STOCHASTIC ANALYSIS OF THE REQUIRED SEPARATION DISTANCE TO AVOID SEISMIC POUNDING BETWEEN ADJACENT BRIDGE DECKS

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ABSTRACT :

Pounding between adjacent bridge decks during severe earthquake may result in significant structural damage or even collapse of the decks. One of the most effective methods to avoid such damage is to provide adequate separation distance between bridge decks. This paper investigates the required separation distance between two adjacent continuous bridge decks which are connected by a modular expansion joint (MEJ). Each bridge deck is modeled as a lumped mass supported on an isolating bearing. Both the bearings and the elastic piers are modeled as spring-damping elements. Spatial ground motions are modeled by the filtered Tajimi-Kanai power spectral density function and an empirical coherency loss function. Site amplification effect is included by a transfer function derived from one dimensional wave propagation theory. Stochastic response equations of the adjacent bridge decks are formulated. Parametric studies are carried out to study the required separation distance between the two adjacent decks to avoid pounding. The effect of non-uniform ground excitation, site amplification, and different parameters of bearing on the required separation distance are investigated. Comparisons and discussions are made based on the numerical results.

KEYWORDS: required separation distance, spatial variation, site effect, bearing stiffness

1. INTRODUCTION

Investigations into some severe earthquake damages revealed that pounding between adjacent bridge structures is one of the main reasons resulted in severe damage or even collapse of bridge decks, e.g., poundings were observed in the 1989 Loma Prieta earthquake and the 1994 Northridge earthquake (Yashinsky and Karshenas 2003), the 1995 Hyogo-Ken Nanbu earthquake (Kawashima and Unjoh 2006), the 1999 Chi-Chi Taiwan earthquake (Earthquake Engineering Research Institute 1999) and the 2006 Yogyakarta earthquake (Elnashai et al. 2007).

For bridge structures with conventional expansion joints, completely avoid pounding between bridge girders during strong earthquake excitations is often not possible. This is because the separation gap in a conventional expansion joint is usually only a few centimeters due to serviceability consideration for smooth traffic flow. Many researchers have been studying the effect of pounding on bridge structures. Owing to the difficulty in modeling ground motion spatial variations, many studies neglected ground motion spatial variation (Ruangrassamee and Kawashima 2001, DesRoches and Muthukumars 2002), or just assumed the variation was caused by wave passage effect only (Jankowaki et al. 1998, Zhu et al. 2002). Only a few studies considered ground motion spatial variations (Hao 1998, Hao and Chouw 2008, Zanzrdo et al. 2002). In these studies, the effect of site amplification was not considered, which may lead to an inaccurate prediction of responses of a bridge located at a non-uniform soil site, because soil layers amplify the base rock motion and hence have a great influence on the structure responses (Hao and Chouw 2007, Dumanoglu and Soyluk 2003). With the new development of the Modular Expansion Joint (MEJ), which allows large relative movement in the joint, completely precluding pounding between adjacent bridge spans becomes possible (Chouw and Hao 2008). However, researches on required separation distances to avoid pounding between adjacent bridge decks were relatively less. Hao (Hao 1998) carried out a parametric study of the required seating length for bridge decks during earthquake. Chouw and Hao (2008) investigated the influence of spatial variation of ground motions and soil-structure interaction on the minimum total gap that a MEJ between two bridge frames must have to prevent pounding. It should be noted that both the studies (Hao 1998, Chouw and Hao 2008) ignored the site effect.

This paper investigates the required separation distance of two adjacent bridge decks connected with a MEJ. Each bridge deck is modeled as a rigid beam with lumped mass supported on isolation bearings, the bearings are modeled as spring-damping elements as shown in Figure 1, and provide the desired isolation effects to the system by the horizontal flexibility and damping characteristics. The piers considered are elastic and also modeled as a spring-damping element. Spectral analyses are carried out. Spatial ground motions are modeled by a filtered Tajimi-Kanai power spectral density function and an empirical coherency loss function. Site amplification effect is simulated by a transfer function derived from the one dimensional wave propagation theory. Parametric studies are carried out to study the required separation distance of the two adjacent decks to avoid pounding. The effect of spatial variation of ground motions, local site conditions (soil depth and soil properties) and the vibration characteristics of the two adjacent bridge decks on the required separation distances are investigated. Comparisons and discussions are made based on the numerical results. The effects of ground motion spatial variation and site amplification on the required separation distance between bridge decks are highlighted. The soil-structure interaction effect is not considered in the present paper.

2. BRIDGE MODEL

Figure 1 shows the schematic view and mathematic model of two adjacent continuous bridge decks located on a soil site. Each bridge deck is modeled as a rigid beam with a lumped mass supported by an isolation bearing. To simplify the analysis, the cross sections of the bridge decks are assumed to be the same, with mass per unit length $1.2\times10^4 kg/m$ and the length $d_1 = d_2 = 100m$, so the mass of the two bridge decks is $m_{A1} = m_{B1} = 1.2\times10^6 kg$. The distance between the two supports is assumed to be $d = 100m$. The two isolation bearings are modeled with two spring-damping elements, with damping ratio $\xi_{bA} = \xi_{bB} = 0.14$ (Jankowaki et al . 1998). The two piers are elastic, with mass $m_{A2} = m_{B2} = 1.2 \times 10^5 kg$. The effective isolation of the bridge decks requires flexible bearings and very stiff piers (Zhu et al. 2002), the stiffness and damping ratio of the pier is given as $k_p = 10^{10} kN/m$ and $\xi_p = 0.05$. Different bearing stiffness k_{bB} is assumed in the analysis in the paper. Based on the assumptions above, both the left and right span of the bridge can be simplified as a 3 degrees of freedom system which is also shown in Figure 1, where v_{gA} and v_{gB} are the spatially varying ground displacements at different supports.

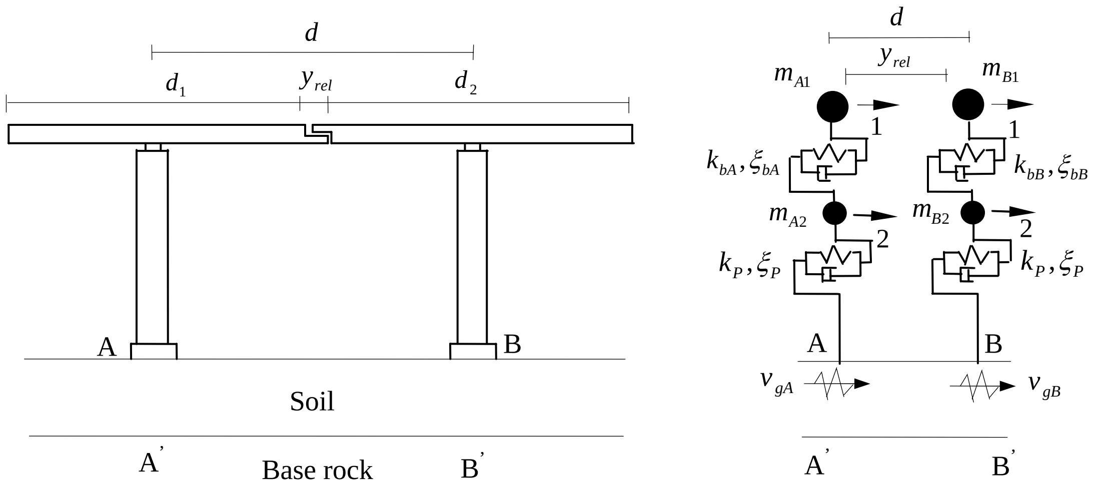

Figure 1 Schematic view and mathematical model of two adjacent bridge decks

3. GROUND MOTION SPATIAL VARIATIONS

Assume ground motion intensities at *A'* and *B'* on the base rock are the same but vary spatially, its power spectral density is modeled by a filtered Tajimi-Kanai power spectral density function as

$$S_g(\omega) = |H_P(\omega)|^2 S_0(\omega) = \frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + (2\omega_f \omega \xi_f)^2} \frac{1 + 4\xi_g \omega^2 / \omega_g^2}{(1 - \omega^2/\omega_g^2)^2 + 4\xi_g \omega^2/\omega_g^2} \Gamma \tag{3.1}$$

with the parameters $f_f = \omega_f / 2\pi = 0.25Hz$, $\xi_f = 0.6$, $f_g = \omega_g / 2\pi = 5.0Hz$, ξ_g =0.6 and $\Gamma = 0.022 m^2/s^3$. These parameters correspond to a ground acceleration of duration $T = 20s$ and peak value (PGA) $0.5g$ (Hao and Zhang 1999).

Ground motion spatial variation at points $A^{'}$ and $B^{'}$ is modeled with a coherency loss function (Hao 1989)

$$\gamma_{A^{'}B^{'}}(i\omega) = |\gamma_{A^{'}B^{'}}(i\omega)| e^{i\omega d_{A^{'}B^{'}}/v_{app}} = e^{-\beta d_{A^{'}B^{'}}} e^{-\alpha(\omega)\sqrt{d_{A^{'}B^{'}}}(\omega/2\pi)^2} e^{i\omega d_{A^{'}B^{'}}/v_{app}} \tag{3.2}$$

in which $\alpha(\omega)$ is a parameter which is related to constants a, b, c and β. $d_{A^{'}B^{'}}$ is the distance between points $A^{'}$ and $B^{'}$, v_{app} is the apparent wave propagation velocity, and $v_{app} = 1000 m/s$ is used in the paper.

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 2007)

$$H_j(i\omega)=\frac{(1+r_j-i\xi_j)e^{-i\omega\tau_j(1-2i\xi_j)}}{1+(r_j-i\xi_j)e^{-2i\omega\tau_j(1-2i\xi_j)}} \tag{3.3}$$

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

$$\begin{gathered} S_j(\omega)=\left|H_j(i\omega)\right|^2 S_g(\omega) \\ S_{AB}(i\omega)=H_A(i\omega)H_B^*(i\omega)S_g(i\omega)\gamma_{A'B'} \end{gathered} \tag{3.4}$$

in which the superscript '*' represents complex conjugate.

4. STRUCTURAL RESPONSES

As mentioned above, soil-structure interaction is not considered in the present paper, the dynamic equilibrium equation of the system shown in Figure 1 can be decoupled into its modal vibration equation as

$$\ddot{q}_i+2\xi_i\omega_i\dot{q}_i+\omega_i^2 q_i=-\frac{\varphi_i^T[K_{sb}]}{\varphi_i^T[M_{ss}]\varphi_i}[v_g] \tag{4.1}$$

where φ_i is the *i*th vibration mode shape of the structure, q_i is the *i*th modal response, ω_i and ξ_i are the corresponding circular frequency and viscous damping ratio, respectively. $[K_{sb}]$ is the coupling stiffness matrix between the structure degrees of freedom and the support degrees of freedom, $[v_g]$ is the ground displacements at the bridge supports.

The structure response of the *k*th degree of freedom is

$$y^k(t)=\sum_{i=1}^{m}\varphi_i^k q_i \tag{4.2}$$

where *m* is the number of modes considered in the calculation, and φ_i^k is the mode shape value corresponding to the *k*th degree of freedom. Using super- or subscripts '*A*' and '*B*' to denote left span and right span of the bridge, the relative displacement between the two bridge decks in the frequency domain is then

$$\bar{y}_{rel}(\omega)=\sum_{i=1}^{2}\varphi_{Ai}^1\bar{q}_{Ai}(i\omega)-\sum_{j=1}^{2}\varphi_{Bj}^1\bar{q}_{Bj}(i\omega) \tag{4.3}$$

The *i*th modal response of the left span can be obtained from Eqn. 4.1 as

$$\bar{q}_{Ai}(i\omega)=H_{Ai}(i\omega)\sum_{r=1}^{n_A}\psi_{ri}^A\bar{v}_{gr}(i\omega) \tag{4.4}$$

in which n_A is the total number of supports of system *A*, and

$$\psi_{ri}^{A} = -\frac{\varphi_{Ai}^{T}[K_{sb}^{Ar}]}{\varphi_{Ai}^{T}[M_{ss}]\varphi_{Ai}} \tag{4.5}$$

is the quasi-static participation coefficient for the *i*th mode corresponding to a movement at support *r*, $[K_{sb}^{Ar}]$ is a vector in coupled stiffness matrix $[K_{sb}^{A}]$ corresponding to support *r* of system *A*, and $H_{Ai}(i\omega) = 1/(\omega_{Ai}^2 - \omega^2 + 2i\xi_{Ai}\omega\omega_{Ai})$ is the *i*th mode transfer function. Substituting Eqn.4.5 and 4.6 into Eqn.4.3, the relative displacement in the frequency domain can be derived as

$$\begin{aligned} \bar{y}_{rel}(i\omega) = & [\varphi_{A1}^{1}H_{A1}(i\omega)\psi_{1}^{A} + \varphi_{A2}^{1}H_{A2}(i\omega)\psi_{2}^{A}]\bar{v}_{gA}(\omega) \\ & -[\varphi_{B1}^{1}H_{B1}(i\omega)\psi_{1}^{B} + \varphi_{B2}^{1}H_{B2}(i\omega)\psi_{2}^{B}]\bar{v}_{gB}(\omega) \end{aligned} \tag{4.6}$$

The power spectral density function of relative displacement is then

$$S_{y_{rel}}(\omega) = S_{y_A^1}(\omega) + S_{y_B^1}(\omega) - 2\mathrm{Re}[S_{y_A^1 y_B^1}(i\omega)] \tag{4.7}$$

where 'Re' indicates the real part of a complex number, and

$$\begin{aligned} S_{y_A^1}(\omega) &= \frac{1}{\omega^4}\left|\varphi_{A1}^{1}H_{A1}(i\omega)\psi_{1}^{A} + \varphi_{A2}^{1}H_{A2}(i\omega)\psi_{2}^{A}\right|^2 S_{\ddot{v}_{gA}}(\omega) \\ S_{y_B^1}(\omega) &= \frac{1}{\omega^4}\left|\varphi_{B1}^{1}H_{B1}(i\omega)\psi_{1}^{A} + \varphi_{B2}^{1}H_{B2}(i\omega)\psi_{2}^{B}\right|^2 S_{\ddot{v}_{gB}}(\omega) \\ S_{y_A^1 y_B^1}(i\omega) &= \frac{1}{\omega^4}[\varphi_{A1}^{1}H_{A1}(i\omega)\psi_{1}^{A} + \varphi_{A2}^{1}H_{A2}(i\omega)\psi_{2}^{A}][\varphi_{B1}^{1}H_{B1}(i\omega)\psi_{1}^{A} + \varphi_{B2}^{1}H_{B2}(i\omega)]^{*} S_{\ddot{v}_{gA}\ddot{v}_{gB}}(i\omega) \end{aligned} \tag{4.8}$$

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

5. NUMERICAL RESULTS AND DISCUSSIONS

In the study, the bridge model shown in Figure 1 is considered, the isolation bearing stiffness of the left span is assumed to be $k_{bA} = 10^7\, kN/m$, which gives the first modal frequency of the left span as 0.46*Hz*. The bearing stiffness of the right span varies from 10^5 to 5×10^9 kN/m, representing very soft to very stiff bearings.

5.1 Effect of Spatial Variation

The effects of spatially varying ground motions are studied first. Assume the bridge locates on the base rock (site amplification is neglected here), five spatially varying ground motions are considered. They are highly, intermediately and weakly correlated ground motions, spatially varying ground motions without considering the coherency loss ($|\gamma_{A'B'}| = 1.0$, wave passage effect only) and uniform ground motion ($\gamma_{A'B'} = 1.0$). Table 5.1 gives the corresponding parameters. Figure 2 shows the required separation distances with respect to the variation of bearing stiffness of the right span.

As shown in Figure 2, with an assumption of uniform excitation, the required separation distance is relatively small when the stiffness of the adjacent structures are similar, and is zero when $k_{bA} = k_{bB}$. This is because the responses of the two adjacent spans are exactly the same when the frequencies coincide with each other (Hao 1998, Hao and Zhang 1999). Therefore there is no relative displacement between them. These results correspond well with the recommendations of the current design regulations to adjusting the vibration frequencies of the adjacent bridge spans to close to each other in order to preclude pounding. As also can be seen from Figure 2, when the right span is flexible, the influence of spatially varying ground motions is relatively small. However, when the right span is relatively stiff, the ground motion spatial correlation has a significant effect on the required separation distance. As shown, the less correlated ground motion results in larger relative displacement. This is because dynamic response dominants the total response when the structure is flexible and quasi-static response dominates the total response when the structure is relatively stiff. The effect of coherency loss between spatial ground motions is more significant to thequasi-static response. Hence the ground motion coherency loss effect becomes prominent when the right span is stiff. Quasi-static response is independent of the structural vibration frequency. Therefore the required separation distance is almost a constant with changing stiffness of the right span as shown in Figure 2. The largest separation distance to avoid pounding is required when $k_{bB} = 1.06\times10^{6} kN/m$ as can be seen in Figure 2. This is because the first modal frequency of the right span is 0.15Hz at this stiffness, which is the central frequency of base rock ground displacement as shown in Figure 3, which means the largest separation distance is required when one of the span resonates with the central frequency of ground displacement.

Table 5.1. Parameters for coherency loss functions

Coherency loss	β	a	b	c
Highly	1.109×10^{-4}	3.583×10^{-3}	-1.811×10^{-5}	1.177×10^{-4}
Intermediately	3.697×10^{-4}	1.194×10^{-2}	-1.811×10^{-5}	1.177×10^{-4}
Weakly	1.109×10^{-3}	3.583×10^{-2}	-1.811×10^{-5}	1.177×10^{-4}

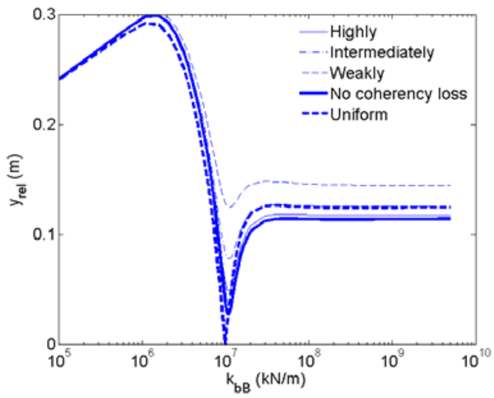

Figure 2 Effect of spatially varying ground motions on the required separation distance

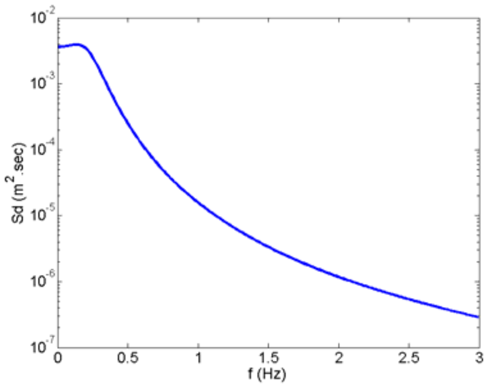

Figure 3 Base rock ground displacement power spectral density

5.2 Effect of Soil Depth

Soil layer amplifies the base rock motion and has a great influence on the structure response. Three different soils are considered in the paper, i.e. firm, medium and soft soil. Table 5.2 gives the corresponding parameters of site conditions considered in the study. In this part, assume ground motion is intermediately correlated and soil under support A and B are the same, and both are medium soil. Three different soil depths are considered, i.e. h=0, 30, 50 m. Figure 4 shows the required separation distance.

Table5. 2. Parameters of different soils

Type	Density (kg/m^3)	Shear wave velocity (m/s)	Damping ratio
Base rock	3000	1500	0.05
Firm soil	2000	450	0.05
Medium soil	1500	300	0.05
Soft soil	1500	100	0.05

As shown in Figure 4, different soil depths result in only slightly different required separation distance to avoid pounding. This is because the required separation distance only depends on the relative response of the adjacent structures, instead of the absolute response of the structure. Increase the soil depth makes the site softer, hence larger structure responses, which slightly increases the required separation distance between two structures, especially when the structure is stiff. As can be seen in Figure 4, the first peak occurs at $k_{bB} = 1.06 \times 10^6 kN/m$, due to the right span resonates with the central frequency of the base rock ground displacement as mentioned above. When h=50*m*, as shown another peak occurs at $k_{bB} = 10^8 kN/m$. This corresponds to the first modal frequency of 1.4*Hz*. At this frequency the right span resonates with the ground motion because the predominant ground motion frequency is 1.4*Hz*, as shown in Figure 5.

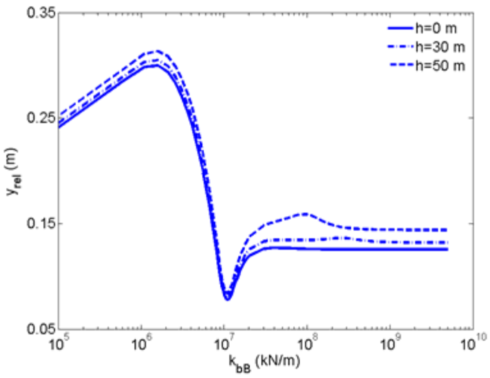

Figure 4 Effect of soil depths on separation distance

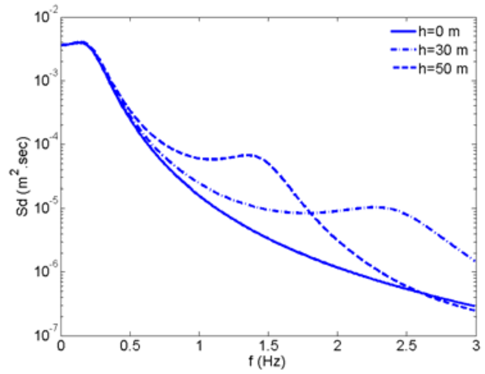

Figure 5 Ground displacement power spectral densities

5.3 Effect of Soil Property

Assume ground motion is intermediately correlated, soil depth is 30*m*. Firm, medium and soft soils are considered to study the effect of soil properties. As shown in Figure 6, the effect of soil properties is significant, soft soil results in the largest required separation distance. Peaks can be obtained when the right span resonates with the ground displacement. Take soft soil site as example, the first peak occurs at the central frequency of the base rock ground displacement. The second peak occurs at $k_{bB} = 2 \times 10^7 kN/m$, at which the first modal frequency of the right bridge structure is 0.8*Hz*. This causes resonance of the right structure with ground motion as the predominant frequency of the ground motion on soft soil site is also 0.8*Hz* as shown in Figure 7.

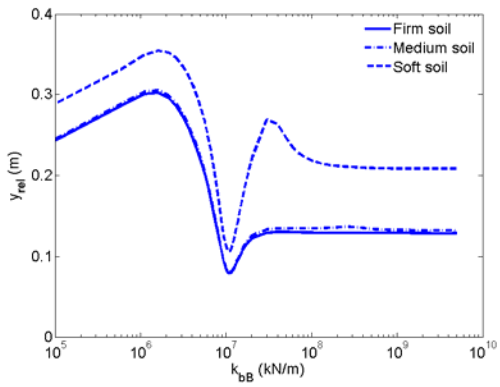

Figure 6 Effect of soil properties on required distance

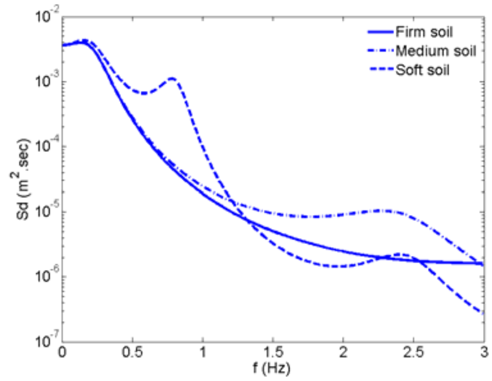

Figure 7 Ground displacement power spectral densities

6. CONCLUSIONS

With the development of modular expansion joint (MEJ), it is possible to completely preclude pounding between adjacent bridge structures without influence the serviceability of the traffic. This paper investigated the required separation distance to avoid pounding of two adjacent bridge decks located on sites with different soil properties. It was found that the largest separation distance is required when one of the spans resonates with the central frequencies of the ground displacement. Spatial variation of ground motion and soil properties has significant influence on the required separation distance. Weakly correlated ground motions result in larger required separation distance. The deeper and softer is the soil site, the larger is the required separation distance. Neglecting either the ground motion spatial variation or soil amplification effect may underestimate the required separation distance to avoid pounding.

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