Sensitivity analysis of site response effect on stochastic response of the Oued Dib cable stayed bridge

N. Mezouer, H. Airouche & A. Kibboua
National Earthquake Engineering Research Center (CGS), Algiers, Algeria

M. Hadid
Ecole Nationale Supérieure des Travaux Publics, Kouba, Algiers, Algeria

SUMMARY:
In this paper, a numerical sensitivity analysis of the spatial variability of ground motions due to site response effect on the stochastic response of a bridge structure subjected to this phenomenon is discussed. The site response effect is due to the difference in local soil conditions at different support points. It is completely defined in term of the transfer functions of the soil columns under each support. The soil profile overlying bedrock is assumed to have a shear modulus with continuous variation which increases with some power exponent of depth. Kanai-Tajimi spectrum parameters are estimated and expressed analytically from the soil profile model. The bridge structure responses allow a comprehensive sensitivity analysis of site response effects. Interesting results in term of root mean square (RMS) responses (displacement, bending moment and shear forces) obtained from the spatial variability of ground motion are derived and compared to those induced by uniform excitation. Each component of the responses (pseudo static and dynamic) is differently influenced by the site response effect. The determination of the pseudo-static components is essential; it represents the differential displacements and induces additional forces which can cause ruptures. The results indicate clearly the importance to consider site effects for rigorous seismic analysis of structures founded on such soil conditions.

Keywords: Bridge, site effect, soil inhomogeneity, pseudo static, dynamic

1. INTRODUCTION

A seismic analysis of extended structures such as bridges, dams and large industrial buildings requires a rational understanding of the free surface seismic motion. Observations during earthquakes showed that at a given site, the records at distinct points are different, in amplitude, duration, and frequency content (Abrahamson *et al*. 1991, Harichandran and Vanmarke 1986). The spatial variation of seismic ground motions (SVGM) has an important effect on the response of extended structures. Der Kiureghian (Der Kiureghian 1996) suggests that SVGM is a consequence of several phenomena: the difference in arrival times of seismic waves to different recording stations (wave passage), the loss of coherency of the seismic movement, the difference in soil mechanical properties under different points of recording and the attenuation of seismic waves. These effects are characterized by a mathematical description in frequency domain given by coherency function which is the normalized cross power spectral density of the records at two stations (Luco and Wang 1986, Harichandran 1991, Laouami and Labbé 2001).
The effects of the SVGM on structure response subjected to such excitations are important and cannot be neglected (Abdelghaffar and Rubin 1982; Abdelghaffar and Rubin 1983; Zerva *et al.* 1988; Zerva 1990; Berrah and Kausel 1992; Der Kiureghian and Neuenhofer 1992; Monti *et al.* 1994; Der Kiureghian *et al.* 1997; Sextos 2001; Zanardo *et al.* 2002, Sextos *et al.* 2003; Lupoi *et al.*, 2005, Mezouer *et al.* 2011, Soyluk and Sicacik 2012). The SVGM effect on the response compared to the one induced by uniform excitations is complex and variable. It depends on the structural configuration, ground motions characterization, soil conditions at supports and response component.
Although significant aspects of the effects of spatial variability (wave passage, loss of coherency and attenuation effects) have been already clarified, there is still a need for more research especially in site response effect. For bridges, crossing alluvial rivers, the local soil conditions may be different especially between extreme supports and intermediate ones. Such important variations may contribute

strongly to generate different excitations at every support and introduce additional resonance due to soil amplification compared to other effects. The different soil conditions affect the amplitude variation of the motions and produce generally higher response than if the soil conditions were assumed to be identical.

Wang *et al* (1999) studied the effects of engineering geological condition on response of suspension bridges and conclude that is unacceptable to not take the large geological difference at the supports of bridge in the prediction of its responses. The site effect represented by different geological conditions: the north bridge support lies on quaternary deposit and the south one on rock site. The artificial accelerations on the ground surface from the seismic hazard analysis at the site of bridge are generated and their auto power spectral densities are computed. Dumanogluid and Soyluk (2003) investigate in details the site response effect on long span structure by situating its supports on distinctly different soil sites. The responses obtained from general excitation case which includes site response effect induce large values compared to those of homogeneous soil conditions; Also the more difference between the soil conditions, the more response values take place.

In the present paper, the cable stayed bridge of Oued Dib in Algeria is taken to allow a comprehensive sensitivity analysis of the site response effect on its stochastic response. The site response effect due to the difference in the soil local conditions is obtained as described by Der Kiureghian *et al.* (1997). The soil frequency response function idealizes the soil layer as a single degree of freedom oscillator. An analytical amplification function of a visco-elastic inhomogeneous soil profile overlying bedrock is used Hadid and Afra (2000). The Kanai-Tajimi spectrum parameters are estimated and expressed analytically from the soil profile model. The Oued Dib Bridge has three spans, the main span of 280m and two side spans of 111m. The central supports assumed to be founded on soft deposits and the two others on firm soil.

2. GROUND MOTION MODEL

The spatial variation of seismic ground motion is due to several phenomena: the incoherence effect (IE), the wave-passage effect (WPE), the attenuation effect (AE) and the site-response effect (SRE) resulting from the modification of the motion due to the different stratification with different dynamic properties of soil under each structure's support.

The model proposed by Der Kiureghian (1996) taking into account the combination of all the effects mentioned earlier is used.

$$\gamma_{kl}(\omega) = \gamma_{kl}^{IE}(\omega)\,\gamma_{kl}^{WPE}(\omega)\,\gamma_{kl}^{AE}(\omega)\,\gamma_{kl}^{SRE}(\omega) \tag{2.1}$$

In the present work, only site effects are considered. Difference in local soil conditions in term of stratification and dynamic properties under different support of the structure contribute, obviously and strongly, to generate different excitations at every support and introduce additional resonance of structure responses. The site response effect is completely defined in term of the transfer functions, $H_k(\omega)$ and $H_l(\omega)$ of the soil columns at the two stations *k* and *l* (Afra and Pecker 2002, Der Kiureghian 1996) given as:

$$\gamma_{kl}^{SRE}(\omega) = exp\left[i\,tan^{-1}\frac{Im\left[H_k(\omega)H_l(-\omega)\right]}{Re\left[H_k(\omega)H_l(-\omega)\right]}\right] \tag{2.2}$$

Im and *Re* are imaginary and real parts. Several methods can be used to evaluate the local soil frequency function. In present work, the soil is modeled as inhomogeneous profile of thickness h, overlying bedrock and having continuous variation of the soil shear modulus which increase with some power exponent of depth (Idriss and Seed 1968; Afra and Pecker 2002)

$$G_s(z_s) = G_0\left(\frac{z_s}{h}\right)^p \tag{2.3}$$

where G_0 is the shear modulus at depth h, z_s is the depth counted positively downwards from free surface and p is a parameter which takes values between zero and *1* depending on the soil nature (Gazetas 1984, Hadid and Afra 2000) (p is approximately equal to zero for stiff overconsolidated clay, vary from 0.45 to 0.6 for cohesionless materials and vary from 0.8 to 1 for normally consolidated

clays).
Enforcing the condition of continuity for the displacement and for the shear stress at the layer interface, the transfer function between the ground surface and bedrock is given by (Hadid and Afra 2000):

$$H(\tilde{\omega}) = \frac{1}{\Gamma\left(\frac{1}{2-p}\right)} \frac{\left(\frac{1}{2-p}\tilde{\omega}\right)^{\nu}}{J_{\nu}\left(\frac{2}{2-p}\tilde{\omega}\right) + iq\,J_{\nu+1}\left(\frac{2}{2-p}\tilde{\omega}\right)} \tag{2.4}$$

Where J_{ν} is the Bessel functions of the first kind of order $\nu = \frac{p-1}{2-p}$, $\tilde{\omega} = \frac{\omega h}{V_0}$ is the dimensionless frequency, $V_0 = \sqrt{\frac{G_0}{\rho_s}}$ is the shear wave velocity at the interface, $q = \frac{\rho_s V_0}{\rho_r V_r}$ is the impedance ratio and Γ is the Gamma function.
With the determination of the transfer function, the site effect phase is completely defined. It will be interesting to show that this amplification function can be used to estimate the Kanai-Tajimi spectrum parameters. These parameters depend on physical and geometrical characteristics of soil profile and then can be estimated with the following approximate analytical expressions (Hadid and Afra 2000):

$$\begin{cases} \xi_g = \dfrac{1}{2\sqrt{S_{max}^2 - 1.15}} \\ \omega_g = \dfrac{2\xi_g}{\left[\left(1+8\xi_g^2\right)^{1/2} - 1\right]^{1/2}}\omega_{max} \end{cases} \tag{2.5}$$

Where $S_{max} = \left| \frac{1}{\Gamma\left(\frac{1}{2-p}\right)} \frac{\left(\frac{1}{2-p}\tilde{\omega}_{max}\right)^{\nu}}{J_{\nu}\left(\frac{2}{2-p}\tilde{\omega}_{max}\right) + iq\,J_{\nu+1}\left(\frac{2}{2-p}\tilde{\omega}_{max}\right)} \right|$ is the peak value of the inhomogeneous soil layer amplification function amplitude at the fundamental frequency $\omega_{max} = \left(-0.3725\,p + \frac{\pi}{2}\right)\frac{V_0}{h}$.

The quantities ω_g and ξ_g represent the ground frequency and the ground damping of the Kanai-Tajimi spectrum given by

$$S(\omega) = S_0 \frac{\omega_g^4 + 4\xi_g^2\omega_g^2\omega^2}{\left(\omega_g^2 - \omega^2\right)^2 + 4\xi_g^2\omega_g^2\omega^2} \tag{2.6}$$

S_0 is the white noise power spectrum intensity of the bedrock outcrop acceleration. It is well noted that the damping ξ_g is affected significantly by the variation of the impedance ratio and the inhomogeneity parameter p (Mezouer, 2010). Also, the filter frequency ω_g is not too affected by the impedance ratio especially for firm soil ($q < 0.75$ and $p < 0.30$).

3. RANDOM VIBRATION ANALYSIS

Based on random vibration theory, the variance of the total response in the case of spatial variability of ground excitation (Harichandran and Wang, 1988), including the contributions of cross-correlation between modes, excitations at supports and components (pseudo static and dynamic), is given as:

$$\sigma_Z^2 = \sum_{k=1}^{m}\sum_{l=1}^{m} a_k a_l \rho_{D_{gk}D_{gl}} \sigma_{D_{gk}} \sigma_{D_{gl}} + 2\sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{j=1}^{n} a_k b_{lj} \rho_{D_{gk}S_{lj}} \sigma_{D_{gk}} \sigma_{S_{lj}} + \sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n} b_{ki} b_{lj} \rho_{S_{ki}S_{lj}} \sigma_{S_{ki}} \sigma_{S_{lj}} \tag{3.1}$$

a_k is the effective influence coefficient, and b_{ki} is the effective modal participation factor, D_{gk} is the ground displacement at the support *k*, S_{lj} is the response of an oscillator (ω_j, ς_j) at support *l*.

$\rho_{D_{gk}D_{gl}}$ is the cross-correlation coefficient between ground displacements at two stations *k* and *l*. It depends on the cross-power spectral density of displacements D_{gk} and D_{gl}. It is equal to unity in the case of uniform excitation, and defined by:

$$\rho_{D_{gk}D_{gl}} = \frac{1}{\sigma_{D_{gk}}\sigma_{D_{gl}}}\int_{-\infty}^{+\infty} S_{D_{gk}D_{gl}}(\omega)d\omega \tag{3.2}$$

$\rho_{D_{gk}S_{lj}}$ is the cross-correlation coefficient between the displacement at support *k* and the response of oscillator (ω_j, ς_j) at support *l*. It depends on the dynamic properties of both soil and oscillator, and is given by:

$$\rho_{DgkSlj} = \frac{1}{\sigma_{Dgk}\sigma_{Slj}} \int_{-\infty}^{+\infty} H_j(\omega) S_{Dgk\ddot{D}gl}(\omega)d\omega \tag{3.3}$$

ρ_{SkiSlj} is the cross-correlation coefficient between the two oscillators (ω_j, ς_j) and(ω_i, ς_i).It depends on the power spectral density of individual support accelerations $\ddot{D}_{gk}$ and $\ddot{D}_{gl}$, and coherency function. It is given by:

$$\rho_{S_{ki}S_{lj}} = \frac{1}{\sigma_{S_{ki}}\sigma_{S_{lj}}} \int_{-\infty}^{+\infty} \Lambda_i^*(\omega)\Lambda_j(\omega) S_{\ddot{D}_{gk}\ddot{D}_{gl}}(\omega)d\omega \tag{3.4}$$

$\sigma_{D_{gk}}$ is the variance of soil displacement D_{gk} and given by $\sigma_{D_{gk}}^2 = \int_{-\infty}^{+\infty} S_{D_{gk}D_{gk}}(\omega)d\omega$,and $\sigma_{S_{ki}}$ is the variance of normalised modal response S_{ki}, given by $\sigma_{S_{ki}}^2 = \int_{-\infty}^{+\infty} |\Lambda_i(\omega)|^2 S_{\ddot{D}_{gk}\ddot{D}_{gk}} d\omega$.

$H_i(\omega)$ is the model frequency response function of the mode *i* defined by $\Lambda_i(\omega) = \left[\omega_i^2 - \omega^2 + 2i\xi_i\omega_i\omega\right]^{-1}$, $S_{xy}(\omega)$ is the cross-power spectral density of processes *x* and *y*, related to auto power spectral densities by the relation $S_{\ddot{D}_{gk}\ddot{D}_{gl}}(\omega) = \gamma_{kl}^{SRE}(\omega)\sqrt{S_{\ddot{D}_{gk}\ddot{D}_{gk}}(\omega) S_{\ddot{D}_{gl}\ddot{D}_{gl}}(\omega)}$

$\gamma_{kl}^{SRE}(\omega)$ is the coherency function given by the eqn. 2.2.

4. SENSITIVITY ANALYSIS OF SITE EFFECTS ON BRIDGE RESPONSES

Sensitivity analysis of the site response effect, due to the difference in local soil conditions, on stochastic response of the Oued Dib Bridge, in Mila Region at East of Algeria is done. The responses under multiple vertical excitations are compared to the cases of uniform excitation where all supports are assumed to be founded on soft soil in one case and firm soil in other case. The excitation is represented by corresponding power spectral density. The variable excitation is considered by assuming the abutments founded on stiff soil and the piers on soft soil. The contribution to the total response of the cross correlation between components (pseudo-static and dynamic) is neglected; it contributes with marginal manner to the total response (less than 4%) (Mezouer, 2010).

4.1. Description of the bridge model

The cable stayed bridge of Oued Dib, in the east of Algeria has three spans; the main span of 280m and two side spans of 111m (Fig.4.1). The prestressed concrete deck is 13.30m wide, supported by two frames (piles) with H form and 110 and 140m height respectively. The 88 stays are converged at the top of the frame towers. To analysis the site effects on stochastic responses of the bridge, a SAP2000 numerical model of the bridge is used. The deck is modeled with 41 equivalent beam elements and 82 link elements. The piles were modeled by 150 beam elements and the stays modeled as cables. Finally, the model is represented by 272 nodes, 191 beam elements, 80 cables and 82 link elements.

The mechanical properties of the model are listed in the table 4.1. Five percent of damping is adopted for the response calculation. The results of modal analysis are given in table 4.2.

Table 4.1. Mechanical characteristics of the bridge structural members

	Elasticity modulus E (kN/m²)	Density (kN/m^3)	Poisson coefficient
deck	39x10^6	25	0.2
cables	190 x 10^6	80	0.3
piers	39x10^6	25	0.2

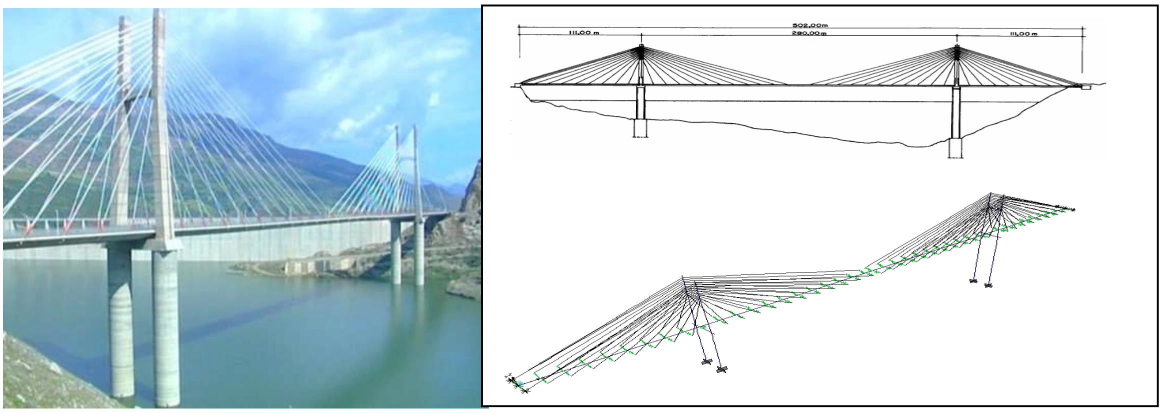

Figure 4.1. General view and mathematical model of Oued Dib bridge

Table 4.2. Periods and frequencies of the bridge

Mode	Period (Sec)	Frequency (Hz)	Sense	Mode	Period (Sec)	Frequency (Hz)	Sense
1	3,69222	0,27084	Vertical	7	1,510077	0,66222	/
2	3,362713	0,29738	Transversal	8	1,496822	0,66808	/
3	2,728865	0,36645	Longitudinal	9	1,398246	0,71518	/
4	2,136481	0,46806	Transversal	10	1,178454	0,84857	/
5	1,896026	0,52742	Longitudinal	11	1,154222	0,86638	/
6	1,603868	0,62349		12	0,996017	1,004	/

4.2. Soil variability description

The geotechnical investigation showed that the Oued Dib site is essentially an alternation of Paleocene marl and calcareous. The alluvium deposits constitute the river bed (LNHC, 1999). The abutments are founded on the first soil type, considered as firm soil (F). The geotechnical report indicates only an average limit pressure of 2,7Mpa and simple compression resistance of 47Mpa. A simple correlation of these values gives an average shear wave velocity of 450m/s (Mezouer et al., 2010). The piers are founded on the river bed, the soil is considered as soft (M) with shear wave velocity around 200m/s. The bridge responses under variable excitations due to the situation of soil variability described before are compared to responses to uniform excitation. The uniform excitation is obtained by assuming the soil underneath the foundations to be either firm type or soft type. The following three combinations of local soil conditions are examined: FFFFF, MMMM and FMMF, where the first and the last letter indicate the soil type underneath the abutments while the two central letters indicate the soil underneath the piers.

4.3. Numerical results

Stochastic analysis of the bridge is performed for spatially varying ground motions due to the site effects and by assuming that the soil conditions are inhomogeneous, soft under piles and firm at abutments. The characteristics of each column of 50m thickness, overlying bedrock with shear wave velocity of 800m/s and density of 2.2, are given in table 4.3.

The power density spectral (PSD) function of accelerations at supports is the one of Kanai-Tajimi modified by Clough and Penzien. The filter parameters of soil profiles are estimated with analytical approximation (eqn. 2.5) using the amplification functions of soil profiles under each support (Firm soil, $p = 0.0$ and soft soil, $p = 0.6$). The choice of the excitation spectra amplitude S_0 at the bedrock is based on the maximal acceleration at surface which has not to exceed 0.25g. Fig 4.2a and 4.2b show the amplification functions for the two type of soil and the corresponding PDS functions. The coherency function, translating the site response effect, is completely defined in term of the amplification functions and given in fig 4.2c.

Table 4.3. Soil columns characteristics

$V_0(m/s)$	$\rho(kg/m^3)$	p	q	ω_g	ξ_g	ω_f	ξ_f
450	1900	0.0	0.55	17.19	0.34	1.72	0.6
200	1750	0.6	0.20	5.42	0.08	0.54	0.6

The vibration of the bridge under vertical excitations at supports is analyzed. The responses quantities along the deck considered are vertical displacement, bending moments around transversal axis and shear forces. For the piles, the considered responses are axial displacement, bending moment and normal forces.

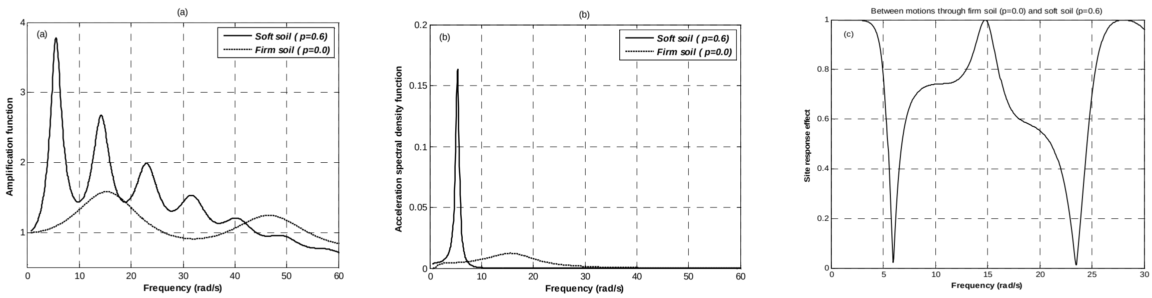

Figure 4.2 : *Amplification functions (a), DSP (b) and site effect coherency (c) for the two profiles*

4.3.1. Effects on deck responses

The next figures show the maxima of response components (dynamic, pseudo-static and total) along the bridge deck for three cases of excitation:

- Uniform excitation considering soft soil at all supports (MMMM)
- Uniform excitation considering firm soil at all supports (FFFF)
- Variable excitation due to soft soil under piles and firm soil under abutments (FMMF)

The pseudo static components of the displacement are presented in Fig. 4.3a. This component is constant because of a rigid body movement in case of uniform excitation and varies by increasing from the maximum soil displacement at abutments which in firm to the ones at piles where the soil is considered as soft. The dynamic components (Fig. 4.3b) are maximal around the middle span and vanish at extremities. The variable excitation generates more important dynamic components; this is completely different to the case of spatial variability of ground motion without considering site effect (Mezouer, 2010), because in this case dynamic response depends essentially on the structural characteristics and in case of including site effects, the dynamic response is influenced by the soil characteristics as well. The total displacement response (Fig. 4.3c) is more important in case of variable excitation over a big part of the deck and especially at mid-span. Around the extreme supports, the uniform case considering soft soil generates higher displacements.

The bending moment pseudo static components are presented in Fig. 4.4a. This component is nil in case of uniform excitation and appears relatively important especially at the central part of the bridge deck when the site effects are considered. The dynamic components (Fig.4.5b) of the bending moment are more important in case of variable excitation than the case of uniform soil. The difference is more important considering firm soil. Around the abutments part, the situation is sometimes inversed. The superposition of the two components (Fig. 4.5c), dynamic and pseudo-static, makes the difference in the total response between cases of excitation more important.

The shear forces pseudo-static components (Fig. 4.5a) are nil in case of uniform excitation and appear not negligible along a big part of the deck. The dynamic component (Fig. 4.5b) induced by variable excitation exhibits higher values than uniform excitation in general. Some exceptions are observed at sections between abutments and half of short spans. The total shear forces (Fig. 4.5c) induced by variable excitation are more important along the bridge deck except at the extreme quarters of short spans where the uniform excitation considering soft soil gives higher total shear forces.

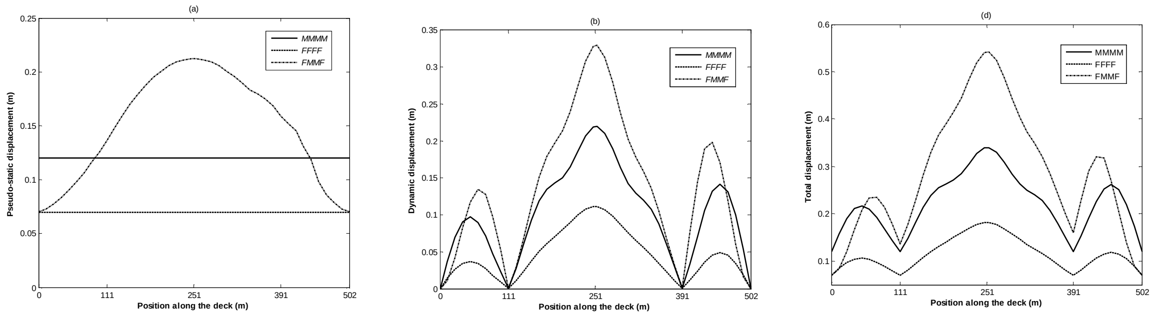

Figure 4.3. Displacement components, pseudo-static (a), dynamic (b) and total (c)

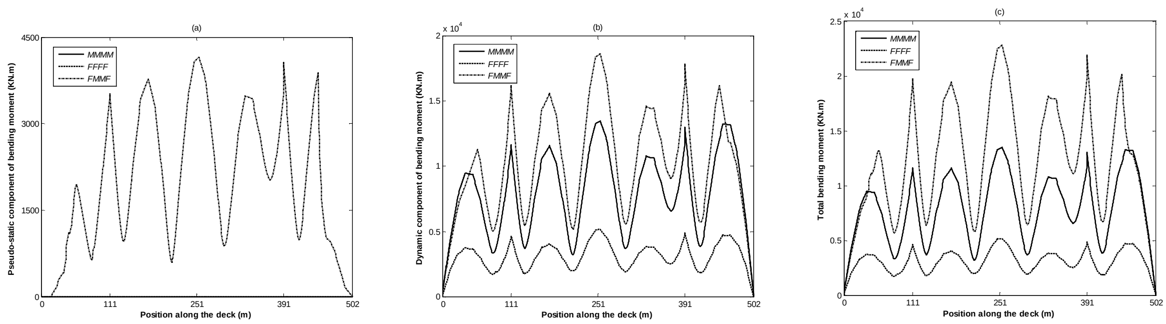

Figure 4.4. Bending moment components, pseudo-static (a), dynamic (b) and total (c)

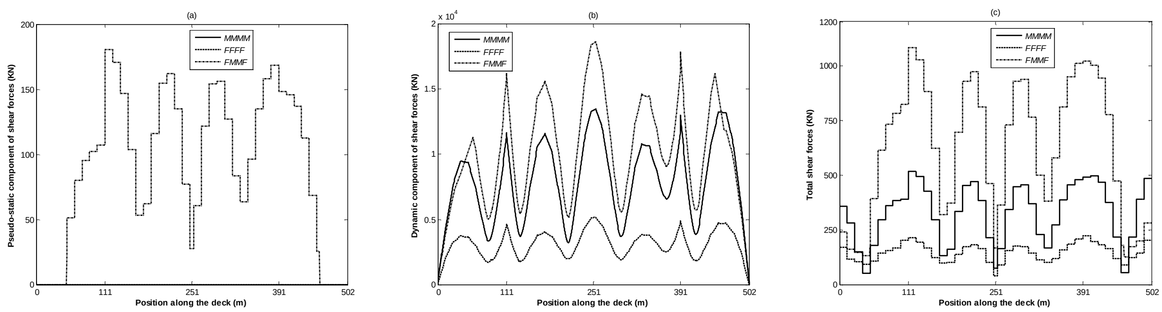

Figure 4.5. Shear forces components, pseudo-static (a), dynamic (b) and total (c)

4.3.2. Effects on pylons responses

The axial pseudo-static displacement components at the two pylons, left and right are presented in figure 4.6a. The uniform excitation cases (FFFF, MMMM) induce constant displacements at all the pylon height. In presence of variable excitation, the pseudo static component is maximal at the top and support and approaches zero at the intersection with the deck. The piles are founded on soft soil in the case of variable excitation; their maximal values are equal to the soil displacement. The figure 4.6b shows the axial dynamic components of the right pylon (higher one), they are maximal at the top and decrease with descending. They exhibit a variation at the sudden variation of the pylon section. At the lower part of the pylon, the uniform excitation induces important dynamic axial displacements considering soft soil and low displacement considering firm soil. The variable excitation is between the two cases. Above the bridge deck, the dynamic axial displacement is more important in presence of site effects than other cases. The same remarks can be observed on the left pylon (Fig. 4.6c).

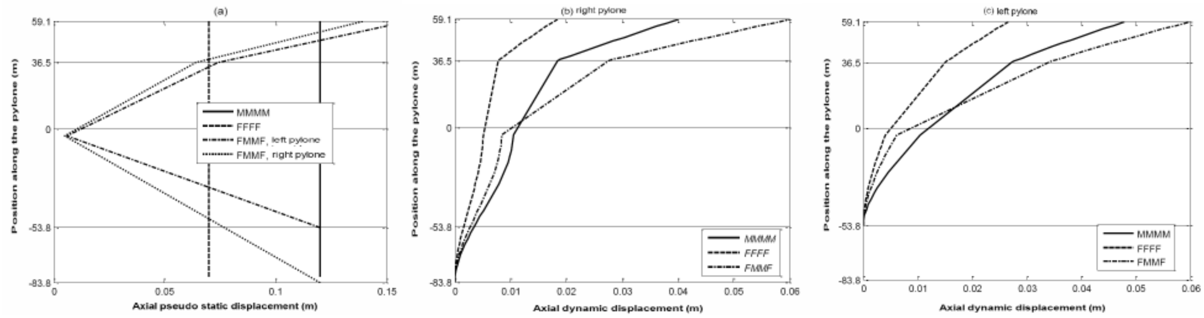

Figure 4.6. Axial displacement components, pseudo-static (a), dynamic right pylon(b) and left pylon (c)

5. CONCLUSION

This paper outlines a sensitivity analysis of the stochastic response of a cable-stayed bridge subjected to site effects due to variation of the soil conditions under different supports, the abutments are founded on firm soil and the piles on soft soil. The analysis is applied to the cable stayed bridge of Oued Dib, in the east of Algeria with a length of 502m. The variance values of responses in term of displacement, bending moment and shear forces are obtained and compared to the variance responses considering uniform excitation represented by to cases, firm or soft soil under all supports.
From the results obtained in this modest study, general conclusions can be issued on the responses along the bridge deck and the height of the piles under variable excitation and uniform ones.
Taking into account the site effect induces total displacement around the bridge abutments lower than the ones induced by uniform excitation. Away the abutments, the uniform excitations underestimate the total displacements. This underestimation is more pronounced in the case of firm soil.
The forces along the central span and around the pylons induced by spatial variability of ground motion are higher than those induced by uniform excitation, this difference is more important when we consider firm soil. Around the abutments, the variable excitation gives lower forces.
The axial displacements in the pylons are always maximal at tops and present variations at sudden variation of pylon's section. At the lower part of the pylon, the uniform excitation induces important dynamic axial displacements considering soft soil and lower displacement considering firm soil. Above the bridge deck, the dynamic axial displacement is more important in case of variable excitation due to site effects.

REFERENCES

Abdel-Ghaffar A.M. and Rubin L.I. (1982). Suspension bridge response to multiple support excitations. *Journal of the Engineering Mechanics Division, ASCE*, **108**, 419– 435.

Abdel-Ghaffar A.M. and Rubin L.I. (1983). Lateral earthquake response of suspension bridges. *Journal of Structural Engineering, ASCE*, **109**, 665–675.

Abrahamson N.A., Schneider J.F. and Stepp J.C. (1991) Empirical spatial coherency functions for applications to soil-structure interaction analyses. *Earthquake Spectra*,**7**, 1–27.

Afra H. and Pecker A. (2002). Calculation of free field response spectrum of a non-homogeneous soil deposit from bed rock response spectrum. *Soil Dynamics and Earthquake Engineering*, **22**, 157–165.

Berrah, M.K. and Kausel, E. (1992). Response Spectrum Analysis of Structures Subjected to Spatially Varying Motions. *Earthquake Engineering and Structural Dynamics*, **21-6**, 461-470.

Der Kiureghian A. (1996). A coherency model for spatially varying ground motions. *Earthquake Engineering and Structural Dynamics*, **25**, 99–111.

Der Kiureghian A. and Neuenhofer A. (1992). Response Spectrum Method for Multi-Support Seismic

Excitations. *Earthquake Engineering and Structural Dynamics*, **21**, 715-740.
Der Kiureghian A., Keshishian P. and Hakobian A. (1997). Multiple support response spectrum analysis of bridges including the site response effect & MSRS code, *Report N° UCB/EERC-97/02,* University of California, Berkeley.
Dumanoglu A.A. and Soyluk K. (2003) A stochastic analysis of long span structures subjected to spatially varying ground motions including the site-response effect. *Engineering Structures,* **25**, 1301–1310.
Gazetas, G. (1984) Seismic response of end-bearing single piles. *Soil dynamics and Earthquake Engineering* **3-2** 82-93.
Hadid M. and Afra H. (2000) Sensitivity analysis of site effects on response of pipelines. *Soil Dynamics and Earthquake Engineering*, **20**, 249–260.
Harichandran, R.S. (1991). Estimating the spatial variation of earthquake ground motion from dense array recordings. *Structural Safety,* **10**, 219-233.
Harichandran R.S. and Vanmarcke E.H. (1986). Stochastic variation of earthquake ground motion in space and time. *Journal of Engineering Mechanics, ASCE*, **112**, 154–174.
Idriss I.M., and Seed H.B. (1968) Seismic response of horizontal layers. *Journal of the Soil Mechanics and Foundations Division,* ASCE, **94**,1003-1031.
Laouami N. and Labbé P. (2001). Analytical approach for evaluation of the seismic ground motion coherency function. *Soil Dynamics and Earthquake Engineering*, **21**, 727–733.
LNHC. (1999) Rapport géotechnique de l'étude du site de la culée nord du viaduct de Beni Haroun, Mila. *Soil report*
Luco, J.E. and Wong, H.L. (1986). Response of a rigid foundation to a spatially random ground motion. *Earthquake Engineering and Structural Dynamics.* **14** 891-908.
Lupoi A., Franchin P., Pinto P.E. and Monti G. (2005). Seismic Design of Bridges Accounting for Spatial Variability of Ground Motion. *Earthquake Engineering and Structural Dynamics,* **34**, 327-348.
Mezouer N. (2010). Effets de la variabilité spatiale du mouvement sismique sur la réponse des structures, *thèse de Doctorat d'Etat, Ecole Nationale Polytechnique d'Alger*.
Mezouer N. Silhadi K. and Hadid M. (2011) Sensitivity analysis of site response effect on stochastic responses of two-span beam. *Advances in Structural Engineering*, **14:3**, 413-425.
Monti G., Nuti C., Pinto P. and Vanzi I. (1994). Effects of non-synchronous seismic input on the inelastic response of bridges. *Proceedings of the 2nd International Workshop on Seismic Design of Bridges*, Queenstown, New Zealand **1**, 90-107.
Sextos A. (2001). Effect of spatial variability, local soil conditions and soil-structure interaction on the inelastic dynamic analysis of RC bridges, *Ph.D. Thesis*, Aristotle University of Thessaloniki, Greece.
Sextos A., Pitilakis K.D. and Kappos A.J. (2003). Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil–structure interaction phenomena. Part 1: Methodology and analytical tools. *Earthquake Engineering and Structural Dynamics*, **32**, 607-628.
Soyluk K. and Sicacik E.A. (2012) Soil–structure interaction analysis of cable-stayed bridges for spatially varying ground motion components. *Soil Dynamics and Earthquake Engineering*, **35**, 80–90.
Zanardo G., Hao H. and Modena C. (2002). Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion. *Earthquake Engineering and Structural Dynamics*, **31**, 1325-1345.
Zerva A. (1990) Response of multi-span beams to spatially incoherent seismic ground motions. *Earthquake Engineering and Structural Dynamics*, **19**, 819–832.
Zerva A. Ang A. H-S. and Wen Y.K. (1988) Lifeline response to spatially variable ground motions. *Earthquake Engineering and Structural Dynamics*, **16**, 361–379.