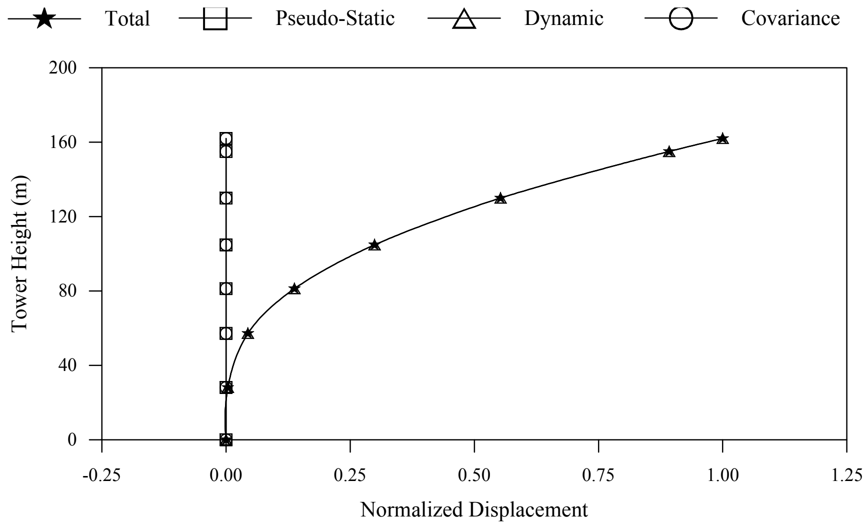

Fig. 15 Normalized displacement variances of the tower for uniform ground motion

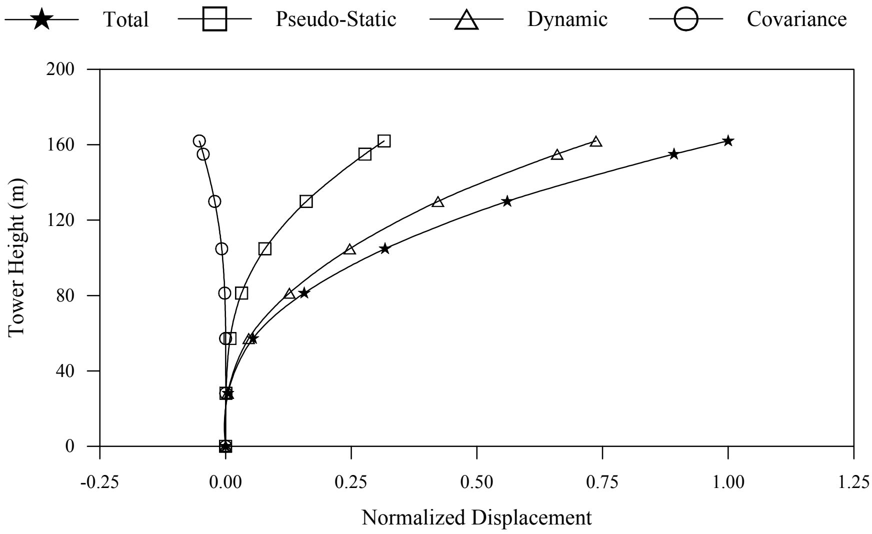

Fig. 16 Normalized displacement variances of the tower for correlated (Model 1) ground motion

5. Conclusions

In this study, stochastic analysis of a suspension bridge subjected to the partially correlated seismic ground motions are carried out considering the geometric nonlinearity and four different spatial variability models proposed by Harichandran and Vanmarcke (Model 1), Abrahamson (Model 2), Hindy and Novak (Model 3) and Uscinski (Model 4). Because each of the loss of coherency models has its own characteristics, it is intended to determine the sensitivity of a suspension bridge due to these loss of coherency models which represent the spatial variability of ground motions.

Mean of maximum response values calculated for the spatial variability models are compared with each other and with those of the uniform ground motion model. It is observed that while the largest response values are obtained for Model 1, Model 4 produces the smallest responses among the considered partially correlated ground motion models. Model 2 and Model 3 yield response values in between the responses obtained from Model 1 and Model 4. These variations are mainly caused due to the different frequency contents of the spatial variability models especially at the low-frequency range that controls the responses. While Model 1 induces partial correlation at low frequencies, Model 2 through Model 4 exhibit full correlation even at zero frequency. Model 4 rapidly approaches to zero at the frequency range that is very close to the fundamental frequency region of the earthquake ground motion model to which the bridge is subjected. The response values obtained from the uniform ground motion case are usually smaller than those of the responses obtained from the partially correlated ground motion cases.

The contributions of the pseudo-static, dynamic and covariance components to the total responses are also investigated for uniform and correlated ground motion cases (Model 1). While the response values obtained at the flexible parts of the bridge from both models are totally dominated by the dynamic component, the pseudo-static component also has significant contributions for the response values at the rigid parts of the bridge although those are mostly dominated by the dynamic component for the correlated ground motion case.

Although spatial variability models of Model 2 to 4 resulted very close responses to those of the uniform ground motion case, this result cannot be generalized to all suspension bridges. The reason behind this conclusion can be attributed to the fact that structural responses depend largely on the relation of the frequency contents of the bridge, the ground motion model and the spatial variability model. To be able to generalize these results different suspension bridges with different fundamental frequencies should be investigated underground motions with different frequency contents.

As it is obvious from the obtained results, spatial variability of ground motions should be taken into account while analysing long-span suspension bridges. However, which model to be selected as the spatial variability model is still a challenging task, because each method is based on a different approximation. However, the consistency observed for the results indicates that all the considered loss of coherency models are somehow representative of the spatial variability of ground motions.

References

Abrahamson, N.A. (1993), “Spatial variation of multiple support inputs”, *Proceedings of the 1st U.S. Seminar*, Seismic Evaluation and Retrofit of Steel Bridges, San Francisco, CA, USA.

Adanur, S., Dumanoglu, A.A. and Soyluk, K. (2003), “Stochastic analysis of suspension bridges for different correlation functions”, *Proceedings of the 9th International Conference on Civil and Structural Engineering Computing*, Egmond-aan-Zee, Netherlands, September, **Volume 86**, pp. 207-208.

Adanur, S., Altunışık, A.C., Soyluk, K., Bayraktar, A. and Dumanoğlu, A.A. (2016), “Multiple-support seismic response of bosporus suspension bridge for various random vibration methods”, *Case Studies in Structural Engineering*, **5**, 54-67.

Ateş, Ş., Bayraktar, A. and Dumanoglu, A.A. (2006), “The effect of spatially varying earthquake ground motions on the stochastic response of bridges isolated with friction pendulum systems”, *Soil Dyn. Earthq. Eng.*, **26**(1), 31-44.

Atmaca, B. and Ateş, Ş. (2012), “Construction stage analysis of three-dimensional cable-stayed bridges”,

Steel Compos. Struct., Int. J., **12**(5), 413-426.
Bi, K. and Hao, H. (2013), "Numerical simulation of pounding damage to bridge structures under spatially varying ground motions", *Eng. Struct.*, **46**, 62-76.
Brownjohn, J.M.W. (1994), "Observations on non-linear dynamic characteristics of suspension bridges", *Earthq. Eng. Struct. Dyn.*, **23**, 1351-1367.
Bryja, D. (2009), "Stochastic response analysis of suspension bridge under gusty wind with time-dependent mean velocity", *Archiv. Civil Mech. Eng.*, **9**(2), 15-38.
Button, M.R., Der Kiureghian, A. and Wilson, E.L. (1981), STOCAL-User information manual; Report no UCB/SEMM-81/2, Department of Civil Engineering, University of California, Berkeley, CA, USA.
Callisto, L., Rampello, S. and Viggiani, G.M.B. (2013), "Soil–structure interaction for the seismic design of the Messina Strait Bridge", *Soil Dyn. Earthq. Eng.*, **52**, 103-115.
Cavdar, O. (2013), "Probabilistic sensitivity analysis of suspension bridges to near-fault ground motion", *Steel Compos. Struct., Int. J.*, **15**(1), 15-39.
Chen, M.T. and Harichandran, R.S. (2001), "Response of an earth dam to spatially varying earthquake ground motion", *J. Eng. Mech.*, **127**(9), 932-939.
Cheng, J. and Liu, X-L. (2012), "Reliability analysis of steel cable-stayed bridges including soil-pile interaction", *Steel Compos. Struct., Int. J.*, **13**(2), 109-122.
Clough, R.W. and Penzien, J. (1993), *Dynamics of Structures*, McGraw Hill, Singapore.
Der Kiureghian, A. and Neuenhofer, A. (1991), A response spectrum method for multiple-support seismic excitations; Report no UCB/EERC-91/08, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, CA, USA.
Dong, H., Du, X. and Zhou, Y. (2015), "Pounding analysis of RC bridge considering spatial variability of ground motion", *Earthq. Struct., Int. J.*, **9**(5), 1029-1044.
Dumanoglu, A.A. and Severn, R.T. (1990), "Stochastic response of suspension bridges to earthquake forces", *Earthq. Eng. Struct. Dyn.*, **19**, 133-152.
Dumanoglu, A.A. and Soyluk, K. (2002), *SVEM, A Stochastic Structural Analysis Program for Spatially Varying Earthquake Motions*, Turkish Earthquake Foundation, TDV/KT 023-76, Istanbul, Turkey.
Erhan, S. and Dicleli, M. (2014), "Effect of dynamic soil-bridge interaction modeling assumptions on the calculated seismic response of integral bridges", *Soil Dyn. Earthq. Eng.*, **66**, 42-55.
Ghotbi, A.R. (2016), "Response sensitivity analyses of skewed bridges with and without considering soil-structure interaction", *Structures*, **5**, 219-232.
Günaydin, M., Adanur, S., Altunışık, A.C., Sevim, B. and Türker, E. (2014), "Determination of structural behavior of Bosporus suspension bridge considering construction stages and different soil conditions", *Steel Compos. Struct., Int. J.*, **17**(4), 405-429.
Harichandran, R.S. and Vanmarcke, E.H. (1986), "Stochastic variation of earthquake ground motion in space and time", *J. Eng. Mech.*, **112**(2), 154-174.
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.
Hindy, A. and Novak, M. (1980), "Pipeline response to random ground motion", *J. Eng. Mech. Div.*, **106**(2), 339-360.
Li, B. and Chouw, N. (2014), "Experimental investigation of inelastic bridge response under spatially varying excitations with pounding", *Eng. Struct.*, **79**(15), 106-116.
Luco, J.E. and Wong, H.L. (1986), "Response of a rigid foundation to a spatially random ground motion", *Earthq. Eng. Struct. Dyn.*, **14**, 891-908.
Shrestha, B., Hao, H. and Bi, K. (2014), "Effectiveness of using rubber bumper and restrainer on mitigating pounding and unseating damage of bridge structures subjected to spatially varying ground motions", *Eng. Struct.*, **79**(15), 195-210.
Soyluk, K. (2004), "Comparison of random vibration methods for multi-support seismic excitation analysis of long-span bridges", *Eng. Struct.*, **26**(11), 1573-1583.
Soyluk, K. and Dumanoglu, A.A. (2004), "Spatial variability effects of ground motions on cable-stayed bridges", *Soil Dyn. Earthq. Eng.*, **24**, 241-250.

Uscinski, B.J. (1977), *The Elements of Wave Propagation in Random Media*, McGraw-Hill, New York, USA.

Wang, J., Liu, W., Wang, L. and Han, X. (2015), "Estimation of main cable tension force of suspension bridges based on ambient vibration frequency measurements", *Struct. Eng. Mech., Int. J.*, **56**(6), 939-957.

Zerva, A. (1992), "Seismic loads predicted by spatial variability models", *Struct. Safety*, **11**, 227-243.

Zhang, Y.H., Li, Q.S., Lin, J.H. and Williams, F.W. (2009), "Random vibration analysis of long-span structures subjected to spatially varying ground motions", *Soil Dyn. Earthq. Eng.*, **29**(4), 620-629.

CC