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Contribution of local site-effect on the seismic response of suspension bridges to spatially varying ground motions

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Abstract. In this paper, it is aimed to determine the stochastic response of a suspension bridge subjected to spatially varying ground motions considering the geometric nonlinearity. Bosphorus Suspension Bridge built in Turkey and connects Europe to Asia in Istanbul is selected as a numerical example. The spatial variability of the ground motion is considered with the incoherence, wave-passage and site-response effects. The importance of site-response effect which arises from the difference in the local soil conditions at different support points of the structure is also investigated. At the end of the study, mean of the maximum and variance response values obtained from the spatially varying ground motions are compared with those of the specialised cases of the ground motion model. It is seen that each component of the spatially varying ground motion model has important effects on the dynamic behaviour of the bridge. The response values obtained from the general excitation case, which also includes the site-response effect causes larger response values than those of the homogeneous soil condition cases. The variance values calculated for the general excitation case are dominated by dynamic component at the deck and Asian side tower. The response values obtained for the site-response effect alone are larger than the response values obtained for the incoherence and wave-passage effects, separately. It can be concluded that suspension bridges are sensitive to the spatial variability of ground motion. Therefore, the incoherence, the wave-passage and especially the site-response effects should be considered in the stochastic analysis of this type of engineering structures.

Keywords: suspension bridge; spatially varying ground motion; incoherence effect; wave-passage effect; site-response effect; geometric nonlinearity

1. Introduction

As the dynamic responses of lifeline structures such as long span bridges will be significantly affected by the spatial variation of the seismic ground motions, the earthquake-response analysis of long span bridges subjected to the spatially varying ground motions are of particular interest over the last two decades. Over the time of this period, many studies have been performed on the linear-nonlinear and deterministic-stochastic earthquake response analyses of various structures

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subjected to uniform as well as multiple-support excitations.

Abdel-Ghaffar and Rubin (1982) investigated the effects of the multiple-support seismic excitations on suspension bridges and concluded that the uncorrelated ground motions overestimate the responses compared to those of the uniform ground motion case. The effect of wave propagation on the deterministic (Dumanoglu and Severn 1989) and stochastic (Adanur and Dumanoglu 2002, Giaralis and Spanos 2012) responses of suspension bridges were studied and concluded that the apparent wave velocity of propagation might have a significant effect on the bridge responses. Sasmal *et al*. (2011), Ni *et al*. (2015), Shrestha *et al*. 2015, Xie *et al*. (2015) investigated the structural response of bridges. Brownjohn (1994) carried out nonlinear dynamic analysis of suspension bridges considering the geometric nonlinearity and concluded that nonlinear dynamic analysis would be necessary for computing the response of suspension bridges.

Simplified bridge models like continuous beams, viaducts, reinforced concrete bridges, incompressible circular arches to spatially varying ground motions were investigated by Ateş *et al*. (2006), Bi and Hao (2013), Li and Chouw (2014), Shrestha *et al*. (2014) and the significance of the spatially varying ground motions was observed. Zhang *et al*. (2009), Soyluk and Sıcacık (2012), and Lin *et al*. (2004) performed stochastic response analyses of bridges subjected to spatially varying ground motions and underlined the significance of the spatial variability of ground motions between the support points.

Rassem *et al*. (1996) analysed the response of a long span suspension bridge to spatially varying ground motion due to the topographic effects. Harichandran *et al*. (1996) carried out stationary and transient response analysis of suspension and deck arch bridges to spatially varying earthquake motions and highlighted the importance of the effect of spatial variation of earthquake ground motions on the response of long structures. Simplified bridge models and a suspension bridge model subjected to the spatially varying earthquake motions were studied based on a newly developed multiple support response spectrum method (Der Kiureghian *et al*. 1997). It was concluded that the new response spectrum method offers a simple and viable alternative for seismic analysis of multiply supported structures subjected to spatially varying ground motions. Ettouney and Gajer (2001) investigated the dynamic soil-structure interaction effects on the response of suspension bridges subjected to multiple support seismic excitation and outlined the importance of the soil-structure interaction and multiple-support seismic excitation effects. Dong *et al*. (2015) carried out the pounding analysis of bridge structures considering spatial variability of ground motion. Also, Hao and Chouw (2010) carried out the detail comparison about the required separation distance between decks and at abutments to avoid seismic pounding.

The objective of this study is to investigate the importance of the heterogeneous soil condition case on the stochastic dynamic behaviour of a suspension bridge. For this purpose the stochastic analysis of a suspension bridge subjected to spatially varying ground motion, including the incoherency, wave-passage and site-response effects are calculated considering the geometric nonlinearity. However, the importance of the site-response effect is investigated particularly. Mean of maximum and variance values of responses are obtained and compared with each other for the specialised ground motion models. Relative contributions of the pseudo-static, dynamic and covariance components to the total response are also presented.

2. Formulation

2.1 Spatially varying ground motion model

In long span structures such as suspension bridges, earthquake motions will not be the same at distances owing to the complex nature of the earth crust. It is obvious that because of the differences of local soil conditions at the supports, loss of coherency due to the reflections and refractions and travelling with finite velocity between the support points, earthquake motions will be subjected to significant variations at the support points of the bridge. This variation will cause internal forces because of the pseudo-static displacements which normally do not produce internal forces for uniform ground motions. So, when analysing suspension bridges, the spatial variability of the earthquake motions should be considered.

Spatial variability of the ground motion is characterised with the coherency function in frequency domain. The coherency function for the accelerations $\ddot{v}_{g_l}$ and $\ddot{v}_{g_m}$ at the support points l and m is written as (Der Kiureghian 1996)

$$\gamma_{lm}(\omega) = \frac{S_{\ddot{v}_{gl}\ddot{v}_{gm}}(\omega)}{\sqrt{S_{\ddot{v}_{gl}\ddot{v}_{gl}}(\omega) * S_{\ddot{v}_{gm}\ddot{v}_{gm}}(\omega)}} \tag{1}$$

where $S_{\ddot{v}_{gl}\ddot{v}_{gl}}(\omega)$, $S_{\ddot{v}_{gm}\ddot{v}_{gm}}(\omega)$ and $S_{\ddot{v}_{gl}\ddot{v}_{gm}}(\omega)$ indicate the auto-power spectral densities of the accelerations and their cross-power spectral density, respectively. This function is dimensionless and complex valued. For the coherency function, the following model proposed by (Der Kiureghian 1996) is used

$$\gamma_{lm}(\omega) = \gamma_{lm}(\omega)^i \gamma_{lm}(\omega)^w \gamma_{lm}(\omega)^s = \gamma_{lm}(\omega)^i \exp\left[i\left(\theta_{lm}(\omega)^w + \theta_{lm}(\omega)^s\right)\right] \tag{2}$$

where $\gamma_{lm}(\omega)^i$ characterises the real valued incoherence effect, $\gamma_{lm}(\omega)^w$ indicates the complex valued wave-passage effect and $\gamma_{lm}(\omega)^s$ defines the complex valued site-response effect. Also, the complex term $\exp\left[i\left(\theta_{lm}(\omega)^w + \theta_{lm}(\omega)^s\right)\right]$ characterizes the wave-passage and site-response effects together.

For the incoherence effect, resulting from the reflections and refractions of seismic waves through the soil during their propagation, the widely used model proposed by Harichandran and Vanmarcke (1986) is considered. This model is based on the analysis of recordings made by the SMART-1 seismograph array in Lotung, Taiwan and defined as

$$\gamma_{lm}(\omega)^i = A\, exp\left[-\frac{2d_{lm}}{\alpha\theta(\omega)}(1-A+\alpha A)\right] + (1-A)\, exp\left[-\frac{2d_{lm}}{\theta(\omega)}(1-A+\alpha A)\right] \tag{3}$$

$$\theta(\omega) = k\left[1+\left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-\frac{1}{2}} \tag{4}$$

where ω is the circular frequency, d_{lm} is the distance between support points l and m. A, α, k, f_0 and b are model parameters and 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).

The wave-passage effect resulting from the difference in the arrival times of waves at support points is defined as (Der Kiureghian 1996)

$$\theta_{lm}(\omega)^d = -\frac{\omega d_{lm}^L}{v_{app}} \tag{5}$$

where v_{app} is the apparent wave velocity and d_{lm}^{L} is the projection of d_{lm} on the ground surface along the direction of propagation of seismic waves. The apparent wave velocities employed in this study are v_{app}=400 m/s for soft soil, v_{app}=700 m/s for medium soil and v_{app}=1000 m/s for firm soil.

The site-response effect resulting from the differences in local soil conditions at the support points is obtained as (Der Kiureghian 1996)

$$\theta_{lm}(\omega)^{s} = tan^{-1}\frac{Im\left[H_l(\omega)H_m(-\omega)\right]}{Re\left[H_l(\omega)H_m(-\omega)\right]} \tag{6}$$

where $H_l(\omega)$ and $H_m(-\omega)$ are the local soil frequency response functions representing the filtration through soil layers.

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

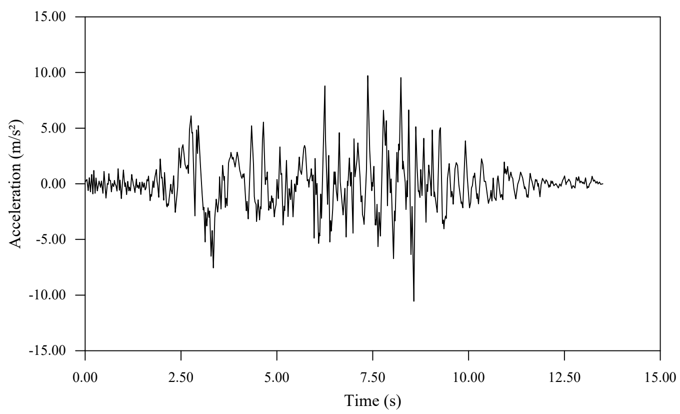
$$S_{\ddot{v}_{gl}\ddot{v}_{gl}}(\omega) = S_0\left|H_l(\omega)\right|^2\left|H_f(\omega)\right|^2 \tag{7}$$

where S_0 is the amplitude of the white-noise bedrock acceleration, $\left|H_1(\omega)\right|^2$ and $\left|H_f(\omega)\right|^2$ are the frequency response functions of the first and second filters representing the dynamic characteristics of the layers of soil medium above the rock bed. These functions are defined as

$$\left|H_l(\omega)\right|^2 = \frac{\omega_l^4 + 4\xi_l^2\omega_l^2\omega^2}{(\omega_l^2-\omega^2)^2 + 4\xi_l^2\omega_l^2\omega^2} \tag{8}$$

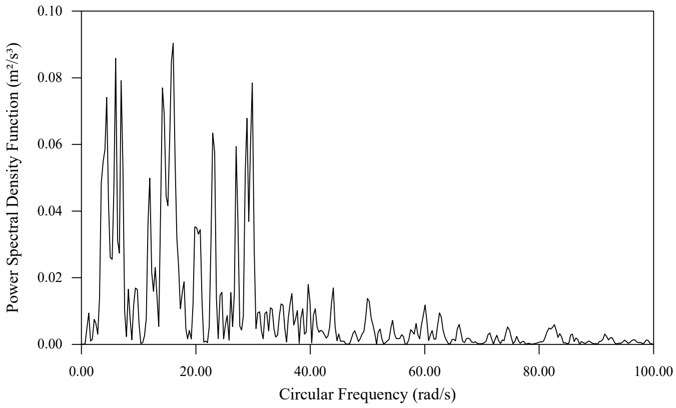
$$\left|H_f(\omega)\right|^2 = \frac{\omega^4}{(\omega_f^2-\omega^2)^2 + 4\xi_f^2\omega_f^2\omega^2} \tag{9}$$

ω_l and ξ_l are the resonant frequency and damping ratio of the first filter, ω_f and ξ_f are those of the second filter.

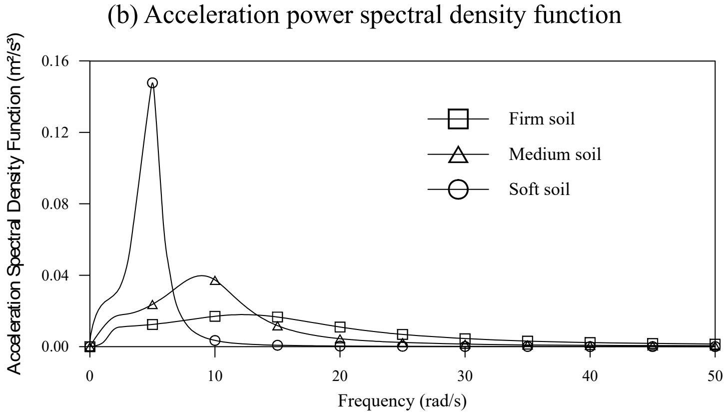


(a) Time history of acceleration

Fig. 1 S16E component of Pacoima Dam record of 1971 San Fernando earthquake



(b) Acceleration power spectral density function



(c) Acceleration power spectral density functions for filtered white noise model

Fig. 1 Continued

Table 1 Power spectral density function parameters for model soil types

Soil Type	ω_l (rad/s)	ξ_g	ω_f (rad/s)	ξ_f
Firm	15.0	0.6	1.5	0.6
Medium	10.0	0.4	1.0	0.6
Soft	5.0	0.2	0.5	0.6

In this study, firm (F), medium (M) and soft (S) soil types are used. The filter parameters for these soil types are utilized as presented in Table 1. The amplitude of the white-noise bedrock acceleration (S_0) is obtained for each soil type by equating the variance of the ground acceleration to the variance of S16E component of Pacoima Dam acceleration records of 1971 San Fernando earthquake (Fig. 1). The calculated values of the intensity parameter for each soil type are S_0(firm)=0.009715 m^2/s^3, S_0(medium)=0.014436 m^2/s^3, S_0(soft)=0.020267 m^2/s^3. Acceleration power spectral density function for each soil type is presented in Fig. 1(c).

2.2 Random vibration theory for spatially varying ground motion

In the random vibration theory, the variance of the ith total response component is expressed as

(Harichandran and Wang 1988)

$$\sigma_{z_i}^2 = \sigma^2{}_{z_i}^{qs} + \sigma^2{}_{z_i}^{d} + 2Cov(z_i^{qs}, z_i^d) \tag{10}$$

where, $\sigma^2{}_{z_i}^{qs}$ is the variance of pseudo-static response component, $\sigma^2{}_{z_i}^{d}$ is the variance of the dynamic response component and $Cov(z_i^{qs}, z_i^d)$ is the covariance between the pseudo-static and dynamic components.

The variance of the ith pseudo-static response component can be written as

$$\sigma^2{}_{z_i}^{qs} = \sum_{l=1}^{r}\sum_{m=1}^{r} A_{il} A_{im} \int_{-\infty}^{\infty} \frac{1}{\omega^4} S_{\ddot{v}_{g_l}\ddot{v}_{g_m}}(\omega) d\omega \tag{11}$$

where r is the number of support degrees of freedom where the ground motion is applied, ω is the circular frequency, A_{il} and A_{im} are the static displacement components due to unit support motions and $S_{\ddot{v}_{g_l}\ddot{v}_{g_m}}(\omega)$ is the cross spectral density function of accelerations between supports l and m.

The variance of the ith dynamic response component may be expressed as

$$\sigma^2{}_{z_i}^{d} = \sum_{j=1}^{n}\sum_{k=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_{ij}\psi_{ik}\Gamma_{lj}\Gamma_{mk} \int_{-\infty}^{\infty} H_j(-\omega) H_k(\omega) S_{\ddot{v}_{g_l}\ddot{v}_{g_m}}(\omega) d\omega \tag{12}$$

where n is the number of the modes used in the analysis, Γ is the modal participation factor, ψ is the eigenvectors, $H(\omega)$ is the modal frequency response function.

The covariance between the ith pseudo-static and dynamic response components can be expressed as

$$Cov(z_i^{qs}, z_i^d) = -\sum_{j=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_{ij} A_{il} \Gamma_{mj} \int_{-\infty}^{\infty} \frac{1}{\omega^2} H_j(\omega) S_{\ddot{v}_{g_l}\ddot{v}_{g_m}}(\omega) d\omega \tag{13}$$

2.3 Mean of maximum value

Depending on the peak response and standard deviation of the total response, the mean of maximum value (μ), in the stochastic analysis can be written as

$$\mu = p\sigma_z \tag{14}$$

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

2.4 Geometrically nonlinear behaviour

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 tension 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.

In the geometrically nonlinear behaviour, the total stiffness matrix of the system (K) can be written as

$$K = K_E + K_G \tag{15}$$

where K_E is the elastic stiffness matrix and K_G the geometric stiffness matrix.

3. Numerical example

In this study, Bosphorus Suspension Bridge built in Turkey and connects Europe to Asia in Istanbul is selected as a numerical example (Fig. 2). The construction of the bridge started in 1973 and completed in 1983.

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. 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.

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.

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 (Dumanoglu and Severn 1985). So, two-dimensional finite element model is adopted for the present calculations. Obviously, if actual design values for the responses are desired, three-dimensional models should be taken into account. As the deck, towers, and cables of the selected bridge are modelled by beam elements; the hangers were modelled by truss elements. A finite element model of the bridge with 161 nodes, 159 beam elements, 118 truss elements, and 475 degree of freedoms are used in the analyses (Fig. 3). The analyses are conducted for 2.5% damping ratio and for first 15 modes by the computer code SVEM (Dumanoglu and Soyluk 2002).

Fig. 2 Bosphorus Suspension Bridge

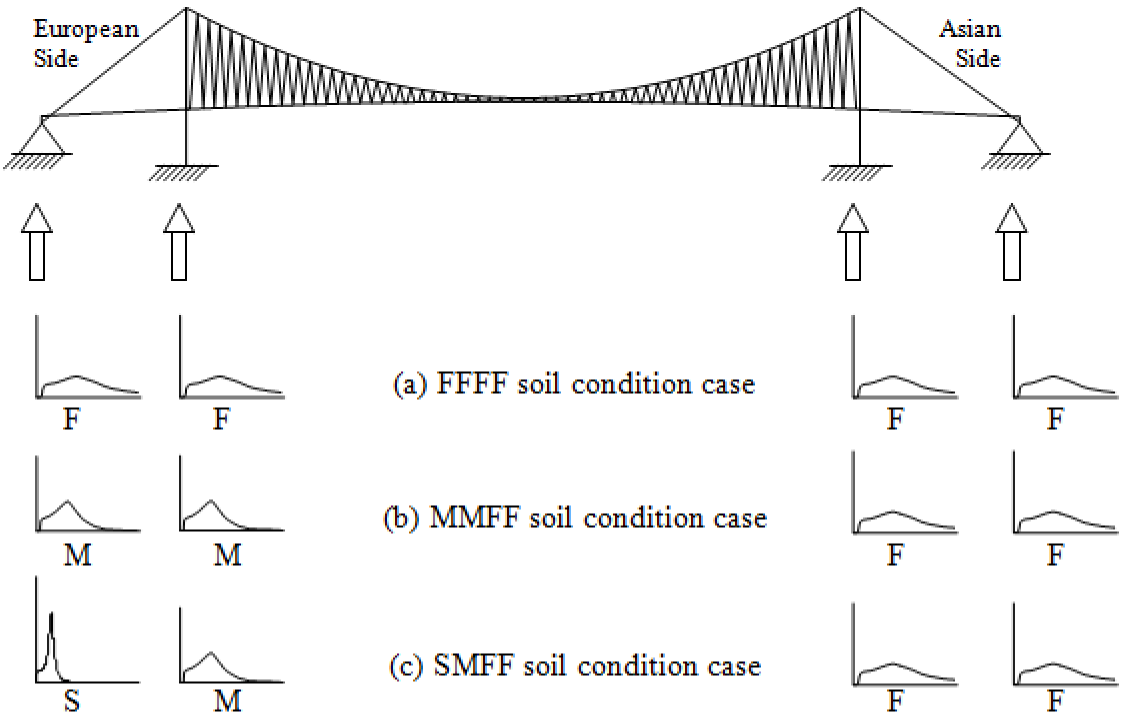


Fig. 3 Suspension bridge subjected to spatially varying ground motions in the vertical direction for the different soil condition case

The filtered white noise ground motion model modified by Clough and Penzien (1993) is used as a ground motion and applied in the vertical direction with two thirds of the recorded amplitude of the ground motion. The spectral density function intensity parameters are determined according to the S16E component of the Pacoima dam record of the San Fernando earthquake in 1971.

The soil-structure interaction is very important for engineering structures especially long-span bridges and considered by different researchers with detail (Ettouney and Gajer 2001, Ateş *et al.* 2013, Callisto *et al.* 2013, Peric *et al.* 2016). But, in this paper, it is aimed to determine the spatial variability effect of ground motions on suspension bridges. So, the soil-structure interaction is not considered and the boundary conditions are considered as fixed. For the next studies, the effect of soil-structure interaction will be performed for comparison.

3.1 Numerical computations

Stochastic analysis of the considered suspension bridge is performed for spatially varying ground motions by taking into account the incoherence, wave-passage and site-response effects as well as the geometric nonlinearity. For this purpose three different soil condition cases are considered for the bridge supports. In this section, each letter corresponds to a support and the soil condition at that support point.

Case A: All the supports are assumed to be founded on soils with firm soil type (FFFF). This case corresponds to the homogeneous soil type.

Case B: While the supports at the European side supports are assumed to be founded on medium soil, the Asian side supports are assumed to be founded on firm soil type (MMFF).

Case C: The European side anchorage is founded on soft soil, the European side tower pier is founded on medium soil and the remaining supports at the Asian site are founded on firm soil (SMFF).

The suspension bridge subjected to the spatially varying ground motion in the vertical direction is shown in Fig. 3. The vertical input motion is assumed to be travelling across the bridge from the European side to the Asian side with finite velocity of 400 m/s at soft soil, 700 m/s at medium soil and 1000 m/s at firm soil. In the Turkish Earthquake Code (TEC 2007), the soil groups are classified (A, B, C, D) and the values such as standard penetration, relative density, unconfined compressive strength and drift wave velocity are given with detail for each groups. The spectral density function applied to each support point as a ground motion is different for each soil type. The following specialised ground motion models are included in this study.

Model 1: Uniform ground motion model (no loss of coherency and no time delay between support excitations and homogeneous soil condition)

$$\gamma_{lm}(\omega)^{i}\gamma_{lm}(\omega)^{w}\gamma_{lm}(\omega)^{s}=1\,.$$

Model 2: Incoherence effect (no time delay between support excitations and homogeneous soil condition)

$$\gamma_{lm}(\omega)^{w}\gamma_{lm}(\omega)^{s}=1,\;\;\gamma_{lm}(\omega)^{i}\neq 1\,.$$

Model 3: Wave-passage effect (no loss of coherency between the support excitations and homogeneous soil condition)

$$\gamma_{lm}(\omega)^{i}\gamma_{lm}(\omega)^{s}=1,\;\;\gamma_{lm}(\omega)^{w}\neq 1\,.$$

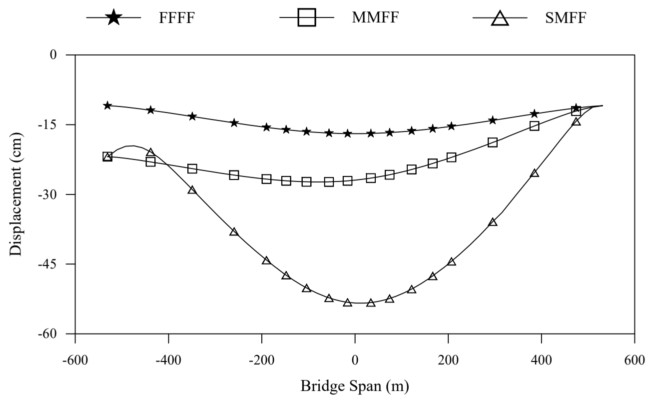
Model 4: Site-response effect (no loss of coherency and no time delay between the support excitations)

$$\gamma_{lm}(\omega)^{i}\gamma_{lm}(\omega)^{w}=1,\;\;\gamma_{lm}(\omega)^{s}\neq 1\,.$$

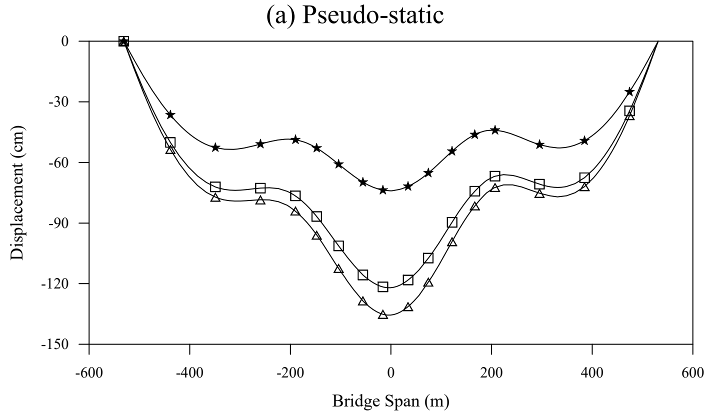
Model 5: General excitation case which includes the three spatial variability effects, namely the incoherence, wave-passage and site-response effects.

3.2 Mean of maximum response components

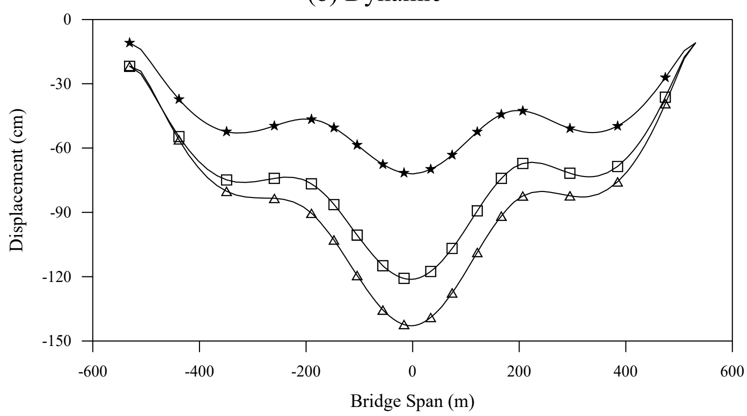
In this section, mean of maximum values of pseudo-static, dynamic and total responses are calculated for FFFF (homogeneous), MMFF and SMFF soil condition cases in the case of general excitation which includes all the three spatial variability effects (incoherence, wave-passage and site-response) of the ground motion. Mean of maximum vertical deck displacements calculated for different soil condition cases are shown in Fig. 4. As can be observed from the figures, the response values obtained for homogeneous (FFFF) soil condition cases are smaller than the response values obtained for MMFF and SMFF soil condition cases. It is also shown that, the response values obtained for SMFF soil condition cases are the largest. While the total vertical displacements at the end of the Asian side for the three different soil condition cases are close to each other, the displacements at the end of the European side for MMFF and SMFF soil condition cases are larger than those of the FFFF (homogeneous) soil condition case. This is caused by the pseudo-static displacements depending on the variation of the soil conditions at the European site from firm to soft. The total displacements at the middle of the deck obtained from the general excitation case overestimates the response by 68% and 100% for MMFF and SMFF soil condition cases, respectively when compared to the response obtained from the homogeneous (FFFF) soil condition case.



(a) Pseudo-static



(b) Dynamic



(c) Total

Fig. 4 Mean of maximum vertical displacements of the deck for general excitation case

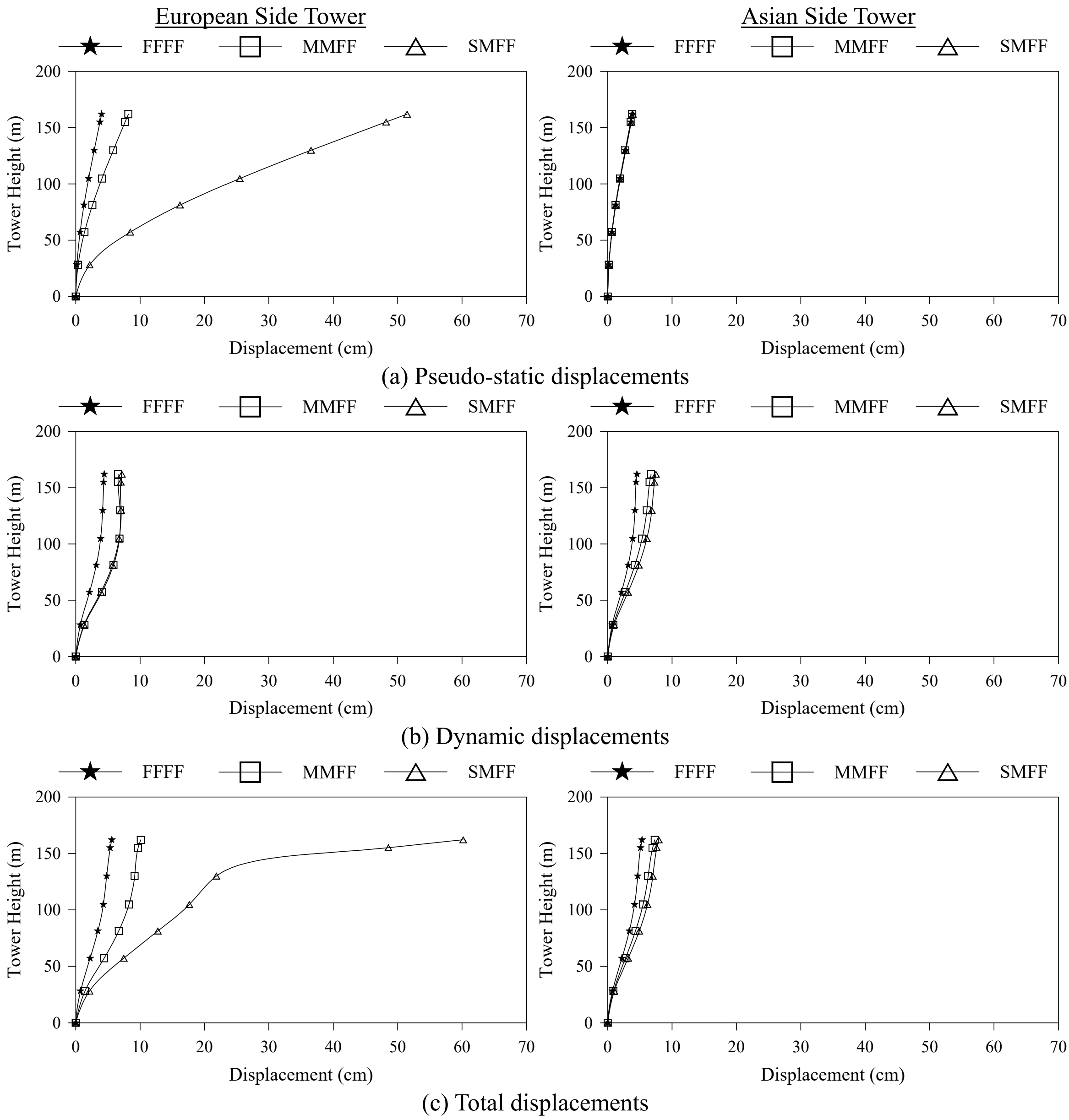
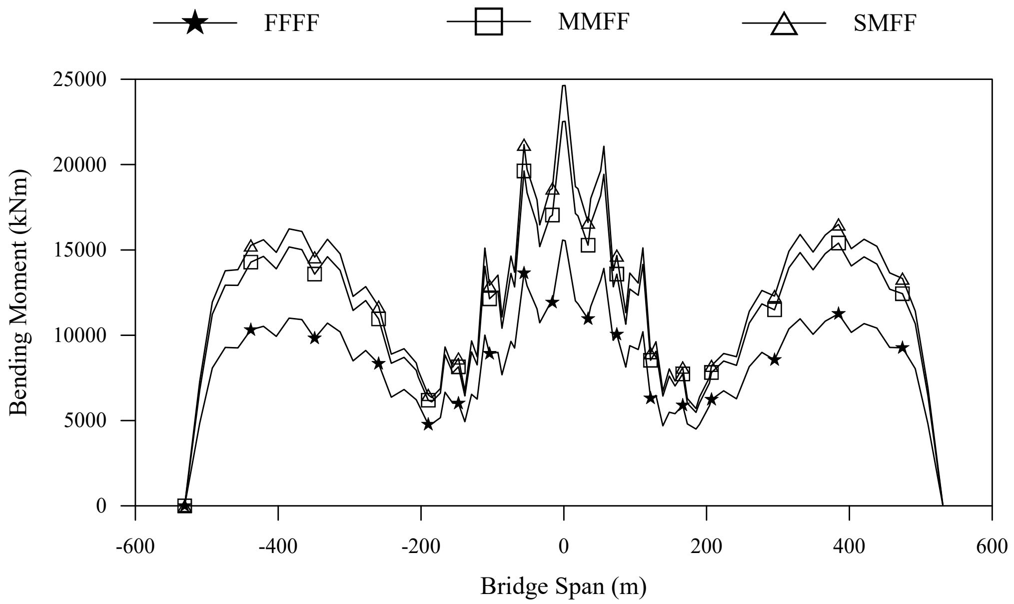


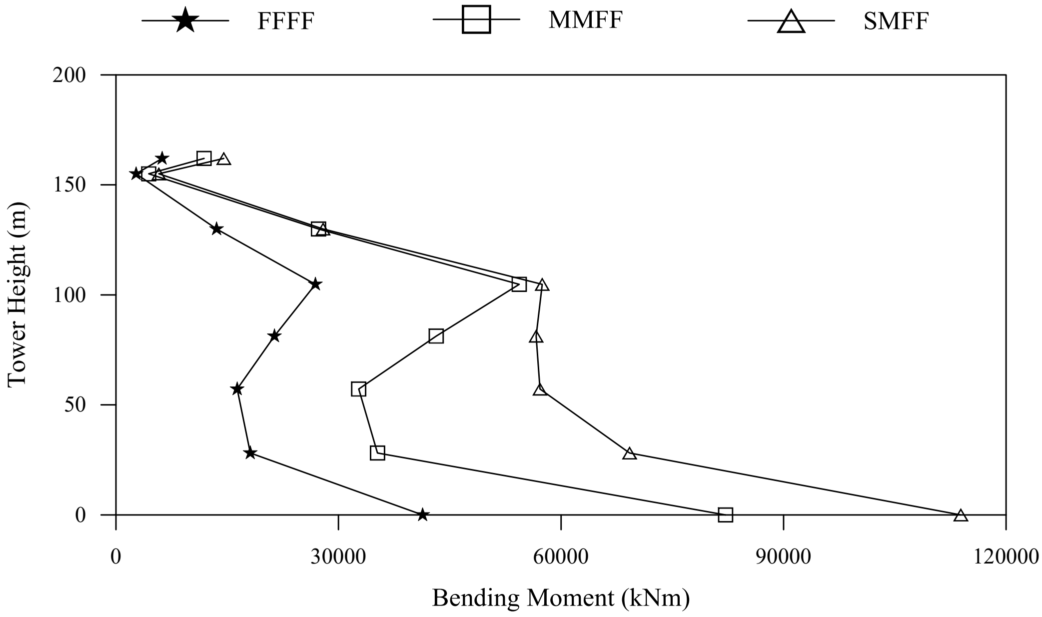
Fig. 5 Mean of maximum horizontal displacements for general excitation case

Mean of maximum values of the European and Asian side towers horizontal displacements obtained for the three different soil condition cases (FFFF, MMFF, SMFF) in the case of general excitation are presented in Fig. 5. Although the variations of the dynamic displacements at both towers are similar, the difference between the variations of the pseudo-static response values at the towers cause different total response values. Especially, the pseudo-static displacements obtained for SMFF soil condition case are very large and consequently cause very large total displacement values at the European side tower.

In Fig. 6, mean of maximum total bending moments of the deck and the European side tower for general excitation case are shown. While the moment values obtained for SMFF soil condition case are the largest, the response values obtained for FFFF (homogeneous) soil condition case are the smallest as obtained for the variation of the displacements.



(a) Deck



(b) European side tower

Fig. 6 Mean of maximum total bending moments for general excitation case

3.3 Variances of response components

The variance of the total response composes of the pseudo-static component, the dynamic component and the covariance component between the pseudo-static and dynamic components. In this part of the study, the contribution of each component to the total response of the bridge is investigated. The process of normalisation is performed by dividing the variance values by the maximum total response. The relative contribution of each component to the total vertical displacement along the bridge deck under the general ground motion case are presented in Fig. 7 for FFFF, MMFF, SMFF soil condition cases. It is shown that the total displacements are dominated by the dynamic component for the three soil condition cases. However, while the soil condition changes from firm soil to soft soil, the contribution of the dynamic component to the total response decreases and the contribution of the pseudo-static and the covariance components increase. At the bridge deck midspan where maximum total displacement occur it can be observed that the dynamic component contribute 101.29%, 98.01%, 84.12%; the pseudo-static component contribute 3.83%, 3.87%, 14.44% and the covariance component contribute -5.12%, -1.88%, 1.44% for FFFF, MMFF, SMFF soil condition cases, respectively.

Fig. 8 show the relative contributions of the response components to the horizontal tower displacements at the European and Asian sides under the general excitation case for the three soil condition cases. While the variations obtained for the displacements at the Asian side tower for each soil condition case are similar, the variations obtained for the European side tower are somehow different. The total response is dominated by the dynamic component at the Asian side tower for each soil condition case. At the European side tower, the total displacements are dominated by the dynamic component for FFFF soil condition case whereas the total displacements are dominated by the pseudo-static component for MMFF and SMFF soil condition cases. This is because the European side support soil conditions range from firm soil type to soft soil type and amplify the pseudo-static components. At the European side tower top point, it can be observed that the dynamic component contribute 65.35%, 41.00%, 4.66%; the pseudo-static component contribute 37.00%, 55.19%, 91.86% and the covariance component contribute -6.15%, 3.81%, 3.48% for FFFF, MMFF, SMFF soil condition cases, respectively. Similarly, at the Asian side tower top point the dynamic component contribute 76.01%, 87.50%, 92.00%; the pseudo-static component contribute 40.51%, 20.23%, 18.60% and the covariance component contribute -16.52%, -7.73%, -10.60% for FFFF, MMFF, SMFF soil condition cases, respectively.

3.4 Relative contribution of the spatial variability effects

In this section, mean of maximum response components are obtained for each of the incoherence, wave-passage and site-response effects as well as for the uniform ground motion and general excitation cases to highlight the relative importance of the spatial variability effects. For the uniform ground motion model the soil conditions where the bridge supports are constructed are defined as homogeneous firm soil type (FFFF). Only MMFF soil condition case is considered in order to investigate the effect of the site-response component of the spatially varying ground motion. Mean of maximum total deck displacements and bending moments are illustrated in Fig. 9 for the specialised ground motion models (uniform ground motion model, wave-passage effect, incoherence effect, MMFF site-response effect and general excitation case). While the response values obtained from the uniform ground motion model are the smallest, the general excitation case (considering wave-passage effect, incoherence effect and MMFF site-response effect) causes

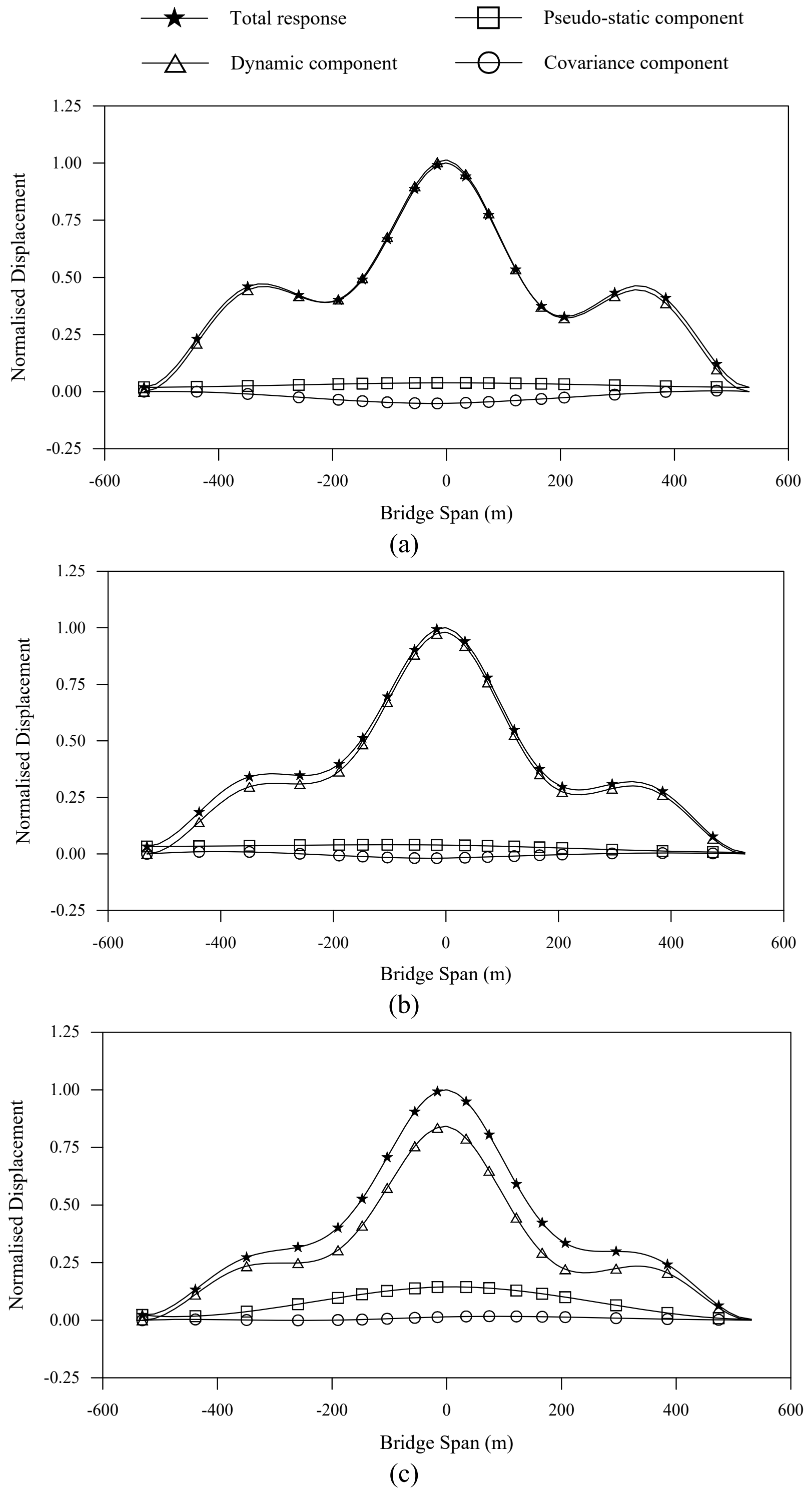
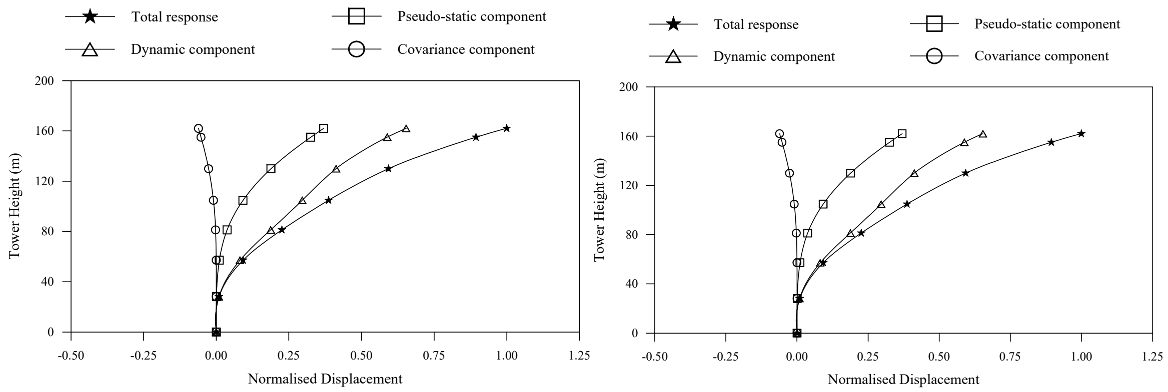
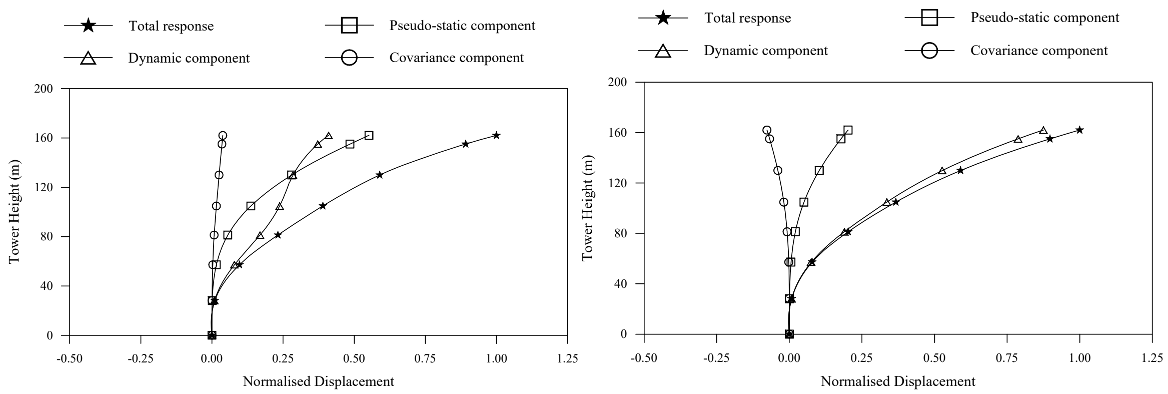


Fig. 7 Normalised displacement variances of the deck for general excitation case (a) FFFF soil condition, (b) MMFF soil condition, and (c) SMFF soil condition

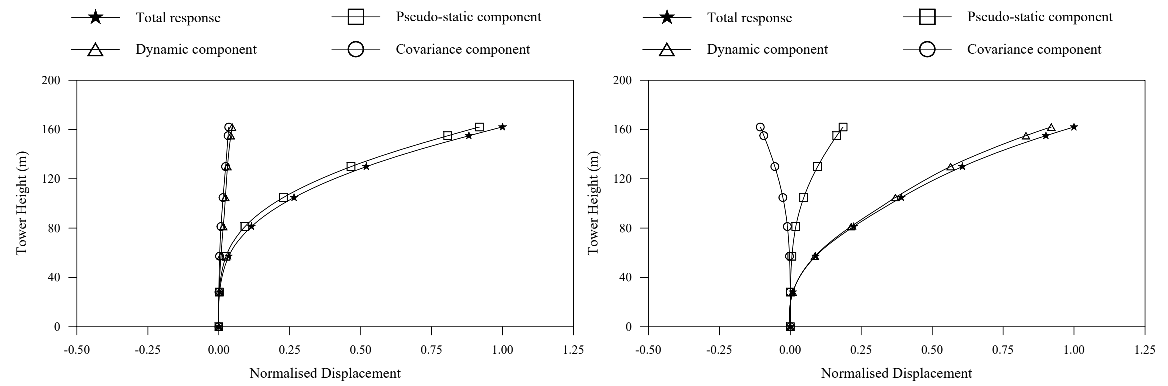
European Side Tower   Asian Side Tower



(a) FFFF soil condition case



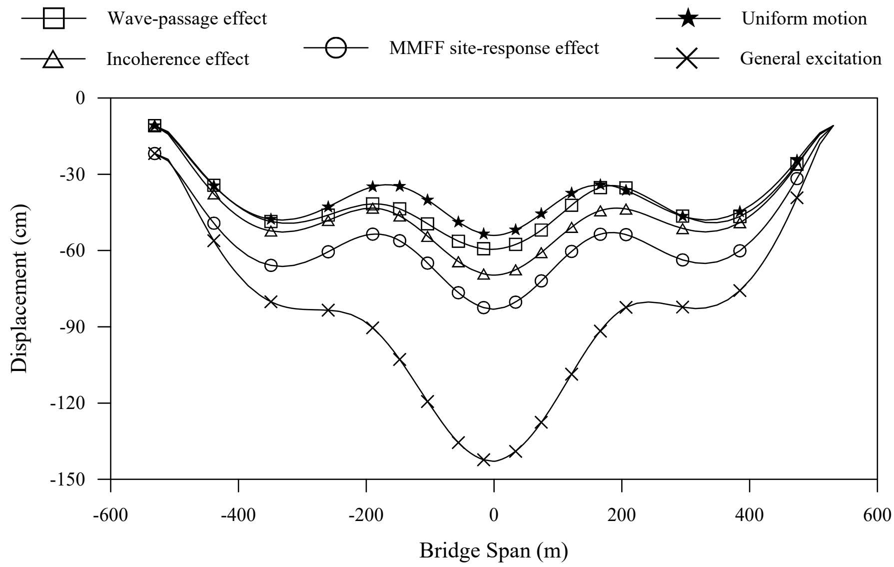
(b) MMFF soil condition case



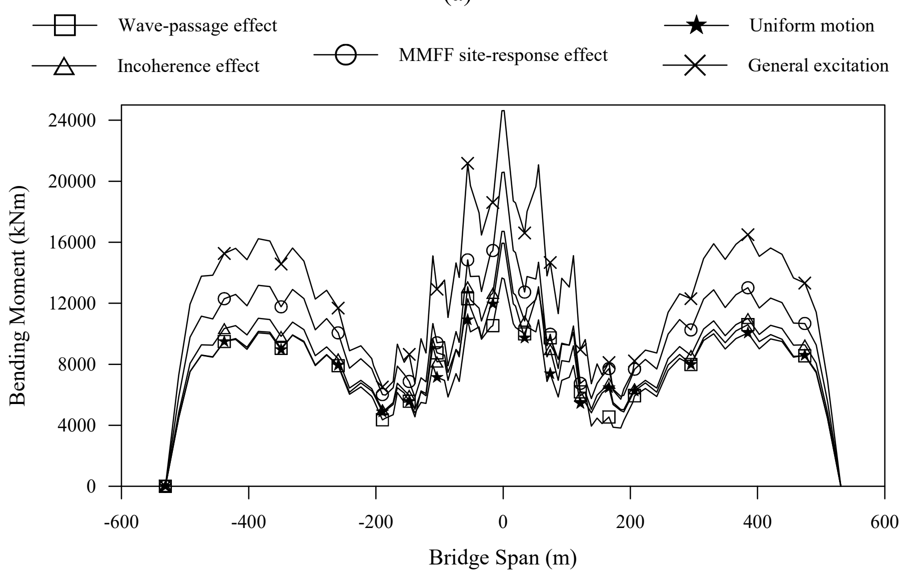
(c) SMFF soil condition case

Fig. 8 Normalised displacement variances of two towers for general excitation case

the largest response values. Compared with the remaining spatially varying ground motion components, it is seen that site-response is the dominant component between the others.



(a)



(b)

Fig. 9 Mean of maximum total deck (a) displacements and (b) bending moments

4. Conclusions

In this study, stochastic analysis of a suspension bridge subjected to spatially varying ground motions is carried out considering the geometric nonlinearity. The spatial variability of the ground motion is incorporated with the incoherence, wave-passage and site-response effects. The site-response effect is investigated in detail by situating the supports of the bridge on distinctly different soil sites. The analysis is applied to Bosporus Suspension Bridge. Mean of maximum and variance values of the responses are obtained and compared with each other for the specialised ground motion models. Relative contributions of pseudo-static, dynamic and covariance components to the total response are also presented.

The response values obtained from the general excitation case, which also includes the site-response effect causes larger response values than those of the homogeneous soil condition cases. Also the more difference between the soil conditions, the more response values take place.

The variance values calculated for the general excitation case, which includes all the three effects of the spatial variability of the ground motion, are dominated by the dynamic component at the deck and Asian side tower. While the total displacements are dominated by the dynamic component at the European side tower for the homogeneous soil condition case, the pseudo-static component dominates the total displacements for the heterogeneous soil condition cases. Furthermore, the relative contribution of the pseudo-static component to the total response generally increases by changing the support soil conditions from firm soil type to the soft soil type.

The response values obtained for the site-response effect alone are larger than the response values obtained for the incoherence and wave-passage effects, separately. The results obtained for the site-response effect are also larger than the results obtained for the uniform ground motion case, but smaller than those of the general excitation case.

This study mainly implies that long span bridges like suspension bridges are sensitive to the spatial variability of ground motion. Therefore, the incoherence, the wave-passage and especially the site-response effects should be considered in the stochastic analysis of such type of engineering structures. The results should never represent definite or final conclusions of this sophisticated dynamic problem, whereas the results obtained from this study show an application of the spatially varying ground motions on suspension bridges.

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