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Estimation of required seating length of bridge girders under non-uniform ground excitation and different ground conditions

Nawawi Chouw* and Hong Hao**

*Okayama University, Graduate School of Environmental Science, Okayama 700-8530, Japan
** The University of Western Australia, School of Civil and Resource Engineering, Crawley WA6009, Australia

This study addresses the simultaneous influence of soil-structure interaction, non-uniform ground excitations and pounding on the required seating length of bridge girders. The recently published Japanese design specification for bridge girder relative displacement and the recommendation of many current design codes for mitigating pounding and unseating potential are also discussed. The spatially varying ground excitation is simulated stochastically using an empirical spectrum and coherency loss function. The ground is a half-space or a soft layer of different thickness on hard soil. The results show that non-uniform ground motions together with soft subsoil can strongly amplify required seating length and pounding forces, and consequently increase unseating and pounding damage potential of bridge girders.

Key Words: relative displacement, spatially varying excitation, soil-structure system, design specifications, seating length

1. Introduction

Bridges often consist of several segments. The dynamic response of each bridge segment depends on the dynamic property of the bridge structure and the supporting subsoil. Even if all bridge segments are the same, the subsoil is normally not the same. Besides, the propagating seismic waves will arrive at the bridge pier foundations at different instants, spatially non-uniform ground excitation is therefore likely. Consequently, each bridge segment will respond differently, and relative displacements between neighbouring structures are unavoidable. If adjacent bridge structures move towards each other and the relative displacement exceeds the existing gap, pounding occurs. If bridge structures move away from each other, unseating can take place. Girder damages due to poundings and collapses due to girder unseating have been observed in many major earthquakes, like the 1994 Northridge earthquake[1], the 1995 Kobe earthquake[2], 1999 Kocaeli earthquake[1], and the 1999 Chi-Chi earthquake[3].

Most of the investigations in the past, however, mainly focused on pounding responses of adjacent structures, measures for reducing pounding effect or the required gap to avoid pounding. Hao et al.[4) 5) 6) 7)] and Chen[8], for example, investigated the required structural distance depending on properties of the adjacent structures and the spatial ground excitation. Oshima et al.[9], Jankowski[10], Ruangrassamee et al.[11] and Zhu et al.[12] studied measures for reducing the effect of pounding at bridge girders. In most of the investigations uniform ground excitation is assumed. So far only a small number of researches on non-uniform ground motion effect was performed, e.g. by Hao[13] and Zanardo[14]. They confirmed that spatially varying ground motion could have strong influence on the response of adjacent structures. Investigations on relative displacement response of adjacent structures including soil-structure interaction (SSI) effect are also rare. If SSI is considered, e.g. Kim et al.[15] and Zhu et al.[16], often only frequency-independent soil stiffness is applied because of the difficulty in the numerical modeling of non-linear soil-structure interaction problems. Chouw and Hao[17, 18] applied frequency-dependent soil stiffness of a half-space in their investigations of pounding effect on bridge girders.

Since relative displacements are significant for determining the required distance to avoid pounding, and also for defining the necessary seating length of bridge girders to avoid unseating-induced collapses, many design specifications, e.g. Caltrans Seismic Design Cretaria[19], recommend an adjustment of the structural fundamental frequencies so that the adjacent structures will vibrate in phase. This study focuses on the simultaneous effect of soil-structure interaction, spatial variation of ground excitations and pounding on the necessary seating length of adjacent bridge girders. Current Japanese design specification[20] for highway bridge is also evaluated.

2. Bridge structures with different soil conditions and non-uniform excitations

Fig. 1 shows the considered bridge structures. For simplicity the displacement of each girder is described by a single-degree-of-freedom (sdof). u_1 and u_2 are the displacements of the left and right girders, respectively. It is assumed that the distance between the bridge piers is 100 m, and each surface foundation is rigid and has a length of 9 m. The pier height is 9 m, and each bridge structure has a damping ratio of 5 %. The considered gap size is 5.0 cm.

The influence of following subsoil is investigated: rigid soil, soft half-space with a shear wave velocity c_s of 100 m/s, a soft soil layer with a thickness H of 2.5 m or 5.0 m and a shear wave velocity c_{s1} of 100 m/s on hard soil (half-space) with a shear wave velocity c_{s2} of 400 m/s. The soil has a density ρ of 2000 kg/m^3 and the Poisson's ratio ν of 0.33. To limit the influence factors it is assumed that only radiation damping due to wave propagation in soil is considered. The left and right bridge structures experience the non-uniform ground excitations u_{g1}(t) and u_{g2}(t). In order to have an insight into the consequence of commonly performed assumption, the influence of uniform ground excitation is also considered. Both bridge structures experience then the same ground excitation u_{g1}(t).

2.1 Interaction between bridge structures and between subsoil and bridge structures

In the numerical analyses the two bridge structures with their foundations are described in the Laplace domain using a finite element method, and the subsoil by a boundary element method. A coupling of these two subsystems leads to the dynamic stiffness of the whole soil-structure system indicated by the stiffness matrix in left part of Eq. 1. The response $\tilde{\boldsymbol{u}}^b$ in the Laplace domain of the system to a given excitation $\tilde{\boldsymbol{P}}^b$ (right part of Eq. 1 obtained using Eq. 2) can then be calculated. The tilde indicates a vector or matrix in the Laplace domain. A transformation of the results from the Laplace to time domain leads to the time history of the result (Eq. 3).

The dynamic stiffness of the bridge structure members is obtained by solving the equation of motion analytically. Continuous-mass model formulation is applied. The dynamic stiffness $\tilde{\boldsymbol{K}}^b$ of the bridge structures is obtained by adding the stiffness of each member using the direct stiffness method. Details of the formulation are given in the reference[21].

The dynamic stiffness $\tilde{\boldsymbol{K}}^s$ of the subsoil can be obtained by transforming the wave equation into the Laplace domain. By using the full-space fundamental solution and by assuming the distribution of displacement and traction along the boundaries the relationship between traction and displacement at the

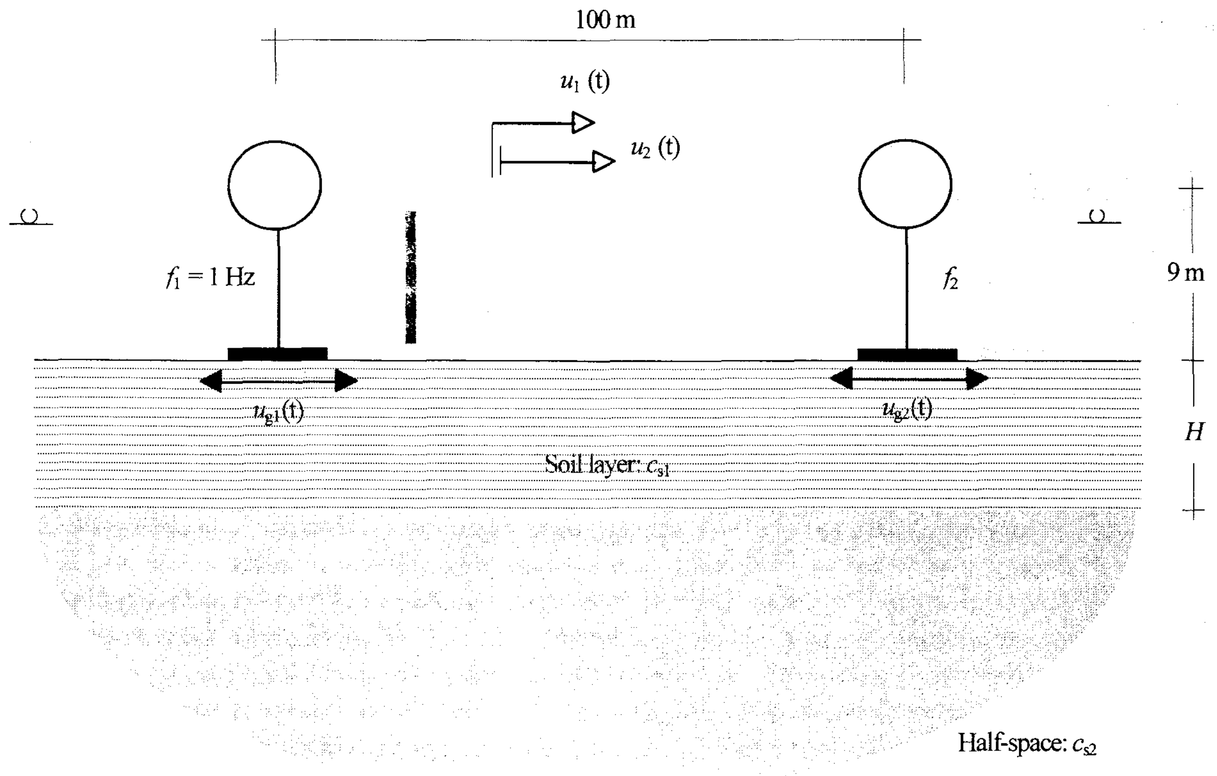

Fig. 1. Simplified model of two adjacent bridge structures with subsoil

contact area between the foundation and subsoil can be defined. In the case of soft soil layer over hard soil the traction-displacement relationship of these two domains, soil layer and half-space, can be condensed to the traction-displacement relation at the ground surface by equating the displacements and equilibrating the interacting tractions at the interface between the two domains. An introduction of the element area leads then to the dynamic soil stiffness $\widetilde{\boldsymbol{K}}^{\mathrm{s}}$ at the contact area between foundations and subsoil. The governing equation for each bridge structure with subsoil is obtained by coupling the two subsystems.

$$\begin{bmatrix} \widetilde{\boldsymbol{K}}_{\mathrm{bb}}^{\mathrm{bn}} & \widetilde{\boldsymbol{K}}_{\mathrm{bc}}^{\mathrm{bn}} \\ \widetilde{\boldsymbol{K}}_{\mathrm{cb}}^{\mathrm{bn}} & \widetilde{\boldsymbol{K}}_{\mathrm{cc}}^{\mathrm{bn}} + \widetilde{\boldsymbol{K}}_{\mathrm{cc}}^{\mathrm{sn}} \end{bmatrix} \begin{Bmatrix} \widetilde{\boldsymbol{u}}_{\mathrm{b}}^{\mathrm{bn}} \\ \widetilde{\boldsymbol{u}}_{\mathrm{c}}^{\mathrm{bn}} \end{Bmatrix} = \begin{Bmatrix} \widetilde{\boldsymbol{P}}_{\mathrm{b}}^{\mathrm{bn}} \\ \widetilde{\boldsymbol{P}}_{\mathrm{c}}^{\mathrm{bn}} \end{Bmatrix} \tag{1}$$

The superscripts b and s stand for the bridge and subsoil, respectively. The superscript n indicates the left or right bridge structures. The subscripts b and c stand for bridge and contact-degree-of-freedom at the soil-foundation interface, respectively. After transforming the excitations into the Laplace domain

$$\left\{\widetilde{\boldsymbol{P}}^{\mathrm{b}}(\mathrm{s})\right\} = \int_0^{\infty} \left\{\boldsymbol{P}^{\mathrm{b}}(\mathrm{t})\right\} \mathrm{e}^{-\mathrm{s}\,\mathrm{t}} \mathrm{dt} \tag{2}$$

where $\mathrm{s} = \delta + \mathrm{i}\omega$ is the Laplace parameter and $i = \sqrt{-1}$, the linear response $\widetilde{\boldsymbol{u}}^{\mathrm{b}}$ of both bridge structures can then be defined using Eq. (1). A transformation of the results back to the time domain gives the time history of the structural responses $\boldsymbol{u}^{\mathrm{b}}$, and the girder relative displacement can be calculated.

$$\left\{\boldsymbol{u}^{\mathrm{b}}(\mathrm{t})\right\} = \frac{1}{2\pi \mathrm{i}} \int_{\delta - \mathrm{i}\omega}^{\delta + \mathrm{i}\omega} \left\{\widetilde{\boldsymbol{u}}^{\mathrm{b}}(\mathrm{s})\right\} \mathrm{e}^{\mathrm{s}\,\mathrm{t}} \mathrm{ds} \tag{3}$$

For incorporation of the pounding effect the unbalanced forces are defined using the relative displacement and the condensed stiffness of one of the bridge structures. For instance, the left bridge structure with foundation and subsoil is condensed into the pounding degree-of-freedom (pdof), the condensed stiffness is

$$\widetilde{\boldsymbol{K}}_{\mathrm{pp}}^{*} = \widetilde{\boldsymbol{K}}_{\mathrm{pp}} - (\widetilde{\boldsymbol{K}}_{\mathrm{pr}} (\widetilde{\boldsymbol{K}}_{\mathrm{rr}})^{-1} \widetilde{\boldsymbol{K}}_{\mathrm{rp}}) \tag{4}$$

The subscripts p and r stand for pdof and other dofs of the left soil-structure system. Since now the two structures are in contact the condensed stiffness has to be added to the stiffness of the uncondensed subsystem in Eq. 1. Using the unbalanced forces the corrective terms can be calculated, and the previous obtained linear responses are corrected in the time domain from the instant when pounding occurs. An examination of the results reveals the instant when the girders will separate. The unbalanced forces to incorporate the separation effect are equal to the contact force, which can be calculated using the condensed stiffness $\widetilde{\boldsymbol{K}}_{\mathrm{pp}}^{*}$ (Eq. 4) and the relative displacement at pdof. The corrective term is obtained from Eq. (1) of the uncoupled subsystem. Using the corrective term the results are corrected from the time of separation. The actual responses are examined again for further poundings. The calculation is complete if no more pounding occurs. Details of the non-linear soil-structure interaction approach are described in the reference[22].

2.2 Simulation of spatially varying ground motions

After processing about 2000 strong ground motion time histories recorded in the SMART-1 array, Hao[23], and Hao et al.[24] proposed an empirical coherency loss function. The coherency loss function between ground motions recorded at two locations i and j on ground surface is

$$\begin{aligned} \gamma_{\mathrm{ij}} &= \exp[-\beta_1 d_1 - \beta_2 d_{\mathrm{t}}] \\ &\quad \exp\{[-\alpha_1(f) d_1^{1/2} - \alpha_2(f) d_{\mathrm{t}}^{1/2}] f^2\} \\ &\quad \exp(-i 2\pi f \frac{d_1}{c_{\mathrm{a}}}) \end{aligned} \tag{5}$$

where β_1 and β_2 are two constants, and d_1 and d_{t} are the projected distances between the two locations i and j in the wave propagation direction and its transverse direction, respectively. f is the frequency in Hz, and c_{a} is the apparent wave propagation velocity in m/s. α_1 and α_2 are two functions

$$\alpha_{\mathrm{k}}(f) = \frac{a_{\mathrm{k}}}{f} + b_{\mathrm{k}} f + c_{\mathrm{k}}, \ \mathrm{k} = 1, 2 \text{ and } f \le 10\,\mathrm{Hz} \tag{6}$$

when $f > 10$ Hz, the α function is a constant and equal to the value at 10 Hz. It is assumed that the ground motion is propagating along the bridge span. Thus $d_{\mathrm{t}} = 0.0$ m, and d_1 equals to the distance between the two piers.

The simulation of the spatially varying ground motions is carried out with the empirical coherency loss function derived from the recorded time histories at the SMART-1 array during the event 45[24]. It has $\beta_1 = 1.109 \times 10^{-4}$, $a_1 = 3.583 \times 10^{-3}$, $b_1 = -1.811 \times 10^{-5}$, and $c_1 = 1.177 \times 10^{-4}$. Ground motions recorded during the event 45 are considered as highly correlated.

The empirical response spectrum of the near-source ground motions within a distance d of 15 km to the surface projection of the rupture plane is adopted from the work

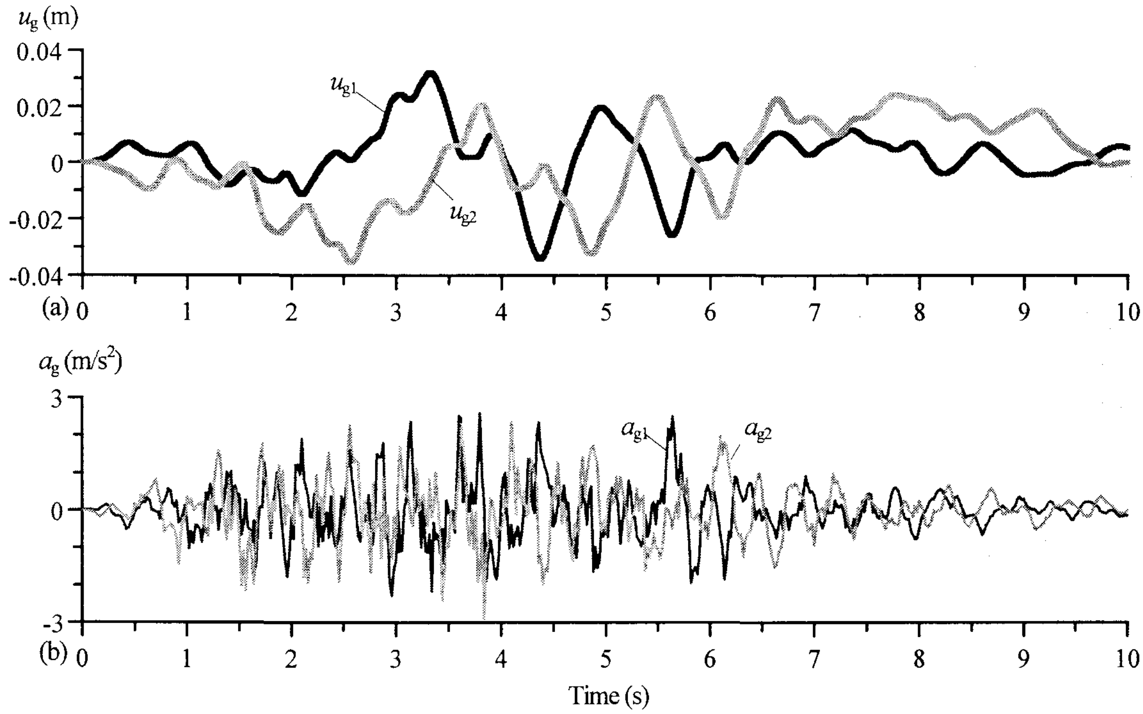

Fig. 2(a) and (b). Simulated highly correlated ground motions at the distance d of 5km with a apparent wave velocity c_a of 200 m/s. (a) Ground displacements u_{g1} and u_{g2} and (b) ground accelerations a_{g1} and a_{g2}

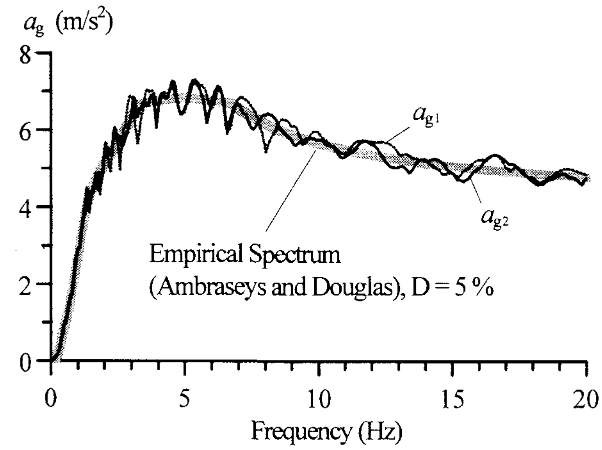

Fig. 3. Empirical spectrum and response spectra of the simulated ground motions a_{g1} and a_{g2}

performed by Ambraseys and Douglas[25]. The authors proposed the spectrum after analyzing time histories of 186 strong motions recorded worldwide with the surface-wave magnitude M_s between 5.8 and 7.8. The empirical acceleration spectrum gives the relationship between the surface-wave magnitude M_s, the distance d, and the local site conditions.

$$log\, y = b_1 + b_2 M_s + b_3 d + b_4 S_A + b_5 S_s \qquad (7)$$

where b_n, n = 1, 2, .., 5 are the frequency-dependent constants. S_A and Ss are the correction factors for the different site conditions. For soft soil, S_A and S_s are 1.0 and 0.0, respectively. The constants b_n are given as functions of vibration period[25]. In this study it is assumed that the surface-wave magnitude M_s is 6.0, and the strong ground motion duration is 10.24 s. The simulation is performed with a time increment of 0.005 s. Details about the simulation procedure are given by Hao et al.[24].

In order to derive more general conclusion twenty sets of spatially correlated ground motions are simulated. Figs. 2(a) and (b) show one set of the highly correlated ground displacements and accelerations with an assumed wave apparent velocity c_a of 200 m/s. Fig. 3 shows that the 5 % damped acceleration response spectra of the simulated ground motions correspond well with the target spectrum.

3. Numerical results

Figs. 4(a) and (b) and Figs. 4(c) and (d) show the influence of the spatially varying ground motions on the activated pounding forces P_F and the relative displacement u_{rel} of the bridge structures with an assumed fixed base and with half-space as subsoil, respectively. $u_{rel}(t)$ is defined as $u_1(t)$ - $u_2(t)$. The positive (opening) and negative (closing) relative displacements show respectively how the adjacent girders move away from and toward each other. When the closing relative displacement exceeds the gap size, pounding will take place. When the opening relative displacement is larger than the seating length, unseating will occur. The maximum opening relative displacement indicates therefore the necessary seating length of the girders. Since it is assumed that both bridge structures have the same fundamental frequency $f_1 = f_2 = 1.0$ Hz, if uniform ground excitation is assumed, both structures will respond in phase. Consequently, the result in Fig. 4 cannot be obtained. The influence of subsoil can be seen in the larger linear relative displacement in Fig. 4(d) (dotted line). While in the case of an

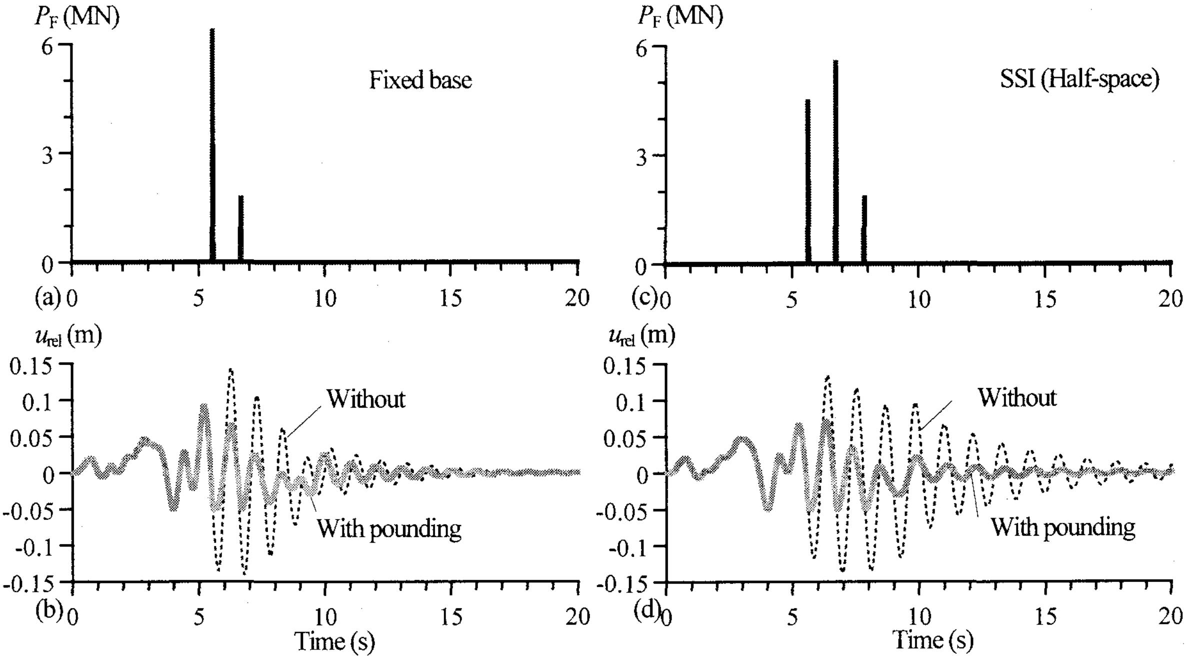

Fig. 4(a)-(d). Influence of soil-structure interaction and no-uniform ground motions on activated pounding force P_F and relative displacement u_{rel} of structures with $f_1 = f_2 = 1.0$ Hz. (a)-(b) Fixed-base case and (c)-(d) half-space case

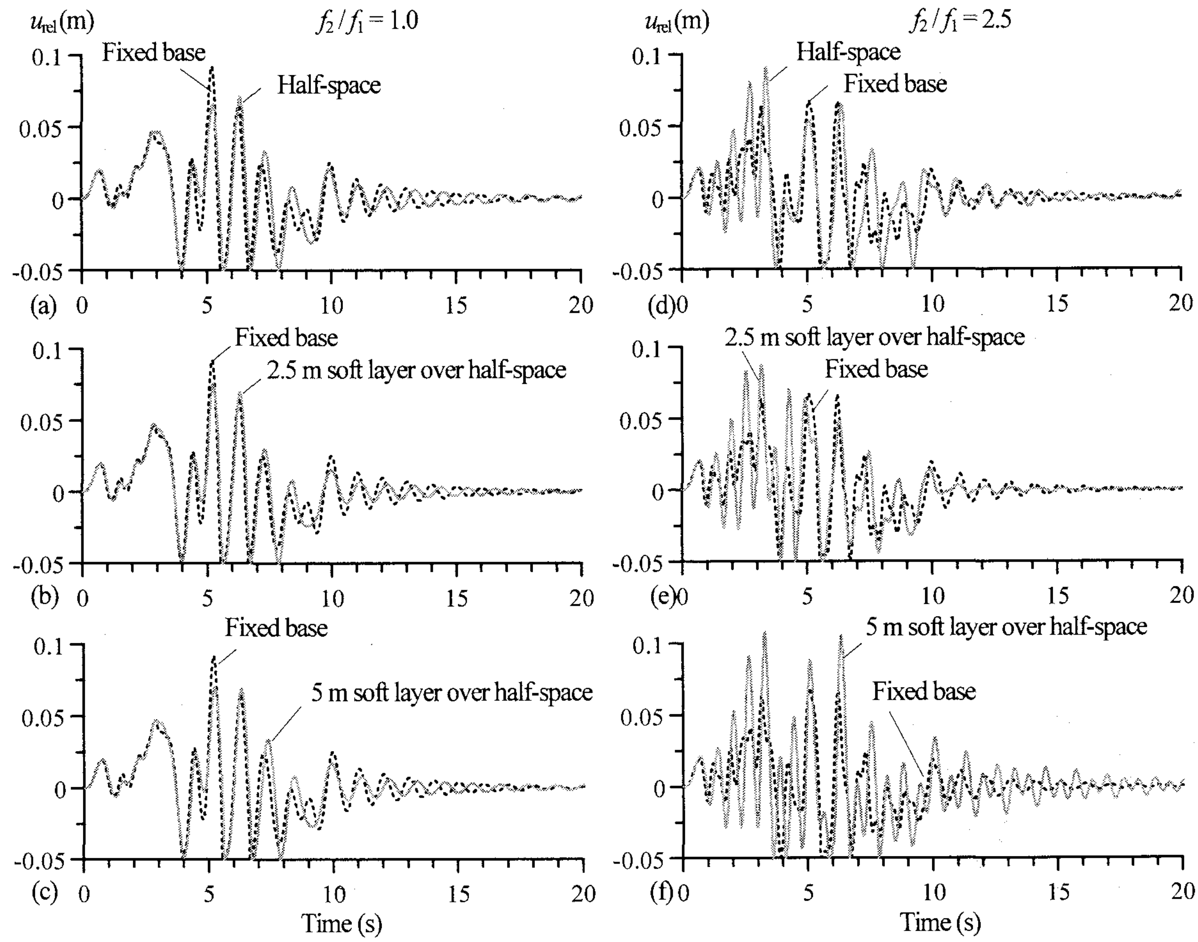

Fig. 5(a)-(f). Influence of soil-structure interaction and the frequency ratio f_2/f_1 on the relative displacement u_{rel}. (a)-(c) $f_2/f_1 = 1.0$, and (d)-(f) $f_2/f_1 = 2.5$

assumed fixed base the non-uniform ground motions will cause only two collisions, a consideration of subsoil leads to additional collision. The result clearly shows that an assumption of uniform ground motions can underestimate the pounding and unseating potential, and consequently the damage potential of the girders.

The left and right columns of Fig. 5 show the influence of the different subsoil on the relative displacement u_{rel} in the cases of $f_2 / f_1 = 1.0$ and 2.5, respectively. Pounding is considered. The reference result is the relative displacement obtained under an assumption of fixed-base bridge structures (dotted line). If both bridge structures have the same fundamental frequency, the subsoil causes a reduction of the relative displacement. All these results also cannot be obtained, if uniform ground excitation is assumed. If the right bridge structure has a higher fundamental frequency of 2.5 Hz, an assumption of fixed base will underestimate the necessary seating length of the girders. The result is expected, since soft subsoil will have especially strong influence on stiff structures. The soft soil layer of 5.0 m thickness has most pronounced effect (Fig. 5(f)). Even though the soft layer is not on bedrock, the influence of the layer still can be estimated from the frequency of a layer over bedrock. For the thickness of 5.0 m the frequency f_{layer} is 5.0 Hz, while the layer with the thickness of 2.5 m has the frequency of 10.0 Hz. As expected the soil layer with the thickness of 5.0 m has therefore the strongest influence on the relative displacement.

In Fig. 6(a) the relative displacement response spectrum recommended by Japanese Road Association (JRA)[20] (dotted line) is compared with the response spectrum (solid line) obtained under the assumption of uniform ground excitation $u_{g1}(t)$ and fixed-base structures. Pounding is not considered. The response spectrum in this study is obtained from twenty sets of ground motions, and ten frequency ratios 0.25 to 2.5 with ratio increment of 0.25 are considered. The JRA response spectrum is generated using 63 strong ground motions of earthquakes in Japan with magnitude 6.5 or above and a focal depth of less than 60 km. Both spectra are normalized with the maximum displacement of the left bridge structure with the fundamental frequency f_1 of 1.0 Hz. The response spectrum obtained using the simulated ground motions has smaller values but the same tendency. It is expected, that both spectra will not have the same values, because they are based on different conditions. At the frequency ratio $f_2 / f_1 = 1.0$ both spectra have zero value, because both bridge structures respond in phase, and

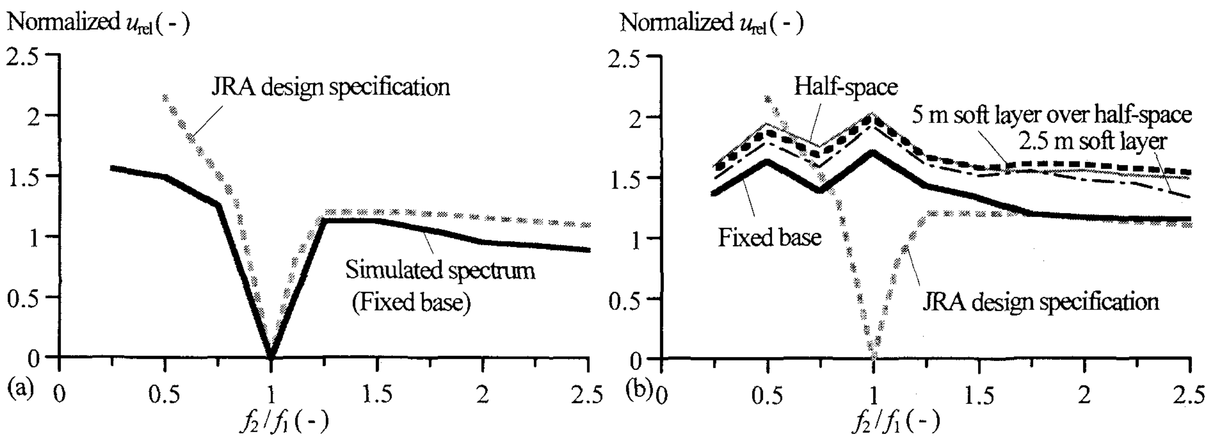

Fig. 6(a) and (b). Normalized relative displacement spectra with f_1 = 1 .0 Hz. (a) Comparison with the design specification spectrum, and (b) influence of spatial variation of the ground motions

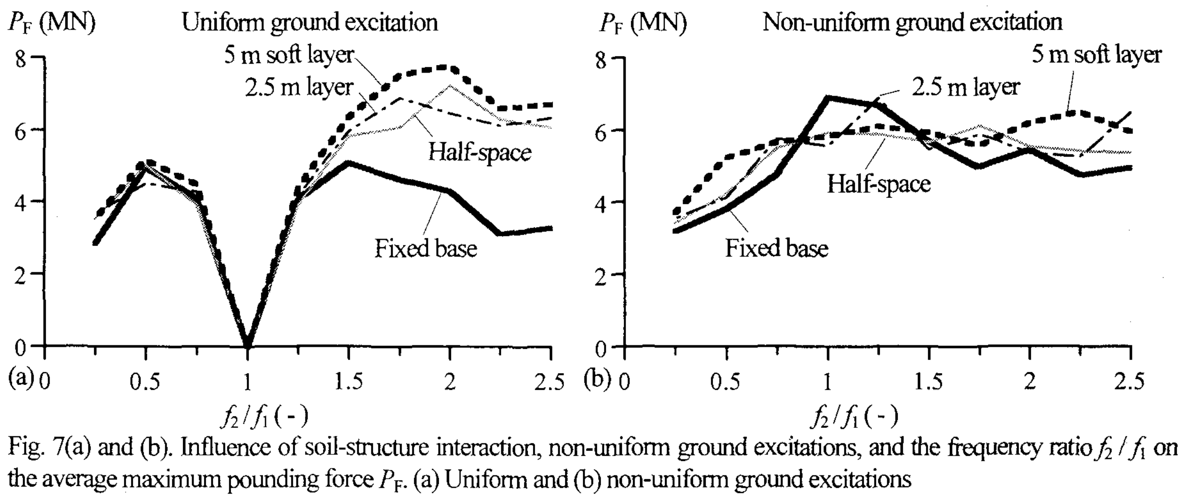

Fig. 7(a) and (b). Influence of soil-structure interaction, non-uniform ground excitations, and the frequency ratio f_2 / f_1 on the average maximum pounding force P_F. (a) Uniform and (b) non-uniform ground excitations consequently no relative displacement takes place. To avoid unseating and pounding damage most design specifications, e.g. Caltrans, therefore recommend an adjustment of dynamic properties of adjacent structures so that their fundamental frequency ratio is as close as possible to unity.

In Fig. 6(b) the JRA response spectrum is compared with the spectra obtained using non-uniform ground excitations. For the normalizing of the relative displacement spectrum values the maximum response of the left structure with the respective support conditions is used. The average maximum displacements with fixed base, 2.5 m and 5.0 m soft layer over hard soil and half-space as subsoil due to twenty sets of ground motions are 7.86 cm, 7.67 cm, 7.48 cm, and 7.20 cm, respectively. The difference between these maximum displacements alone show

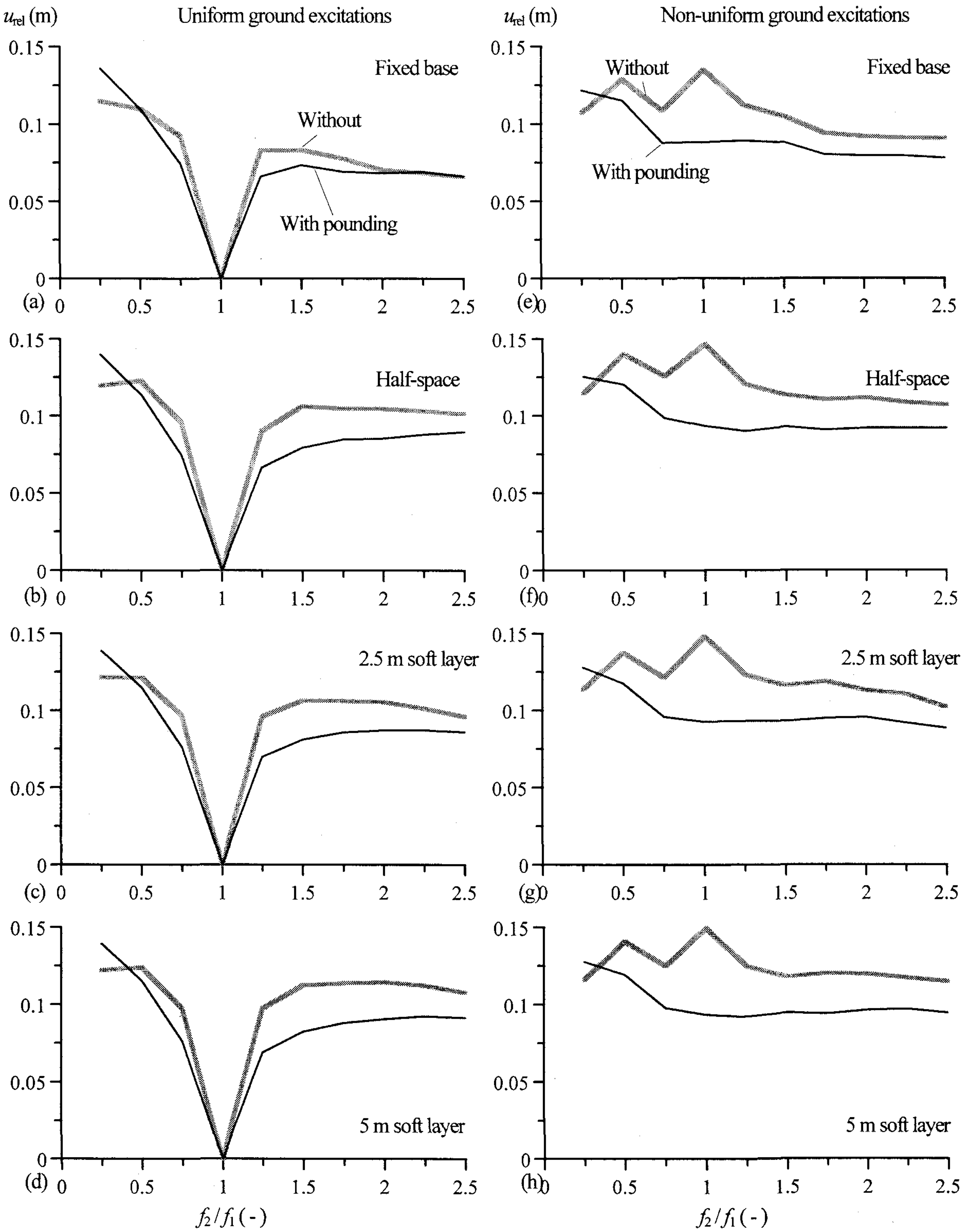

Fig. 8(a)-(h). Influence of soil-structure interaction, pounding, non-uniform ground excitation on relative displacement u_{rel}. (a)-(d) Uniform, and (e)-(h) non-uniform ground excitations already the complexity of the problem, because the response of the structure depends not only on the relationship between the fundamental frequencies of the soil-structure systems and the dominant frequencies of the ground motions, it also depend on the spatial variation of the ground motion, frequency-dependent stiffness of the subsoil and the soil layer frequency. In general, we can expect that with increasing layer thickness the radiation damping becomes larger. A structure on half-space will experience larger radiation damping than a structure on a soft-soil layer with certain thickness. However, in reality radiation damping is also frequency-dependent. The relative displacement values depend additionally on the quasi-static and dynamic response of both soil-structure systems to spatially varying ground motions and on the relationship between the dynamic properties of the adjacent structures. Even though the response spectrum obtained using uniform ground motions has smaller values than JRA spectrum values, the non-uniform ground motion spectrum values are much larger in almost all frequency-ratio range. Only the fixed-base condition (bold solid line) has similar values in the higher-frequency ratio range above 1.7. Half-space as subsoil produces the largest spectrum values in the frequency ratios below 1.0. In the higher frequency-ratio range above 1.5 the soil layer with the thickness of 5.0 m produces the largest values. Compared to the fixed-base condition the subsoil cause in the considered cases clearly larger values. The result shows that the relative displacement response spectrum recommended by JRA clearly underestimates the spectra due to non-uniform ground motions, especially at the frequency ratio f_2 / f_1 of 1.0, where equal fundamental frequencies are supposed to prevent girders from unseating.

Figs. 7(a) and (b) show the mean values of the maximum activated pounding force P_F due to uniform and non-uniform ground motions, respectively. Twenty sets of ground motions are considered. In the case of uniform ground excitation the assumption of fixed-base structures underestimate the pounding forces in the frequency-ratio range above 1.25. The soil layer with the thickness of 5.0 m causes the largest pounding forces in this high frequency-ratio range. The non-uniform ground excitation provides different results. While girders of the fixed-base structures will experience largest pounding forces, if the frequency ratios are around 1.0 and 1.25, below and above this range the soil layer with the thickness of 5.0 m will cause the largest pounding forces.

Fig. 8 shows the influence of pounding, soil-structure interaction and non-uniform ground motions on the relative displacement spectrum. The left and right columns are the results due to uniform and non-uniform ground excitations, respectively. The uppermost results are obtained if bridge structures with an assumed fixed base are considered. The second, third and lowest rows are the results with half space, soil layer of the thickness of 2.5 m and 5.0 m on hard soil as

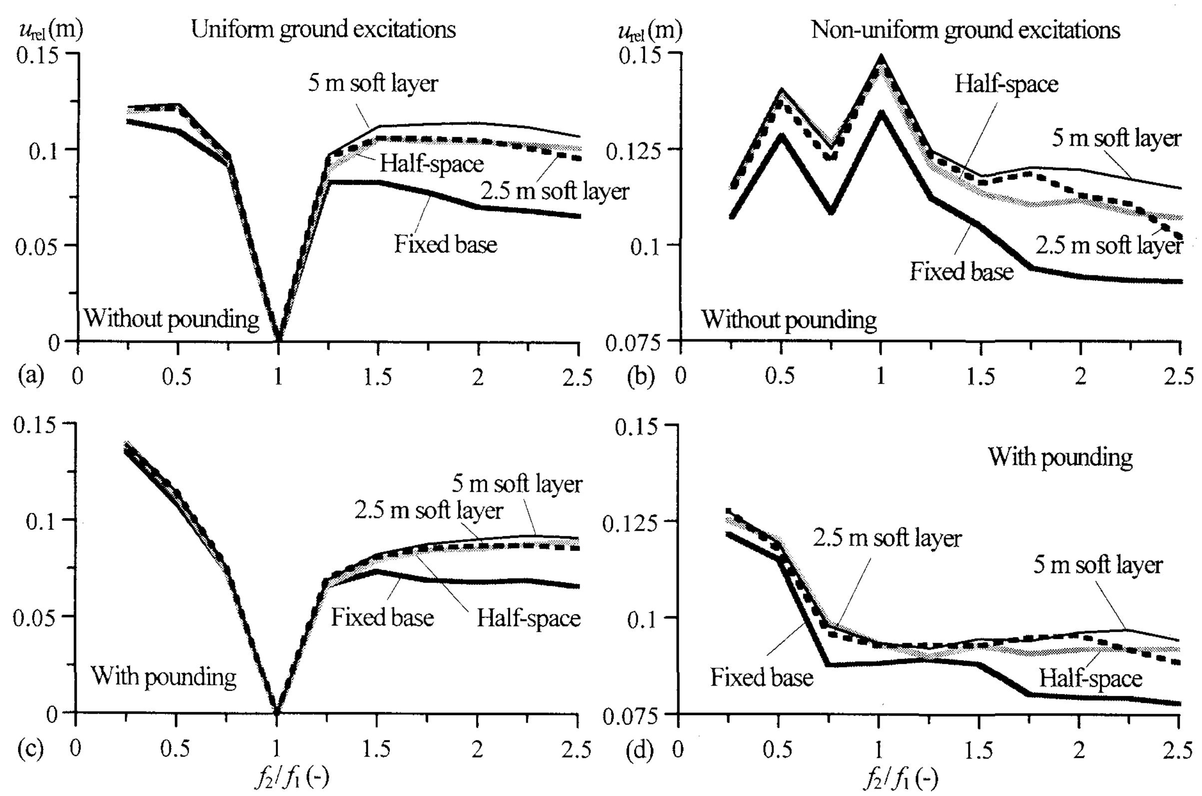

Fig. 9(a)-(d). Effect of the soil condition, frequency ratio f_2/f_1, pounding and non-uniform ground excitation on relative displacement u_{rel}. (a) and (c) Uniform and (b) and (d) non-uniform ground excitations subsoil, respectively. All results show that only if the neighbouring bridge structure is very flexible ($f_2 / f_1 = 0.25$) pounding will cause an amplification of the relative displacement (thin line). In the considered case pounding cause in general a reduction of the relative displacement. Non-uniform ground motions produce in all subsoil cases almost the same displacement values, and they are larger than the fixed-base values (compared Figs. 8(f), (g) and (h) with Fig. 8(e)).

To have a better insight into the simultaneous influence of the different soil conditions, structural fundamental frequency ratio and spatial variation of the ground excitation, the result with and without pounding effect from Fig. 8 are displayed together in Figs. 9(c) and (d) and Figs. 9(a) and (b), respectively. Without pounding effect the results obtained under an assumption of fixed base structures clearly underestimate the relative girder displacement in all considered frequency ratios, if the more realistic spatially varying ground excitations are considered. Subsoil of 5 m soft layer on hard soil causes in the considered cases the largest relative displacement. One possible reason -as mentioned earlier- is that the layer frequency f_{layer} of 5.0 Hz is closest to the considered structural fundamental frequencies. The subsoil can therefore strongly affect the structural responses. With an assumption of uniform ground excitations pounding amplifies the relative displacement in the low frequency ratio range and at the same time suppresses the effect of the different soil conditions. In high frequency ratio range above 1.25 pounding reduces the relative displacement in all considered support conditions. Similar finding can be observed from a comparison of the results due to non-uniform ground excitation in Figs. 9(b) and 9(d).

4. Conclusions

Two neighbouring bridge structures are considered to study the influence of pounding, soil-structure interaction in different soil conditions, and non-uniform ground excitation on the relative displacement response spectrum. Each girder displacement is described by a single-degree-of-freedom. The bridge structures with their foundations and the subsoil are modeled using finite elements and boundary elements, respectively. The spatially varying ground motions are simulated stochastically using an empirical spectrum for near-source earthquakes and an empirical coherency loss function. The current relative displacement response spectrum proposed by the Japanese Road Association is also evaluated. In total 10 frequency ratios are considered. For each frequency ratio 8 cases resulting from different ground conditions and load assumptions are analyzed. In each case 20 sets of ground motions are applied. The considered soil conditions are half-space and a soft layer of different thickness over hard soil, and the load assumptions are uniform and non-uniform ground excitations.

The study reveals:

If spatial variation of the ground motions is unavoidable, adjusting the fundamental frequencies of the adjacent structures cannot prevent the occurrence of relative displacement of bridge girders. An adjustment of frequency ratio to unity can even amplify the pounding and unseating potential of the bridge girders.

In general, pounding will reduce the relative displacement. If the neighbouring structure is much more flexible, pounding can amplify the relative displacement.

In the considered cases, if non-uniform ground motions are applied, the different subsoil conditions produce almost the same relative displacement response spectrum values.

The relative displacement response spectrum proposed by the Japanese Road Association does not considered the influence of non-uniform ground excitations, pounding, and soil-structure interaction. The combined effect of these factors can strongly amplify the spectrum values, especially around the frequency ratio of 1.0.

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