Dynamic SSI effect on the required separation distances of bridge structures to avoid seismic pounding

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Abstract

For long-span bridges**,** earthquake ground motions at different supports are inevitably not the same owing to seismic wave propagation effect and different local site conditions. The influence of soil-structure interaction (SSI) cannot be neglected either. This paper studies the combined effect of ground motion spatial variation, local site and SSI on the minimum separation distances that modular expansion joints (MEJs) must provide to preclude seismic pounding. The base rock motions are assumed to have the same intensity. They are modelled with a filtered Tajimi-Kanai power spectral density function and an empirical spatial ground motion coherency loss function. The power spectral density function of surface ground motion of a canyon site is derived by considering the site amplification effect based on one-dimensional wave propagation theory. Soil around the pile group foundation is modelled by the frequency-dependent spring and damper acting in the horizontal and rotational directions. Stochastic response equations of the bridge deck are formulated. The required separation distances are estimated, and the influence of SSI is highlighted.

Keywords: Required separation distance; SSI; wave passage effect; local site effect; MEJ

1. Introduction

Observations from past strong earthquakes revealed that for large-dimensional structures such as long span bridges, pipelines and communication transmission systems, ground motion at one foundation may significantly differ from that at adjacent footing. There are many reasons that may result in the variability of seismic ground motions, e.g. wave passage effect results from finite velocity of travelling waves; loss of coherency due to multiple reflections, refractions and super-positioning of the incident seismic waves; site effect owing to the differences of local soil conditions; additionally to the above, seismic motion is further modified by the constraint of soil movement due to the footing, which is known as kinematic soil-structural interaction (SSI) effect. Seismic ground motion variations may result in pounding or even collapse of adjacent bridge decks owing to the out-of-phase responses.

To preclude pounding effect, the most straightforward approach is to provide sufficient distances between adjacent structures. For bridge structures with conventional expansion joints, a complete avoidance of pounding between bridge decks during strong earthquakes is often impossible. With the new development of MEJ, which allows large relative movement in the joint, precluding pounding between bridge decks becomes possible [1]. Though the MEJ systems have already been used in many new bridges, very limited information on the required separation distance that a MEJ should provide to preclude seismic pounding is available. Chouw and Hao [1] took two independent bridge frames as an example, stressed the influences of SSI and non-uniform ground motions on the separation distance between two adjoined girders connected by a MEJ and then introduced a new design philosophy for a MEJ [2]. In a recent study [3], the authors combined ground motion spatial variation with site effect, studied the minimum total gap that a MEJ must have to avoid seismic pounding at the abutments and between bridge decks. However, these studies either neglected site effect [1, 2] or SSI [3]. To the authors' best knowledge, a comprehensive consideration of ground motion spatial variation, site and SSI effect in the determination of the total gap in MEJs has never been reported. In this paper the simultaneous impact of these influence factors is considered.

The spatial ground motions on the base rock are assumed to have the same intensity. They are modelled by a filtered Tajimi-Kanai power spectral density function. The effect of wave passage and coherency loss on the spatial base-rock ground motions is modelled by an empirical coherency loss function. Site amplification effect is included by a transfer function derived from the one dimensional wave propagation theory. SSI effect is modelled by using the substructure approach. The soil surrounding the pile foundation is described by equivalent frequency-dependent horizontal and rotational spring-dashpot systems. With linear elastic response assumption, the bridge responses are formulated and solved in the frequency domain. The power spectral density functions of the relative displacements between adjacent bridge decks and between bridge deck and abutment are derived, and their mean peak responses are estimated. The minimum total gaps between abutment and bridge deck and two adjacent bridge decks connected by modular expansion joints to avoid seismic poundings are then determined. The effect of SSI is highlighted.

2. Bridge-soil System

The bridge system is the same as that in Reference [3]. The only difference is that dynamic interaction effect between the pile foundation and the surrounding soil is included in this paper. The pier is founded on a rigid cap which is supported by a 2×2 pile group. The diameter of each pile d is 0.6 m, and axis to axis distance between two adjacent piles s is 3 m. The length l of the pile is assumed to be 12 m. The ground surface locations of the bridge supports are denoted as point 1, 2 and 3 as shown in Figure 1. The corresponding points at base rock are 1', 2' and 3'. The depth of soil at the considered sites is assumed to be 50, 30 and 50 m, respectively. The soil at pile-foundation site 3 is modelled as springs and dashpots with the frequency-dependent coefficients k_h, c_h in the horizontal direction and k_r, c_r in the rotational direction. The rigid cap supporting the pier is assumed massless.

The bridge can be modelled as a five-degree-of-freedom system as shown in Figure 1(b): u_1 and u_2 are the bridge deck displacements relative to the free field motion u_{g1} and u_{g2}; u_0 is the horizontal displacement of the pile foundation at site 3 relative to the free field motion u_{g3}; ϕ is the rotation of the pier at the foundation level and u_3 is the displacement of the pier top.

3. Method of Analysis

3.1 Base Rock Motion

The ground motion intensities at points 1', 2' and 3' on the base rock are assumed to be the same in the analysis, and modelled by a filtered Tajimi-Kanai power spectral density function as

$$S_g(\omega) = \left|H_P(\omega)\right|^2 S_0(\omega) = \frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + (2\omega_f \omega \xi_f)^2} \frac{1 + 4\xi_g^2 \omega^2 / \omega_g^2}{(1 - \omega^2/\omega_g^2)^2 + 4\xi_g^2 \omega^2 / \omega_g^2} \Gamma \qquad (1)$$

In this study, it is assumed that $f_f = \omega_f / 2\pi = 0.25$ Hz, $\xi_f = 0.6$, $f_g = \omega_g / 2\pi = 5.0$ Hz, ξ_g =0.6, $\Gamma = 0.022$ $\mathrm{m^2/s^3}$.

The spatial variations of the base-rock ground motions are caused by the wave passage and coherency loss effects. The coherency loss function between points j' and n' (where *j*, *n* represents 1, 2 or 3) derived from the SMART-1 array data by Hao et al. [4] is used. It has the following form

$$\gamma_{j'n'}(i\omega) = \left|\gamma_{j'n'}(i\omega)\right| e^{i\omega d_{j'n'}/v_{app}} = e^{-\beta d_{j'n'}} e^{-\alpha(\omega)\sqrt{d_{j'n'}}(\omega/2\pi)^2} e^{i\omega d_{j'n'}/v_{app}} \qquad (2)$$

in which

$$\alpha(\omega) = \begin{cases} 2\pi a/\omega + b\omega/2\pi + c & 0.314 rad/s \le \omega \le 62.83 rad/s \\ 0.1a + 10b + c & \omega > 62.83 rad/s \end{cases} \qquad (3)$$

The cross power spectral density function of the base-rock motion between points j' and n' is thus

$$S_{j'n'}(i\omega) = S_g(\omega)\gamma_{j'n'}(i\omega) \qquad (4)$$

3.2 Site Effect

Local site conditions have significant effect on structural responses. Hao and Chouw's model [5] with the following expression is used in this paper

$$TF_j(i\omega) = \frac{(1 + r_j - i\xi_j)e^{-i\omega\tau_j(1-2i\xi_j)}}{1 + (r_j - i\xi_j)e^{-2i\omega\tau_j(1-2i\xi_j)}} \qquad (5)$$

in which ξ_j is the damping ratio of soil layer, $\tau_j = h_j / v_j$ is the wave propagation time from point j' to *j*, and r_j is the reflection coefficient for up-going waves

$$r_j = \frac{\rho_R V_{SR} - \rho_j V_{Sj}}{\rho_R V_{SR} + \rho_j V_{Sj}} \qquad (6)$$

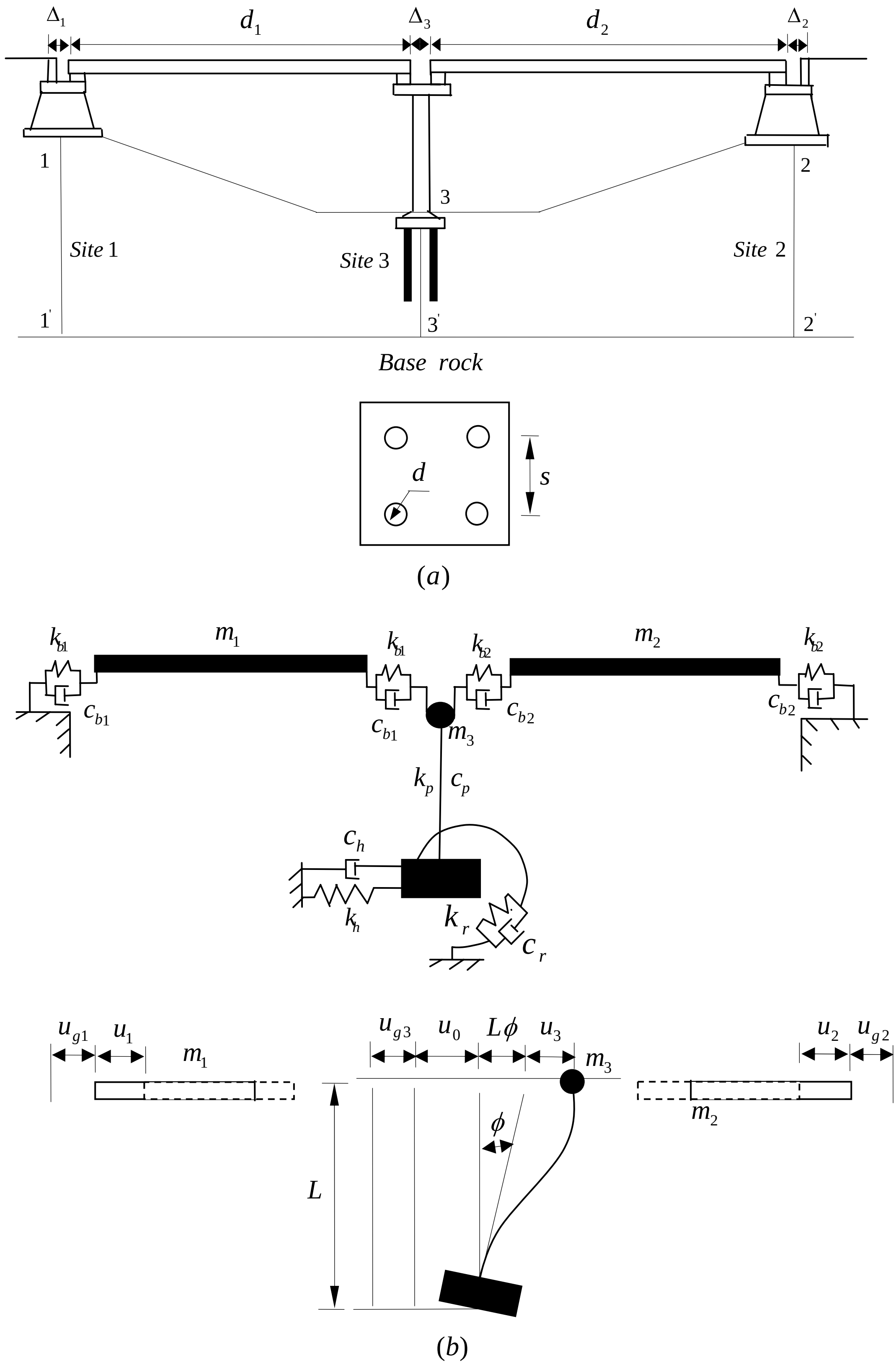

Figure 1. (a) Schematic view of a girder bridge crossing a canyon site; (b) structural model

where ρ_j and V_{Sj} are respectively the density and shear wave velocity of soil at site *j;* ρ_R and V_{SR} are the corresponding parameters of the base rock.

The power spectral density function at point *j* is thus

$$S_j(\omega) = \left|TF_j(i\omega)\right|^2 S_g(\omega) \tag{7}$$

and the cross power spectral density function between *j* and *n* is

$$S_{jn}(i\omega) = TF_j(i\omega)TF_n^*(i\omega)S_{j'n'}(i\omega) \tag{8}$$

where the superscript ‘*’ represents complex conjugate. The free field motions given in Equations (7) and (8) are used as inputs at the foundations.

3.3 Soil-Structure Interaction Effect

The dynamic stiffness of a pile group (K^G) can be calculated using the dynamic stiffness of a single pile (K^S) in conjunction with dynamic interaction factors (α) [6]. The coefficients for the single pile suggested by Gazetas [7] are used, and the analytical solution for α proposed by Dobry and Gazetas [8] is adopted herein.

The frequency-dependent dynamic stiffness and damping coefficient of the pile group can then be estimated as

$$k_h = \mathrm{Re}\left(K_h^G\right), \quad c_h = \mathrm{Im}\left(K_h^G\right)/\omega \tag{9a}$$

in the horizontal direction, and

$$k_r = \mathrm{Re}\left(K_r^G\right), \quad c_r = \mathrm{Im}\left(K_r^G\right)/\omega \tag{9b}$$

in the rotational direction, where K_h^G and K_r^G are the complex stiffnesses of the pile group in the horizontal and rotational direction, respectively. ‘Im’ and ‘Re’ denote respectively the real and imaginary parts of the pile group impedances.

3.4 Structural Response Formulation

The dynamic equilibrium equations of the idealized model in Figure 1(b) can be expressed in the matrix form as follows:

$$\begin{bmatrix} m_1 & 0 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 & 0 \\ 0 & 0 & m_3 & m_3 & m_3 \\ 0 & 0 & m_3 & m_3 & m_3 \\ 0 & 0 & m_3 & m_3 & m_3 \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \\ \ddot{u}_o \\ L\ddot{\phi} \end{Bmatrix} + \begin{bmatrix} 2c_{b1} & 0 & -c_{b1} & -c_{b1} & -c_{b1} \\ 0 & 2c_{b2} & -c_{b2} & -c_{b2} & -c_{b2} \\ -c_{b1} & -c_{b2} & c_{b1}+c_{b2}+c_p & c_{b1}+c_{b2} & c_{b1}+c_{b2} \\ -c_{b1} & -c_{b2} & c_{b1}+c_{b2} & c_{b1}+c_{b2}+c_h & c_{b1}+c_{b2} \\ -c_{b1} & -c_{b2} & c_{b1}+c_{b2} & c_{b1}+c_{b2} & c_{b1}+c_{b2}+c_r/L^2 \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \dot{u}_3 \\ \dot{u}_o \\ L\dot{\phi} \end{Bmatrix}$$

$$+ \begin{bmatrix} 2k_{b1} & 0 & -k_{b1} & -k_{b1} & -k_{b1} \\ 0 & 2k_{b2} & -k_{b2} & -k_{b2} & -k_{b2} \\ -k_{b1} & -k_{b2} & k_{b1}+k_{b2}+k_p & k_{b1}+k_{b2} & k_{b1}+k_{b2} \\ -k_{b1} & -k_{b2} & k_{b1}+k_{b2} & k_{b1}+k_{b2}+k_h & k_{b1}+k_{b2} \\ -k_{b1} & -k_{b2} & k_{b1}+k_{b2} & k_{b1}+k_{b2} & k_{b1}+k_{b2}+k_r/L^2 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \\ u_o \\ L\phi \end{Bmatrix}$$

$$= \begin{bmatrix} -m_1 & 0 & 0 \\ 0 & -m_2 & 0 \\ 0 & 0 & -m_3 \\ 0 & 0 & -m_3 \\ 0 & 0 & -m_3 \end{bmatrix} \begin{Bmatrix} \ddot{u}_{g1} \\ \ddot{u}_{g2} \\ \ddot{u}_{g3} \end{Bmatrix} + \begin{bmatrix} -c_{b1} & 0 & c_{b1} \\ 0 & -c_{b2} & c_{b2} \\ c_{b1} & c_{b2} & -c_{b1}-c_{b2} \\ c_{b1} & c_{b2} & -c_{b1}-c_{b2} \\ c_{b1} & c_{b2} & -c_{b1}-c_{b2} \end{bmatrix} \begin{Bmatrix} \dot{u}_{g1} \\ \dot{u}_{g2} \\ \dot{u}_{g3} \end{Bmatrix} + \begin{bmatrix} -k_{b1} & 0 & k_{b1} \\ 0 & -k_{b2} & k_{b2} \\ k_{b1} & k_{b2} & -k_{b1}-k_{b2} \\ k_{b1} & k_{b2} & -k_{b1}-k_{b2} \\ k_{b1} & k_{b2} & -k_{b1}-k_{b2} \end{bmatrix} \begin{Bmatrix} u_{g1} \\ u_{g2} \\ u_{g3} \end{Bmatrix} \tag{10}$$

After defining the frequencies and damping of the structure and sub-soil system, Equation (10) can be expressed in the frequency domain as

$$[Z(i\omega)]\{u(i\omega)\} = [Z_g(i\omega)]\{u_g(i\omega)\} \tag{11}$$

where

$$\{u(i\omega)\} = \{u_1(i\omega) \quad u_2(i\omega) \quad u_3(i\omega) \quad u_0(i\omega) \quad L\phi(i\omega)\}^T \tag{12a}$$

$$\{u_g(i\omega)\} = \{u_{g1}(i\omega) \quad u_{g2}(i\omega) \quad u_{g3}(i\omega)\}^T \tag{12b}$$

are the response and the input ground motion, respectively. $[Z(i\omega)]$ and $[Z_g(i\omega)]$ are the impedance matrices of the system, which are in the following form

$$[Z(i\omega)] = \begin{bmatrix} z_{11}(i\omega) & \cdots & z_{15}(i\omega) \\ \vdots & & \vdots \\ z_{51}(i\omega) & \cdots & z_{55}(i\omega) \end{bmatrix} \quad [Z_g(i\omega)] = \begin{bmatrix} z_{g11}(i\omega) & \cdots & z_{g13}(i\omega) \\ \vdots & & \vdots \\ z_{g51}(i\omega) & \cdots & z_{g53}(i\omega) \end{bmatrix} \tag{13}$$

The dynamic response of the bridge structure can then be obtained from

$$\{u(i\omega)\} = [Z(i\omega)]^{-1}[Z_g(i\omega)]\{u_g(i\omega)\} = [H(i\omega)]\{u_g(i\omega)\} \tag{14}$$

For the bridge model shown in Figure 1, the minimum separation distance a MEJ must provide to preclude pounding equals the relative displacement of the bridge system, which can be expressed in the frequency domain as

$$\Delta_1(i\omega) = u_1(i\omega)\,, \quad \Delta_2(i\omega) = u_2(i\omega)\,, \quad \Delta_3(i\omega) = u_1^t(i\omega) - u_2^t(i\omega) \tag{15}$$

The power spectral density functions of Δ_1, Δ_2 and Δ_3 thus can be derived. The mean peak responses can then be estimated based on the standard random vibration method [9].

4. Numerical Example

In this study, three types of soils, i.e. firm, medium and soft soil, are considered, Table I gives the corresponding parameters of soil and base rock. The ground motion is assumed to be intermediately correlated with an apparent wave velocity $v_{app} = 1000$ m/s. The stiffness of the left span is assumed to be constant with $k_{b1} = 2.4\times10^7$ N/m, which corresponds to the uncoupled frequency $f_1 = \sqrt{2k_{b1}/m_1}/2\pi = 1.0$ Hz. The bearing stiffness of the right span varies from 2×10^5 N/m to 1.5×10^8 N/m in the present study to obtain different frequency ratios f_2/f_1.

Table I. Parameters for local site conditions

Type	Density (kg/m^3)	Shear wave velocity (m/s)	Damping ratio	Poisson's ratio
Base rock	3000	1500	0.05	0.4
Firm soil	2000	400	0.05	0.4
Medium soil	2000	200	0.05	0.4
Soft soil	1500	150	0.05	0.4

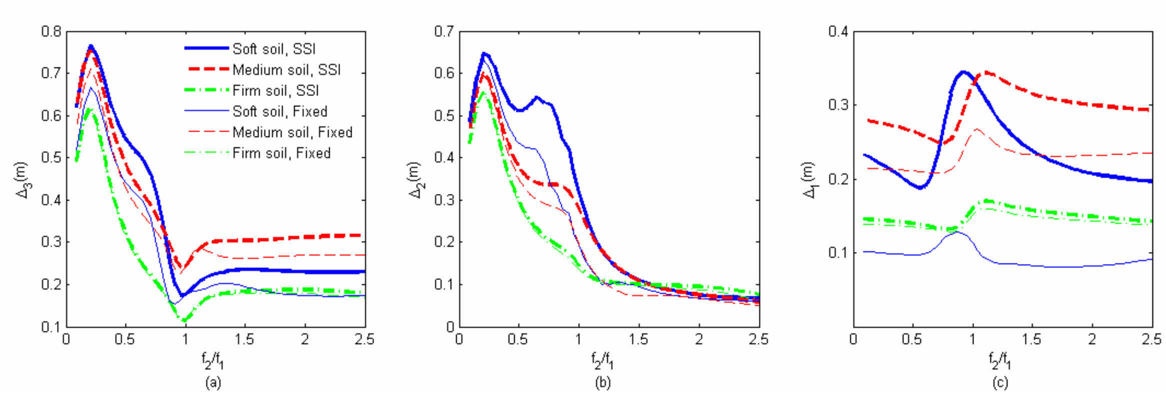

Figure 2. Influence of site effect and SSI on the required separation distances (a) Δ_3, (b) Δ_2 and (c) Δ_1

Figure 2 shows the estimated Δ_1, Δ_2 and Δ_3 with and without consideration of SSI. As can be seen from Figure 2, SSI only slightly changes the frequency content of the system, which results in the peak responses occur at the same frequency ratio with or without considering SSI in the analysis. This is because the power spectral densities of the required separation distances depend on the product of surface ground motion power spectral density ($S(i\omega)$) and the frequency response function of the system ($H(i\omega)$). Local site amplifies certain frequencies significantly at various vibration modes of the site, which results in the energy of the surface ground motion concentrates at a few frequencies. Large structural responses occur when the bridge vibration frequency coincides with those site vibration frequencies. Therefore larger separation distance is predicted at almost the same frequency ratio with or without considering SSI in the analysis. To observe the contribution of SSI more clearly, the required separation distances with consideration of SSI are subtracted by those without SSI effect, and the results are presented in Figure 3. It is evident that the influence of SSI is significant, especially, for soft and medium soil. The required separation distances will be significantly underestimated when SSI effect is ignored. As shown in Figure 3(b), when soft or medium soil is considered, the influence of SSI increases with larger frequency ratio. SSI effect becomes the most pronounced when the structure resonates with local site, e.g., the contribution of SSI is nearly 0.2 m for soft soil when f_2 / f_1 is around 0.75. This is because the right span resonates with local site at this frequency ratio. When $f_2 / f_1 > 0.75$, the influence of SSI on the total responses decreases and becomes almost constant when the right span is stiff enough, because quasi-static response dominates the total response. It is generally true that SSI effect is more obvious in the case of soft soil than medium site. When firm soil is considered the influence of SSI can be neglected. For Δ_3, similar observations can be obtained. It should be noted that no obvious peak response is predicted when resonance happens for medium soil, because the two spans tend to vibrate in phase. For Δ_1, it is observed again that SSI effect in the case of soft site is more prominent than that of firm site.

5. Conclusions

This paper studies the combined effect of ground motion spatial variation, site condition and SSI on the required separation distances, and the effect of SSI is highlighted. Numerical results reveal:

1. The influence of SSI on the required separation distances is significant. Larger separation distances are usually required when SSI is considered.
2. SSI effect cannot be neglected when the structures are founded on soft soil site. The contribution of SSI is relatively small when firm site is considered.
3. For a specific site condition, SSI effect is evident when the structure resonates with the site.

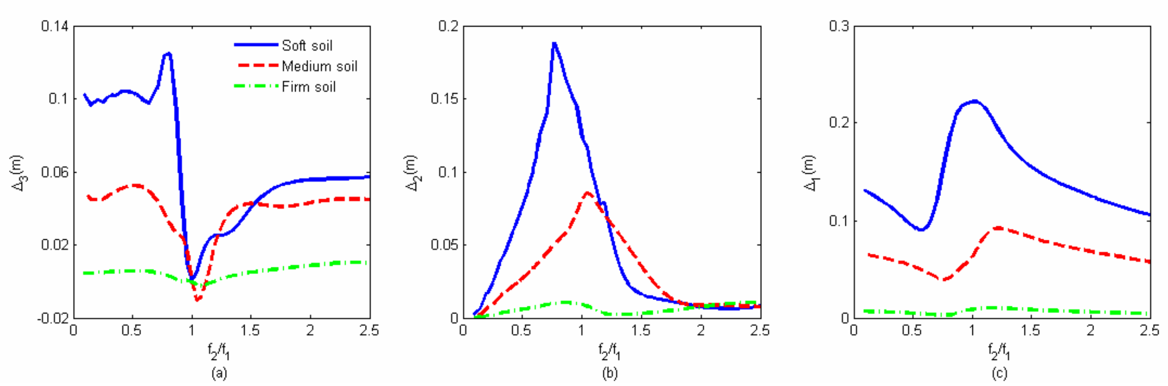

Figure 3. Contribution of SSI to the required separation distances for different soil conditions (a) Δ_3, (b) Δ_2 and (c) Δ_1

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