SEISMIC RESPONSE ANALYSIS OF A BRIDGE FRAME AT A CANYON SITE IN WESTERN AUSTRALIA

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ABSTRACT :

Seismic responses of large dimensional structures excited by spatially varying ground motions are studied by many researchers. Most of previous studies assumed a flat site and uniform site properties. Studies of the combined site amplification and ground motion spatial variation effects on structural responses are limited. This paper investigates the effects of spatially varying ground motions on responses of a PC bridge frame located at a canyon site in Perth, Western Australia. The ground motions at base rock are assumed to have the same intensity but vary spatially. They are modelled with an earthquake ground motion attenuation model derived for Western Australia and a coherency loss function. Site amplification effect is modelled by an explicit transfer function based on one dimensional wave propagation theory. Quasi-static, dynamic and total responses of the bridge frame are calculated with the stochastic method. Numerical results of various response quantities to spatial ground motions are presented. The importance of the site and ground motion spatial variation effect on bridge frame responses is highlighted.

KEYWORDS: ground motion spatial variations, mean peak responses, site effects, soil conditions

1. INTRODUCTION

Ground motions at multiple supports of large dimensional structures are different because of wave passage effect, the loss of coherency and site effect. Many efforts have been spent on modelling the ground motion spatial variations (Bolt 1982, Abrahamson 1985 and Hao 1989) and their effects on structure responses, e.g. Harichandran and Wang (1988 and 1990), Zerva (1990), Hao (1994), Hao and Duan (1995 and 1996), Ates et al. (2005). Most of these studies assumed a flat site and ground motion spatial variations were modelled by a coherency loss function and phase shift owing to wave passage effect, the influence of site effect was ignored in these studies.

Taking the local site conditions into consideration, Der Kiureghian et al. (1991 and 1997) proposed a transfer function that implicitly models the site effect on seismic wave propagation. Hao and Chouw (2006) modelled the site amplification by a transfer function based on one dimensional wave propagation theory. Compared to the model by Der Kiureghian, the base rock motion in the latter study is the Tajimi-Kanai power spectral density function instead of a white noise, and the wave propagation and site amplification effects are explicitly represented in terms of the site conditions such as the depth and soil properties. Based on the model developed by Der Kiureghian (1997), Zembaty and Rutenburg (2002) derived the displacement and shear force response spectra with consideration of ground motion spatial variation and site effects. Dumanogluid and Soyluk (2004) analysed responses of a long span structure to spatially varying ground motions with site effect. They concluded that ground motion spatial variation and site effects significantly affect the structural responses.

This paper calculates the mean peak dynamic, quasi-static and total responses of a bridge frame located at a canyon site in Perth, Western Australia. The base rock ground motion intensity is modelled by a stochastic ground motion attenuation model proposed for Southwest Western Australia (Hao and Gull 2004). The base rock spatial variation is modelled by a theoretical coherency loss function and the site amplification effect is modelled by the transfer function proposed by Hao and Chouw (2006). Different soil depth and soil properties are considered in the paper, comparisons and discussions on their effects on structural responses are made. The importance of site effect is highlighted.

2. SPATIAL GROUND MOTION MODEL

The bridge frame crossing a canyon site as shown in Figure 1 is considered. In the figure, A and B are the two supports on ground surface, the corresponding points at base rock are A' and B', d is the horizontal distance between the two supports and h_j is the depth of the soil layer under the jth support, where j represents A or B. ρ_R, v_R and ξ_R represent density, shear wave velocity and damping ratio of the base rock, the corresponding parameters of soil layer are ρ_j, v_j and ξ_j.

Assume ground motion intensity at base rock is the same, with its power spectral density (Hao and Gull 2004) and coherency loss function as (Hao and Chouw 2006)

$$\begin{aligned} S_R(\omega) &= \frac{1}{2\pi T}[S(M_0,\omega)D(R,\omega)P(\omega)]^2 \\ \gamma_{A'B'}(i\omega) &= e^{-\beta\omega d^2/v_{app}} \cdot e^{-i\omega d/v_{app}} = \left|\gamma_{A'B'}(i\omega)\right| e^{-i\omega d/v_{app}} \end{aligned} \tag{2.1}$$

where ω is the circular frequency, $S(M_0,\omega)$ is the earthquake source spectrum, $D(R,\omega)$ is an attenuation function, $P(\omega)$ is a filter function and T is the ground motion duration, M_0 is the seismic moment, which is a function of Richter magnitude (Atkinson and Boore 1995). β is a coefficient depending on the level of coherency loss, v_{app} is the apparent wave propagation velocity.

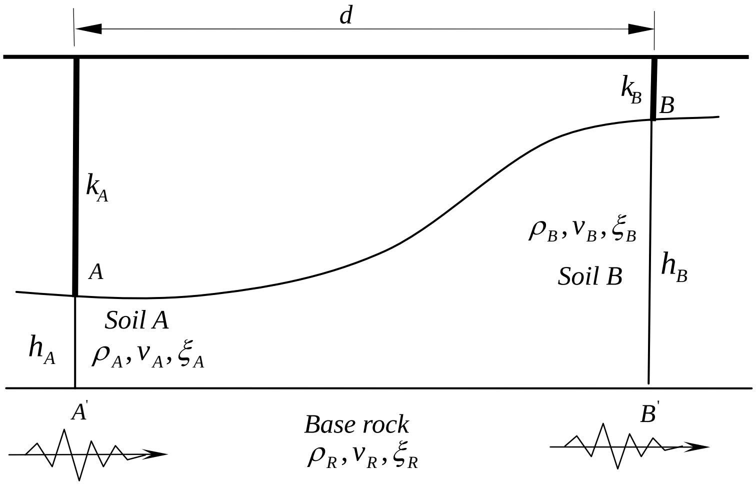

Figure 1 Schematic view of a bridge span crossing a canyon site

Based on one dimensional wave propagation assumption, it can be derived that the transfer function of ground motion due to wave propagation from base rock j' to ground surface j is (Hao and Chouw 2006)

$$H_j(i\omega) = \frac{(1 + r_j - i\xi_j)e^{-i\omega\tau_j(1-2i\xi_j)}}{1 + (r_j - i\xi_j)e^{-2i\omega\tau_j(1-2i\xi_j)}} \tag{2.2}$$

where $\tau_j = h_j / v_j$ is the wave propagation time from point j' to j, and $r_j = (\rho_R v_R - \rho_j v_j)/(\rho_R v_R + \rho_j v_j)$ is the reflection coefficient for up-going waves. The power spectral density function at point j and the cross power spectral density function between A and B is thus

$$\begin{gathered} S_j(\omega) = \left|H_j(i\omega)\right|^2 S_R(\omega) \\ S_{AB}(i\omega) = H_A(i\omega)H_B^*(i\omega)S_{A'B'}(i\omega) \end{gathered} \tag{2.3}$$

in which the superscript '*' represents complex conjugate.

3. STRUCTURAL RESPONSES

The primary objective of this study is to investigate the site effect on responses of multiply-supported structures, a one-span bridge frame shown in Figure 1 is used as an example. It is simplified to a rigid beam supported by two columns of stiffness k_A and k_B. Neglecting soil-structure interaction effect, the dynamic equilibrium equation can be written as

$$\begin{bmatrix} m & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{v}^t \\ \ddot{u}_A \\ \ddot{u}_B \end{Bmatrix} + \begin{bmatrix} c & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} \dot{v}^t \\ \dot{u}_A \\ \dot{u}_B \end{Bmatrix} + \begin{bmatrix} k_A + k_B & -k_A & -k_B \\ -k_A & k_A & 0 \\ -k_B & 0 & k_B \end{bmatrix} \begin{Bmatrix} v^t \\ u_A \\ u_B \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} \tag{3.1}$$

where m is the lumped mass at the deck level, v^t is the total displacement response, and u_A and u_B are the ground displacement at support A and B, respectively. The total response v^t consists of dynamic response v and quasi-static response v^{qs}, and the quasi-static response can be derived as

$$v^{qs} = \frac{1}{k_A + k_B}\begin{bmatrix} k_A & k_B \end{bmatrix}\begin{bmatrix} u_A \\ u_B \end{bmatrix} = \begin{bmatrix} \varphi_A & \varphi_B \end{bmatrix}\begin{bmatrix} u_A \\ u_B \end{bmatrix} = (\varphi_A u_A + \varphi_B u_B) \tag{3.2}$$

in which $\varphi_A = k_A/(k_A + k_B)$ and $\varphi_B = k_B/(k_A + k_B)$. The dynamic response can be obtained by solving the dynamic equilibrium equation

$$m\ddot{v} + c\dot{v} + (k_A + k_B)v = -m\ddot{v}^{qs} = -m(\varphi_A \ddot{u}_A + \varphi_B \ddot{u}_B) \tag{3.3}$$

Transfer Eqn. 3.3 into the frequency domain, the dynamic response can be obtained by

$$\bar{v}(i\omega) = \frac{-1}{\omega_0^2 - \omega^2 + 2i\xi_0\omega_0\omega}\bar{\ddot{v}}^{qs}(i\omega) = -H_s(i\omega)\bar{\ddot{v}}^{qs}(i\omega) = -H_s(i\omega)[\varphi_A \bar{\ddot{u}}_A(i\omega) + \varphi_B \bar{\ddot{u}}_B(i\omega)] \tag{3.4}$$

in which $\omega_0 = \sqrt{(k_A + k_B)/m}$ is the circular natural vibration frequency of the structure, ξ_0 is the damping ratio, and $H_s(i\omega)$ is the transfer function of the structure.

The power spectral density function of dynamic response, quasi-static response and total response can then be derived as

$$\begin{aligned} S_v(\omega) &= |H_s(i\omega)|^2 \left\{\varphi_A^2 S_A(\omega) + \varphi_B^2 S_B(\omega) + 2\varphi_A\varphi_B \,\mathrm{Re}[S_{AB}(i\omega)]\right\} \\ S_{v^{qs}}(\omega) &= \frac{1}{\omega^4}\left\{\varphi_A^2 S_A(\omega) + \varphi_B^2 S_B(\omega) + 2\varphi_A\varphi_B \,\mathrm{Re}[S_{AB}(i\omega)]\right\} \\ S_{v^t}(\omega) &= S_v(\omega) + S_{v^{qs}}(\omega) - \frac{2}{\omega^2}\mathrm{Re}\left\{H_s(i\omega)\left(\varphi_A^2 S_A(\omega) + \varphi_B^2 S_B(\omega) + 2\varphi_A\varphi_B \,\mathrm{Re}[S_{AB}(i\omega)]\right)\right\} \end{aligned} \tag{3.5}$$

in which 'Re' denotes the real part of a complex number. In this study, the uniform ground motion is assumed to be the same as u_A. Under uniform ground excitation, Eqn. 3.5 reduces to

$$\begin{aligned} S_{v_u}(\omega) &= |H_s(i\omega)|^2 S_A(\omega) \\ S_{v_u^{qs}}(\omega) &= \frac{1}{\omega^4} S_A(\omega) \\ S_{v_u^t}(\omega) &= S_{v_u}(\omega) + S_{v_u^{qs}}(\omega) - \frac{2}{\omega^2}\mathrm{Re}[H_s(i\omega)S_A(\omega)] \end{aligned} \tag{3.6}$$

After obtaining the power spectral density function of each response quantity, the mean peak response can be calculated based on the standard random vibration method (Der Kiureghian 1980).

4. NUMERICAL RESULTS AND DISCUSSIONS

The bridge frame shown in Figure1 is considered in the paper. The power spectrum of ground motion on the base rock is assumed to be the model derived for Western Australia by Hao and Gaull (2004) with $\rho_R = 2500 kg/m^3$, $v_R = 2000 m/s$ and $\xi_R = 0.05$. The Richter magnitude considered is *ML*7.5, epicentre distance 100 *km*, focal depth 5 *km* and shear wave velocity of the focal area is 3910 *m/s*. The horizontal distance between the two supports *A* and *B* is set to be d=40 *m*, viscous damping ratio is assumed to be 5%, and the stiffness of the two piers are assumed to be the same, i.e. $k_A = k_B$. The coefficient $\beta = 0.002$ is assumed which represents weakly correlated ground motion, and the apparent wave velocity v_{app}=500 *m/s* is considered in the paper.

The effect of soil depth on the structure responses is investigated first. In these cases, the soil under both site *A* and *B* are assumed to be medium soil with $\rho_A = \rho_B = 2000$ kg/m^3, $v_A = v_B = 450 m/s$ and $\xi_A = \xi_B = 0.05$. As shown in Figure 2, different soil depths lead to different transfer functions. The peaks occur at the corresponding vibration frequencies of the sites. The deeper is the soil, the more flexible is the site, and the lower is the fundamental vibration frequency. The transfer function directly alters the power spectral density function at ground surface, as shown in Figure 3. Motions on ground surface have a narrower band, but higher peak, as compared to that at the base rock, indicating the effect of site filtering and amplification on base rock motion.

Dynamic response, quasi-static response, and total response with varying structural vibration frequencies are calculated, and normalized by the corresponding responses to uniform excitation, which is defined as the motion at Point *A*, as discussed above. Figure 4 and 5 show respectively the normalized dynamic responses and total responses with respect to the dimensionless parameter, $f_0 t_d$, where f_0 is the vibration frequency of the structure, $t_d = d/v_{app} + \tau_B - \tau_A$, τ_B and τ_A are time required for wave to propagate from *B'* to *B* and *A'* to *A*, respectively. This parameter measures the relation between phase shift or time lag of spatial ground motions at points *A* and *B* and the fundamental vibration frequency f_0. If the ground surface is flat, $f_0 t_d = f_0 d/v_{app}$.

As shown in Figure 4, if the site is flat ($h_A = h_B$), the normalized dynamic responses reach their minimum value at $f_0 t_d = 0.5, 1.5$ and reach their maximum value at $f_0 t_d = 1.0, 2.0$ because of the out-of-phase and in-phase ground motion inputs. If support *A* locates on the base rock and support *B* locates on the soil layer (h_A=0 *m*, h_B=30 or 50 *m*), the maximum responses, however, do not appear at $f_0 t_d = 1.0$. This is because of the dominance of site amplification effect on ground motions and resonant responses. The peaks appear when the structure is resonant with the soil site. For example, when h_A=0 *m*, h_B=30 *m*, the first peak occurs at $f_0 t_d = 0.55$, or $f_0 = 3.75 Hz$ because $t_d = d/v_{app} + \tau_B = d/v_{app} + h_B/v_B = 0.14667$ sec. As shown in Figure 2 and 3, site *B* with soil depth of 30 m has the fundamental natural vibration frequency of 3.75 Hz. Similar conclusions can be obtained when h_A=0 *m*, h_B=50 *m*. Besides this resonant peak, the ground motion spatial variation effect is prominent with local maximum occurring at $f_0 t_d = 2.0, 3.0$ and local minimum at $f_0 t_d = 1.5, 2.5$. When h_A=30*m* and h_B=50*m*, the spatial ground motion wave passage effect dominates the site effect on dynamic structural responses, this is because, although site *A* and *B* have different fundamental vibration modes and different peak values in their respective power spectral density function as shown in Figure 3, the mean peak responses to ground motion at site *A* and *B* are similar to each other because they depend on the spectral moments as defined above. Therefore, normalization removes the site amplification effects, which leaves the wave passage effects to govern the normalized dynamic response in this case.

Quasi-static responses are independent of the fundamental vibration frequency of the structure (Eqn.3.5), the normalized quasi-static responses are therefore constant for each case with respect to $f_0 t_d$. The normalized total responses are given in Figure 5. As shown, when the dimensionless parameter $f_0 t_d$ is less than 2.5, the normalized total responses are similar to the normalized dynamic responses, indicating dynamic response dominates the total response. When $f_0 t_d$ increases, however, the normalized responses approach to a constant, equal to the quasi-static response. Neither spatial ground motion wave passage effect, nor the site amplification effect is prominent. This is because increasing $f_0 t_d$ implies the structure becomes stiffer, as f_0 is increased in this study. The dynamic response is smaller when structure is stiffer. At large $f_0 t_d$, quasi-static response dominates the total response. This observation indicates the importance of quasi-static responses to stiff structures.

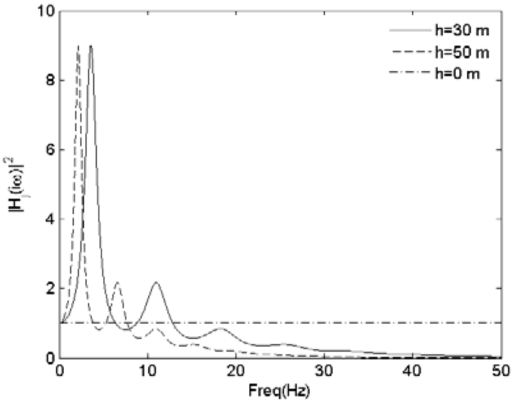

Figure 2 Site transfer functions

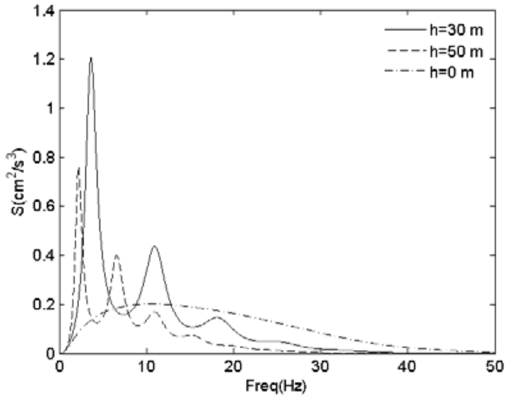

Figure 3 PSD on ground surface

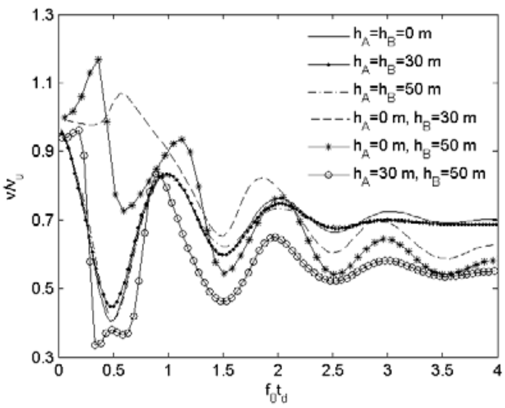

Figure 4 Normalized dynamic responses

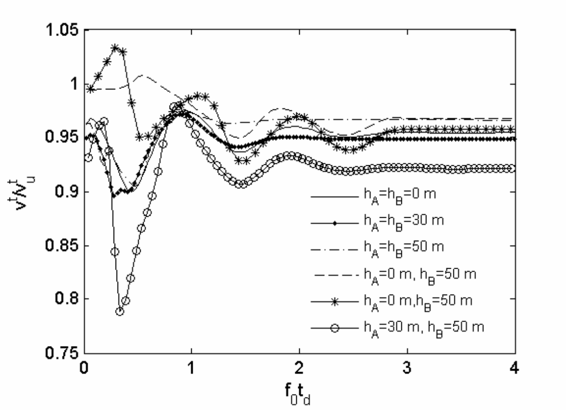

Figure 5 Normalized total responses

To study the effect of soil properties on ground motion spatial variation and hence on structural responses, different soil properties are considered. The soil at site *A* is assumed to be firm soil and unchanged in these cases, while soil *B* varies from firm soil ($\rho_B = 2000$ kg/m^3, $v_B = 450m/s$ and $\xi_B = 0.05$), medium soil ($\rho_B = 1500$ kg/m^3, $v_B = 300m/s$ and $\xi_B = 0.05$) to soft soil ($\rho_B = 1500$ kg/m^3, $v_B = 100m/s$ and $\xi_B = 0.05$).

Figure 6 clearly shows the site effects again. As shown, the peak value of the transfer function increases, while the frequency band becomes narrower with the decrease of the site stiffness. This directly affects the ground motions on ground surface, resulting in substantial spatial variations of ground motions at Points *A* and *B*. Soft soil significantly amplifies the ground motions at its resonant frequencies, firm soil also amplifies ground motions, but at higher frequencies and with a less extent. As a result, the ground motion power spectral densities at ground surface are very different as shown in Figure 7.

Again, as shown in Figure 8, when the vibration frequency of the structure is low, site effect dominates the dynamic responses if the soil properties of site *A* and *B* are different from each other significantly, i.e. the peaks occur when the structure resonates with the soil site. For example, when it is medium soil at site *B*, the first peak occurs at $f_0t_d=0.27$, or $f_0=1.5Hz$ because $t_d=0.18$ sec. As shown in Figure 6 and 7, the fundamental vibration frequency of site *B* is 1.5 Hz. When it is soft soil at site *B*, similar conclusions can be obtained. Subsequent peaks are associated with the out-of-phase excitations and the minimum values are associated with the in-phase excitation. This is because when the structure becomes stiffer, the dynamic response and hence the site resonance effect becomes less significant as compared to the ground motion spatial variation effect, as observed above. As also can be seen in Figure 8, soft soil amplification effect results in larger dynamic responses, normalized dynamic responses are usually larger than 1.0 when the responses are dominated by the site effect, and the results are always less than 1.0 when spatial ground motion phase shift effect governs the dynamic responses. Normalized total responses shown in Figure 9 follow the similar pattern as that for different soil depth, i.e. the normalized total responses are similar to the normalized dynamic responses when $f_0 t_d$ is less than 2.0, however, if the structure is stiff, the dynamic responses are small and the total responses are dominated by the quasi-static responses.

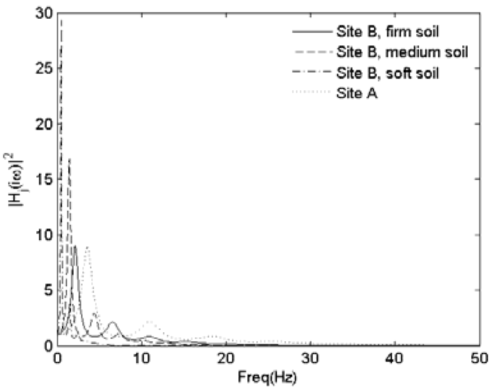

Figure 6 Transfer function of sites

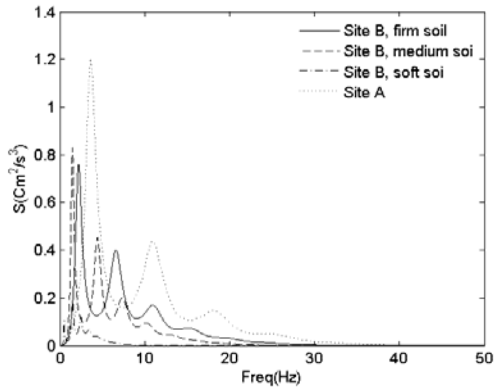

Figure 7 Ground motion PSD

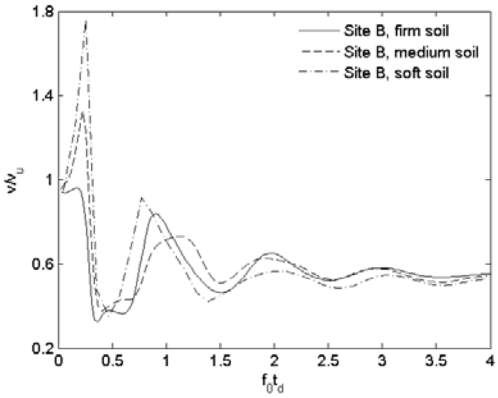

Figure 8 Normalized dynamic responses

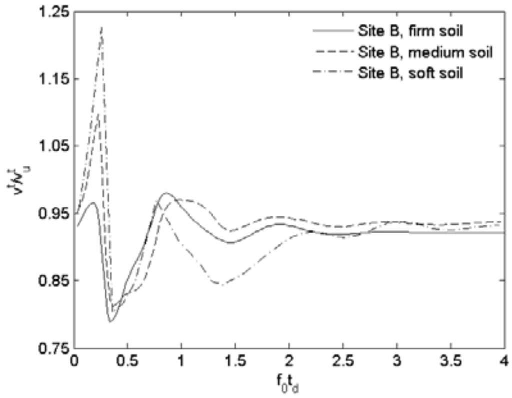

Figure 9 Normalized total responses

5. CONCLUSIONS

This paper studies the effects of different soil depth and different soil properties on the responses of a bridge frame located at a canyon site in WA, Western Australia. Base rock ground motion was modelled using a ground motion attenuation model developed for Western Australia. Ground motion spatial variation was modelled with an empirical coherency loss function. Site amplification effect was estimated by considering one dimensional wave propagation. It is found that varying the soil depth and soil properties of the site result in different site filtering and amplification effects on ground motions. These cause more significant spatial variations of ground motions at multiple bridge supports. Numerical results indicate that site amplification effects govern the dynamic response when structure is relatively flexible owing to resonance, and spatial ground motion wave passage effects dominate the dynamic response when structure is stiff. When structure is flexible, dynamic response dominates the total response, and quasi-static response becomes more important when structure is stiff. Neglecting the site or ground motion spatial variation effects may lead to erroneous prediction of structural responses on an uneven site.

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