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Effect of simultaneous spatial near-source ground excitation and soil on the pounding response of bridge girders

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The study addresses the influence of the characteristic of the near-source Kobe earthquake; the spatially varying ground motions, and the soil-structure interaction on the pounding response of two adjacent bridge frames. For the simulation of the non-uniform ground excitation of the bridge piers the Japanese design spectrum for soft soil is used. The numerical result of the investigation of the simplified model reveals that -depending on the ratio of the natural frequencies of the two adjacent structures- soft soil can cause a strong amplification of the pounding forces on structures. When their natural frequencies are close to each other non-uniform ground motions may significantly increase the relative structural responses. The result of the finite-element and boundary-element model confirms the significant effect of the spatial ground excitation and the soil-structure interaction.

Key Words: pounding, spatially varying excitation, soil-structure system, near-source earthquakes

1. Introduction

Pounding damage has been observed not only at buildings but also at many bridge decks in major earthquakes of the past like the 1994 Northridge earthquake[1], the 1995 Kobe earthquake[2], and the 1999 Chi-Chi earthquake[3]. Moreover unseating of bridge decks was also observed in many cases. Subsequent investigation revealed that on some occasions the unseating of decks were caused by the combining effect of tilting of the bridge pier due to liquefaction and pounding of the adjacent span induced by large relative displacement between the bridge decks[2),4)]. Even if the bridge deck does not fall down, the damage due to pounding can significantly affect the safety and serviceability of the bridge. Bridge integrity is very essential, especially, just after earthquakes for the rescue works.

The main objectives of many investigations on pounding between bridge girders are to understand the causes that lead to poundings, to determine the required distance to avoid pounding, to develop proper reduction measures. Nishikawa et al.[5], for example, investigated the damage mechanism by using the observation data of the damage of bridge decks during the Kobe earthquake. Ruangrassamee and Kawashima[6] proposed design spectra for determining the necessary seat length of bridge girder. Oshima et al.[7], Sato et al. [8], Jankowski et al.[9] investigated measures to reduce pounding effect.

Earthquake ground motions at different locations can significantly vary depending on the characteristic of the earthquake source, the soil properties in the path of the wave propagation, and at the considered local sites. The larger the distance between the bridge piers is, the more likely the piers will experience varying ground excitations. Many studies on spatial variations of earthquake ground motions are performed in past 30 years, e.g. Hao et al.[10] proposed a refined empirical coherency loss function, which is significant for the simulation of the spatial ground motions. Most of the studies of structural poundings assumed ground motion at multiple structural supports as uniform. Only a few investigation considered the spatial ground motion effect, e.g. Jeng and Kasai[11] developed a spectral difference method to estimate the required separation distance to avoid pounding, Hao et al.[12),13),14),15)] estimated the required distance depending on the structural characteristic and spatial ground motion. Using random vibration method Hao[16] analyzed the required seating length to avoid unseating and pounding of bridge decks. Zanardo et al.[17] studied girder pounding response to varying ground motions including phase shift and coherency loss.

Many works on pounding between bridge girders are published in the past years; however, the effect of soil-structure interaction is normally neglected. If the soil effect is considered at all, then it is considered only approximately by using frequency independent soil stiffness, e.g. Zhu et al.[18] and Kim et al.[19]. Investigation on the simultaneous effect of the spatially varying near-source ground excitation and the soil-structure interaction on pounding response of bridge girders is not known.

In this study the response of two adjacent bridge girders is considered. The ground excitation is the ground motion of the 1995 Kobe earthquake at the Kobe Port Island, the Kobe University as well as the simulated non-uniform ground motions. The soil is assumed to be a half-space.

2. Numerical procedure

2. 1 Ground motions

In order to investigate the effect of the characteristic of near-source earthquakes on the pounding response of the bridge decks the 1995 Kobe earthquake is considered. In this study the near-source earthquake is defined when the epicentral distance of the observation locations is less than 50km. Fig. 1(a) and (b) show the chosen ground motions in the north-south direction at the Kobe Port Island with an epicentral distance of about 20km, and in the north-south direction at the Kobe University with an epicentral distance of around 25km, as well as their corresponding response spectra with a damping ratio of 5%, respectively. Both time histories have strong long-period pulses. In current investigation only horizontal ground motions are considered.

To investigate the influence of the non-uniform ground excitation on the pounding behaviour of bridge girder the ground motion type II of the Japan design specification[20] is used. The design spectrum was determined simply by taking envelopes of the response acceleration of major strong motions recorded at Kobe in the 1995 Kobe earthquake. Fig. 2(b) shows the chosen response spectral acceleration at the soft soil site with the largest value of $15 m/s^2$ at the frequency range between 0.67Hz and 2Hz. The time history of the spatially varying ground motions $a_{g1}(t)$ and $a_{g2}(t)$ are stochastically simulated. The motions are simulated with duration of 20.48s and a time increment of 0.01s. The seismic wave is assumed to propagate in the longitudinal direction of the bridge. It is simulated with the assumption of seismic wave apparent velocity of 200m/s. Apparent velocity is the projected propagation velocity in the horizontal direction, it depends on seismic wave velocity and incident angle. It is in the range between shear wave velocity and infinity. If seismic wave consists of mainly shear wave and

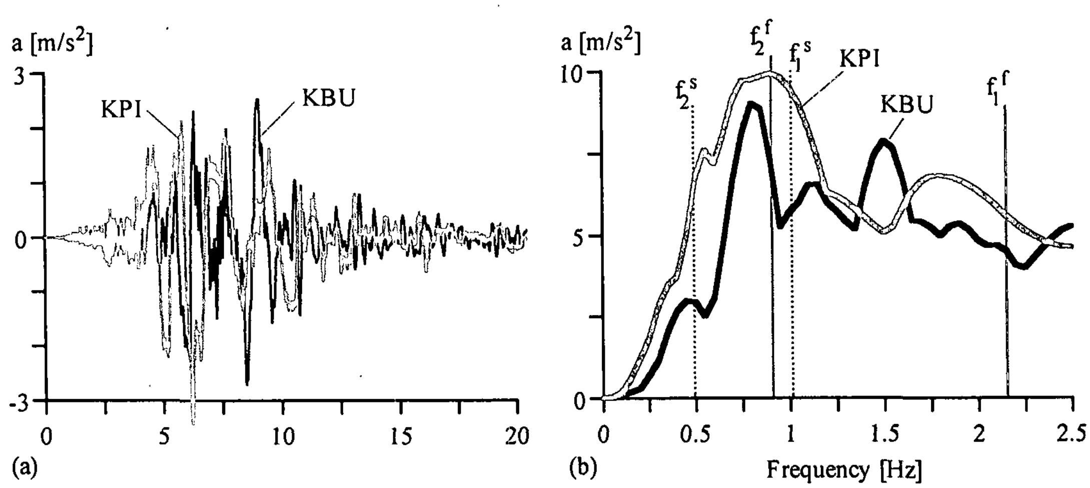

Fig. 1(a) and (b). Kobe earthquake at the Kobe Port Island and Kobe University. (a) Time histories and (b) Response spectra

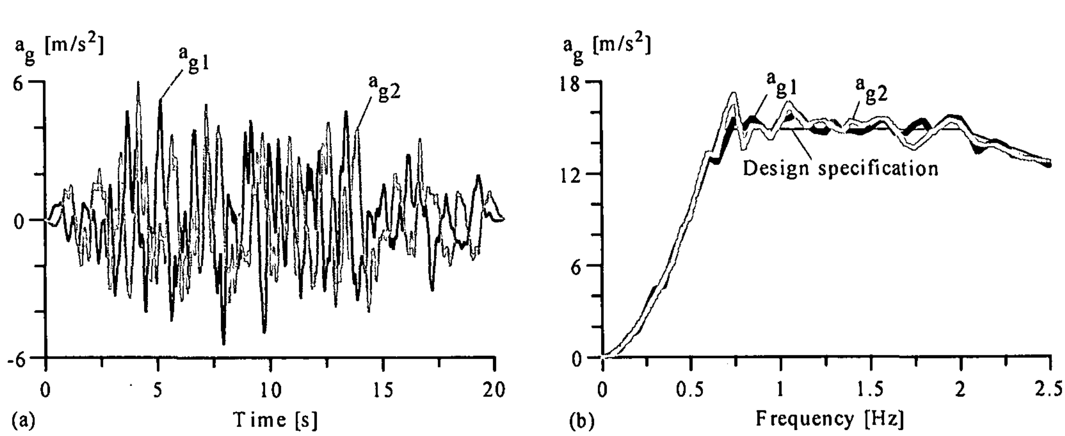

Fig. 2(a) and (b). Spatial ground motions according to Japanese design specification. (a) Time histories and (b) Response spectra comes into the site horizontally, the apparent velocity equals shear wave velocity of the site. If wave penetrates into the site vertically, the apparent velocity is infinity. The empirical coherency loss function derived from the recorded time histories at the SMART-1 array during the event 46 is employed. Changes between any pair of the simulated spatial motion in the same direction (horizontal or vertical) follow the empirical coherency loss function. Fig. 2(a) shows the simulated time histories. The corresponding response spectra with a damping ratio of 5% in Fig. 2(b) show that the frequency content of the simulated spatial ground motions is compatible with the design spectrum. The coherency loss function between simulated time histories with a separation distance of 100m and the empirical coherency loss function is given in Fig. 3. Details about the ground motion simulation and the information regarding the empirical coherency loss are given by Hao et al.[10].

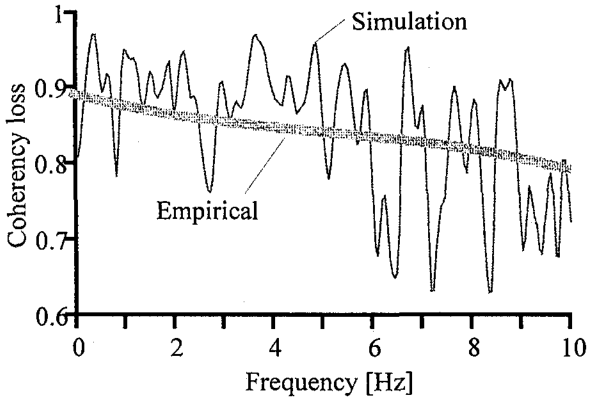

Fig. 3. Coherency loss function

2. 2 Frame bridge with subsoil

In the study the bridge with two adjacent frames in Fig. 5(a) is considered. It is adopted from the work performed by DesRoches and Muthukumar[21]. In their work each of the multiple-frames are modeled by a single-degree-of-freedom (SDOF) system, and the effect of soil as well as spatial ground motion is not considered. In contrast to their investigation we modeled the bridge system by SDOF model and by finite element and boundary element model.

In the SDOF model a mass is attached to the top of a column, which is fixed at the middle of a beam. The length of the beam is 9m, and it is supported by three vertical springs and viscous dampers (Fig. 4). The mass simulates the girder of the bridge, the column the piers of the bridge frame, and the vertical springs and dampers the subsoil. The equivalent mass M_e and height h_e of the structure can be determined by using the effective earthquake load and the value of the natural vibration mode of the considered structure[22]. Kodama and Chouw[23] demonstrated that the dynamic response is not only determined by the relationship between the frequency content of the ground excitation and the natural frequency of the structure, but also by the slenderness of the structure, which is defined by the ratio of the structure height and foundation width. This effect cannot be seen if the influence of the soil is ignored.

In order not to have an additional influence of the system slenderness, we keep M_e of 2059.59kNs^2/m, and h_e of 13.3m constant. A change in the natural frequency of the system is therefore only caused by the change of the flexural rigidity EI of the column. The considered natural frequency of the adjacent structures ranges from 0.1Hz to 2Hz with an increment of 0.1Hz. The soil stiffness k_z is the frequency-independent stiffness of a surface foundation on a homogenous half-space[24] with a Poisson's ratio ν of 0.33, and a density ρ of 2000kg/m^3. The viscous damper c_z is chosen in this way that the vertical vibration of the system has a damping ratio of 3%. The estimated soil stiffness and damping values in the analysis with the SDOF model are given in Table 1. The description of the SDOF model is given by Kodama and Chouw[23].

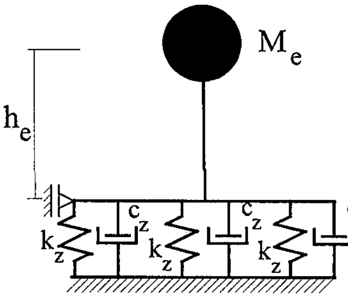

Fig. 4. SDOF model for each of the bridge frame

Table 1. Simplified soil stiffness and viscous dampers

c_s [m/s]	100	200	400	2000
k_z [10^5 kN/m]	3.642	14.57	58.27	1457
c_z [10^2 kNs/m]	9.487	18.97	37.95	189.7

With this simplified model the influence of the soil stiffness, and the characteristic of the ground motion KPI as well as the spatial ground motions $a_{g1}(t)$ and $a_{g2}(t)$ is investigated.

In the more detailed analysis the bridge frames and foundations are modeled with finite elements with uniformly distributed mass and stiffness, and the subsoil with boundary elements in the Laplace domain.

The dynamic stiffness of the structural member is obtained by solving the equation of motion in the axial and lateral direction in the Laplace domain analytically. By assembling the stiffness of each structural member we obtain the dynamic stiffness $[\tilde{K}_b]$ of the bridge. ($\tilde{}$) indicates a vector or matrix in the Laplace domain. A description of the structural stiffness is given by Chouw and Pflanz[25]. The dynamic stiffness $[\tilde{K}_s]$ of the subsoil can be obtained by using the full-space fundamental solution. Details of the determination of the soil stiffness are given by Hashimoto and Chouw[26].

For the two adjacent bridge frames with subsoil the governing equation is

$$\begin{bmatrix} \tilde{K}^{b1}_{bb} & \tilde{K}^{b1}_{bc} & 0 & 0 \\ \tilde{K}^{b1}_{cb} & \tilde{K}^{b1}_{cc}+\tilde{K}^{s1}_{cc} & 0 & \tilde{K}^{s12}_{cc} \\ 0 & 0 & \tilde{K}^{b2}_{bb} & \tilde{K}^{b2}_{bc} \\ 0 & \tilde{K}^{s21}_{cc} & \tilde{K}^{b2}_{cb} & \tilde{K}^{b2}_{cc}+\tilde{K}^{s2}_{cc} \end{bmatrix} \begin{Bmatrix} \tilde{u}^{b1}_{b} \\ \tilde{u}^{b1}_{c} \\ \tilde{u}^{b2}_{b} \\ \tilde{u}^{b2}_{c} \end{Bmatrix} = \begin{Bmatrix} \tilde{P}^{b1}_{b} \\ \tilde{P}^{b1}_{c} \\ \tilde{P}^{b2}_{b} \\ \tilde{P}^{b2}_{c} \end{Bmatrix} \quad (1)$$

The index 1 and 2 stand for the left and right bridge frame, respectively. The index b, s, and c stand for bridge, subsoil, and contact-degree-of-freedoms (cdof) at the interface between the bridge foundations and the soil, respectively. After transforming the ground excitation from the time domain into the Laplace domain

$$\{\tilde{P}(s)\} = \int_0^{\infty} \{P(t)\} e^{-st} dt \ , \quad (2)$$

where the Laplace parameter $s = \delta + i\,\omega$, and $i = \sqrt{-1}$, the linear response of the bridge in the Laplace domain can be determined. A transformation to the time domain gives the time history of the bridge response

$$\{u(t)\} = \frac{1}{2\pi i} \int_{\delta-i\omega}^{\delta+i\omega} \{\tilde{u}(s)\} e^{st} ds \quad (3)$$

The nonlinear analyses are performed subsequently in the Laplace and time domain. We examine the linear response of the girders in the time domain for poundings. If the two girders collide at time t_i we define the difference value and the condensed dynamic stiffness of one of the bridge frame. Since the two bridge frames are in contact, we have to add the condensed stiffness into the stiffness of the uncondensed bridge frame. Using the governing equation of the uncondensed bridge frame of the uncoupled system, this means one part of the equations (1) without the couple terms $\tilde{K}^{s12}_{cc}$ and $\tilde{K}^{s21}_{cc}$, we then obtain the corrective results. After transforming the results into the time domain, the previous linear results can be corrected from the time t_i. If the bridge frames are separated at time t_j, then the unbalanced load is equal to the contact forces. By using the governing equation of the uncoupled system we obtain the corrective results. After transforming this result to the time domain the influence of the separation between bridge decks can be incorporated in the former result. We examine the response whether further pounding occurs. The calculation is complete when no more pounding or separation. A description of the nonlinear soil-structure interaction analysis procedure is given by Chouw[27].

3. Numerical result

3.1 SDOF model

Fig. 6 shows the influence of the stiffness of the subsoil, indicated by the shear wave velocity c_s, and the relationship between the natural frequencies f_2/f_1 of the two adjacent structures on the maximum relative displacement in case of the

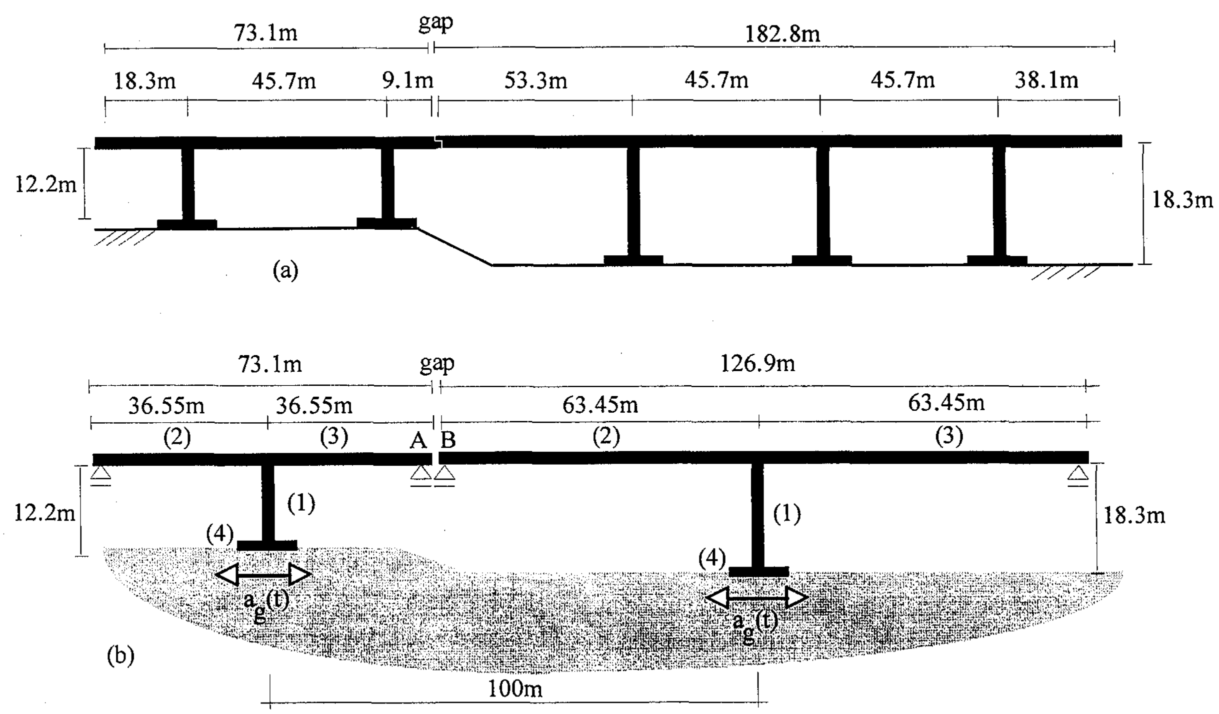

Fig. 5(a) and (b). Idealization of a two-adjacent-frame bridge. (a) Bridge with multiple-pier frames and (b) Single-pier frame bridge with subsoil

KPI ground motion. The soil with the shear wave velocity c_s of 2000m/s can be interpreted as rigid soil. The result of the investigation indicates that the relative displacement is strongly affected by the frequency ratio f_2/f_1. We have decided to use the frequency of 0.9Hz as the reference natural frequency f_1, because the dominant frequency of the ground excitation is located around 0.9Hz.

The result shows that the largest relative displacements occur in zone I. In zone II no soil influence can be observed. In zone III soft soil clearly has significant amplification effect. Since both structure experience the same ground excitation, as expected, there is no relative displacement in case of a ratio $f_2/f_1 = 1$. In the study soft soil is defined when the shear wave velocity in the soil is less than 200m/s. In the later analysis with single pier model a soft soil with the shear wave velocity of 100m/s is assumed.

The influence of the spatial ground excitation can be clearly seen in Fig. 7 in the case of the ratio $f_2/f_1 = 1$. The spatial ground motion effect is independent of the soil stiffness. In zone II the influence of the spatial ground motion becomes pronounced, if we have soft soil. The effect of soil is also significant in the higher ratio range beyond the ratio of 1.6, indicating soil effects become more pronounced when the structure is relatively stiff.

The result of the investigation with simplified models shows that in higher ratio range the effect of soft soil should not be neglected. In case of bridges with a very large distance between the bridge piers the spatially varying ground excitation should be taken into account in the analysis, especially, if the natural frequencies of the adjacent structure are close to each other, and if the ground is soft.

3.2 Single-pier model

In the current study the multiple-pier bridge frames are simplified as single-pier bridge frames (Fig. 5(b)). The material data of the single-pier model is given in Table 2. To verify the accuracy of the single-pier model the response of the bridge girder of the multiple-pier and single-pier model is compared. For this comparison it is assumed that the piers are fixed at the base. The material damping of the bridge structure is defined by the complex Young's modulus, and the real and imaginary part of the modulus are a function of the Kelvin-chain parameters E_1 of 0.1, and E_n of 10^{28}. For these chosen parameters the equivalent damping ratio is about 1.4%. Details of the material damping description are given by Hashimoto and Chouw[26]. It is assumed that the soil is a half-space, and has no material damping.

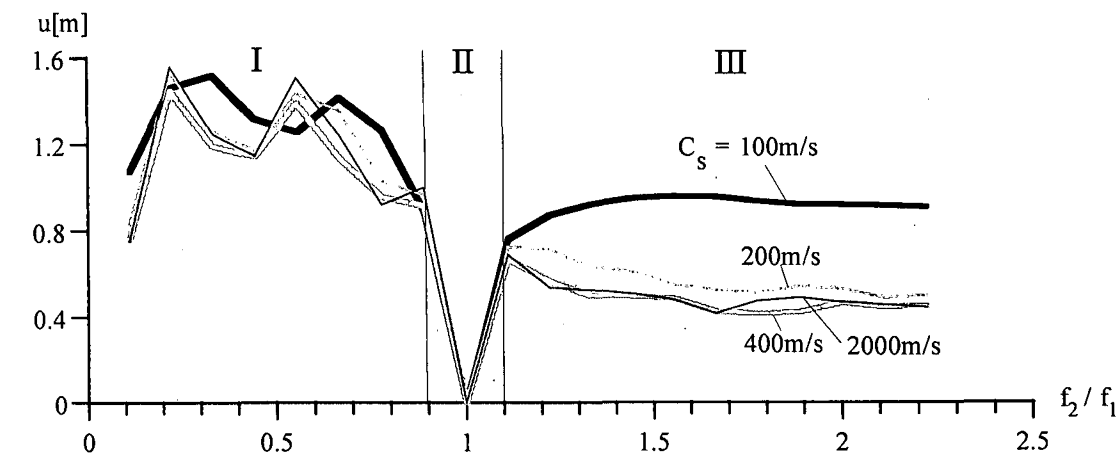

Fig. 6. Soil influence on the maximum relative displacement response of two SDOF-structures

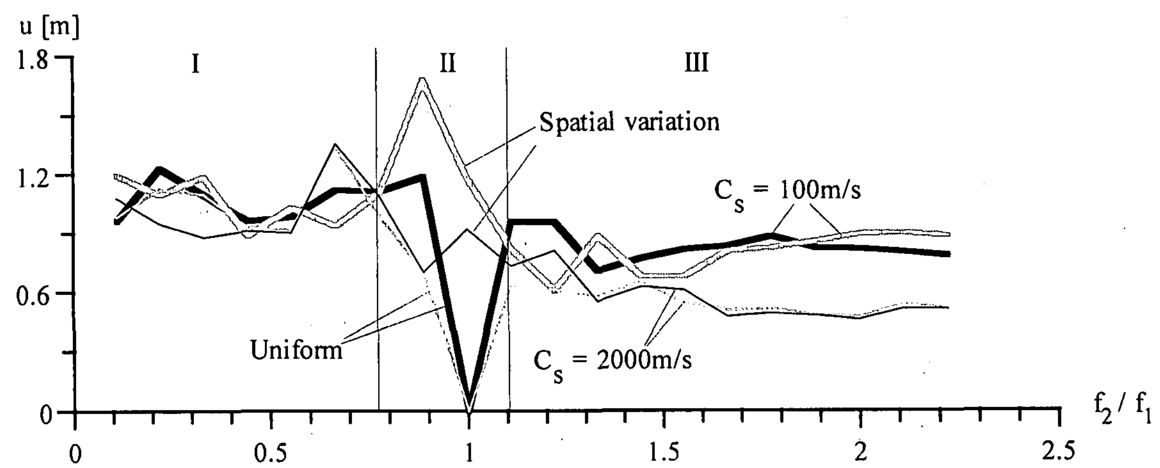

Fig. 7. Influence of soil and spatially varying ground excitation on the maximum relative displacement of two SDOF structures

Fig. 8(a) and (b) shows the horizontal displacement u_A and u_B of the left and right bridge frame due to the KPI ground motions. The left and right bridge frames of the single-pier model with an assumed fixed base have the fundamental frequencies f_1^f and f_2^f of 2.146Hz and 0.905Hz, respectively.

As expected the single-pier model gives larger response with lower frequency. However, the single-pier model behaves very similar as the multiple-pier model, especially, in the period of the strong motions.

Table 2. Material property of the single-pier frame bridge

Member number	Mass [t/m]	EA [10^8kN]	EI [10^8kNm^2]	Length [m]
Left frame				
(1)	5.26	1.407	1.546	12.2
(2)	75.5	63.42	50.49	36.55
(3)	75.5	63.42	50.49	36.55
(4)	91.5	768.6	1024.8	9.0
Right frame				
(1)	7.89	2.111	2.32	18.3
(2)	108.75	63.42	50.49	63.45
(3)	108.75	63.42	50.49	63.45
(4)	91.5	768.6	1024.8	9.0

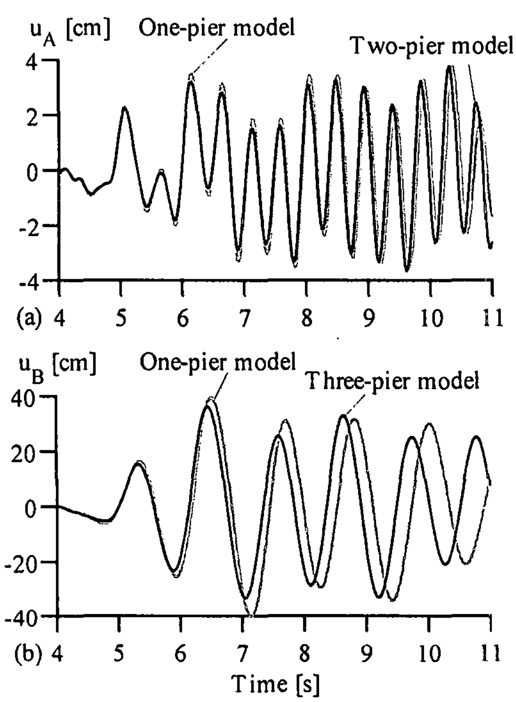

Fig. 8(a) and (b). Comparison of the girder responses obtained from the two models

(1) Effect of soil

Fig. 9(a) and (b) show the effect of poundings on the response of the left and right girder due to the KPI ground motions, respectively. Fig. 9(c) and (d) display the corresponding effect due to the KBU ground motions. No soil-structure interaction is considered. Poundings occur due to the chosen gap of 30cm. In both cases poundings cause a strong increase of the girder response of the stiffer left bridge frame, and a slight decrease of the response of the weaker right bridge frame. Similar pounding behaviour is already observed in many investigations, e. g. the experimental work performed by Chau et al.[28]. The effect of pounding can be clearly seen in the change of the direction of the girder vibration u_A at 6.35s in case of KPI excitation, and at 9.82s in case of KBU excitation.

Fig. 10 shows the influence of pounding as well as soil-structure interaction on the pounding behaviour of the adjacent bridge girders. The poundings occur in case of KPI ground motions due to the considered gap of 70cm, and in case of KBU due to the gap of 10cm. Similar influence can be observed, especially, at 10.72s in case of KPI ground excitation, and at 10.18s in case of KBU ground motions. The weaker right bridge frame vibrates with much larger magnitude than the stiffer left bridge frame. At the instant of impact the weaker frame with larger magnitude pushes the stiffer frame away. This causes a sudden change of the vibration direction of the stiffer frame, and at the same time reduces the vibration amplitude of the weaker frame.

In case of KPI ground motions the considered soft soil clearly amplify the vibration magnitude of both bridge girders, as we can see from a comparison between the girder responses with and without soil-structure interaction. In case of KBU, however, the soil causes only an increase of the response of the girder of the stiffer bridge frame. The girder response of the weaker bridge frame remains almost the same. This result shows that for a proper estimation of the pounding potential the knowledge of the soil property alone is not enough. A simultaneous effect of the soil and the characteristic of the ground motions should be taken into account. As we can see from the relationship between the natural frequency of the bridge frame and the frequency content of the ground excitation in Fig. 1(b) where the vertical solid and dotted bars indicate the location of the natural frequencies of the system with an assumed fixed base and with subsoil, respectively. f_2^f and f_2^s are the natural frequency of the weaker right bridge frame with fixed base and with subsoil, respectively. While in case of KPI ground motion the soil causes a reduction of the spectrum value, in case of KBU ground excitation the soil leads to a much smaller excitation. Therefore the soil does not cause an amplification of the girder response.

(2) Simultaneous effect of soil and spatial ground excitation

While in case of KPI and KBU ground motions the bridge frames with an assumed fixed base will experience different excitation mainly due to the different characteristic of the ground motions. In case of non-uniform ground motions $a_{g1}(t)$ and $a_{g2}(t)$, the bridge frames will experience almost the same excitation, because both ground motions have almost the same characteristic as we can see from Fig. 2(b). The soil will cause almost the same degree of change to the structural excitation.

Fig. 11(a) and (b) show the influence of the spatial ground motions on the girder response u_A and u_B of the bridge frame with an assumed fixed base and with SSI, respectively. In case of uniform ground excitation both bridge frames experience the same ground motion $a_{g1}(t)$, and in case of non-uniform ground excitation the stiffer left bridge frame experience the ground motions $a_{g1}(t)$ and the weaker right bridge frame the ground motions $a_{g2}(t)$. Corresponding to the later arrival time of the ground motion $a_{g2}(t)$ the response of the girders occurs later than the one due to the uniform ground motions.

The soil clearly increases the girder responses. In the case with the apparent wave propagation velocity of 200 m/s the spatial ground motions cause a stronger increase in case of bridge frame with assumed fixed base than in case of bridge frame with SSI. The amplification of the maximum structural response without the pounding effect is given in Table 3.

Fig. 12(a) and (c) show the nonlinear response of the girders in case of bridge frames with fixed base. Fig. 12(b) and (d) display the nonlinear response including the soil-structure interaction.

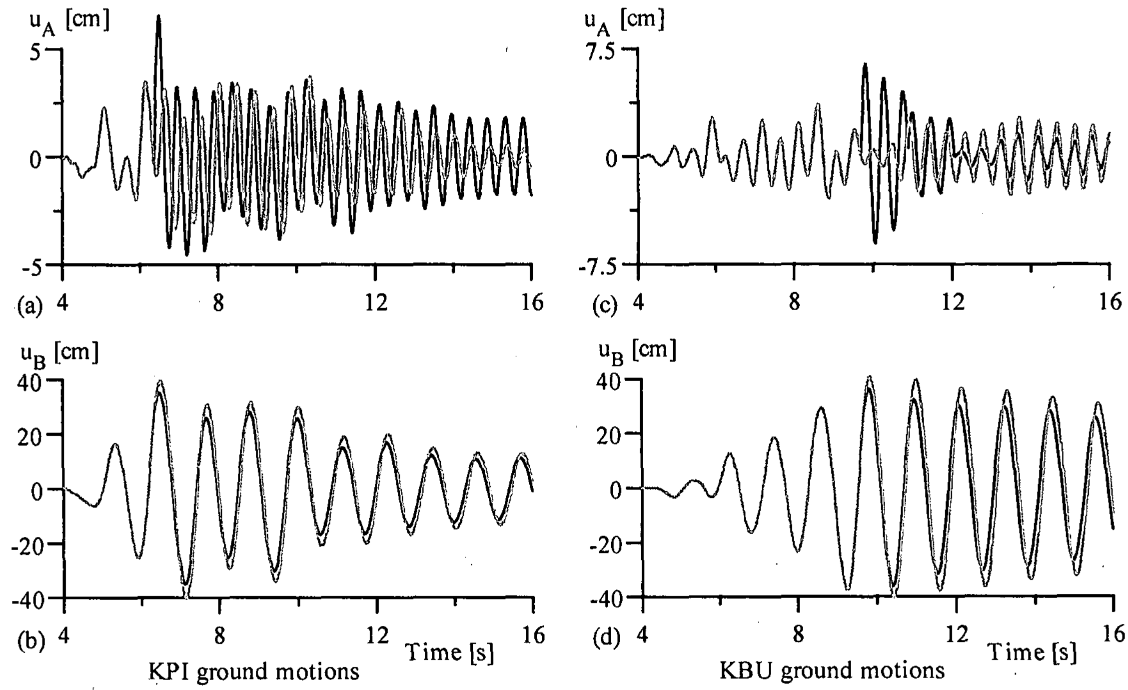

Fig. 9(a)-(d). Pounding effect on the girder responses u_A and u_B without SSI

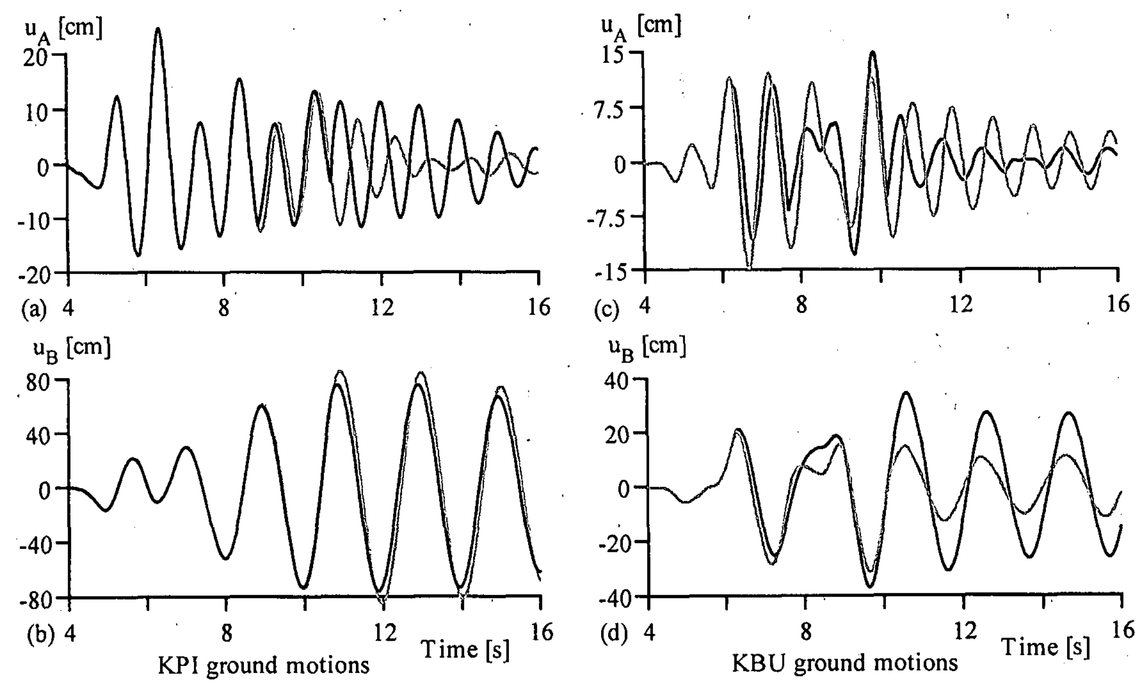

Fig. 10(a)-(d). Pounding effect on the girder responses u_A and u_B with SSI

In Fig. 12(c) and (d) the effect of the non-uniform ground excitation is additionally included. The considered gap in Fig. 12(a)-(d) is 30cm, 60cm, 40cm and 50cm, respectively.

A comparison between the results in Fig. 11 shows that the non-uniform ground excitation causes poundings at different occasions. While the spatial ground motions cause in fixed base case larger necessary separation distance to avoid pounding, in the case of SSI the non-uniform ground excitation cause, in contrast, smaller required separation distance. The reason is that the non-uniform ground motions cause in this particular case a shift of the pounding potential at 10.5s to the later occasion (see Fig. 11(b)). The two bridge girders therefore require smaller separation distance to avoid pounding.

Table 3. Influence of the spatial ground excitation and SSI on the linear girder response

	Fixed base		With SSI	
Excitation of the left bridge frame	$a_{g1}(t)$			
$u_{A,max}$ [cm]	7.4		19.9	
Excitation of the right bridge frame	$a_{g1}(t)$	$a_{g2}(t)$	$a_{g1}(t)$	$a_{g2}(t)$
$u_{B,max}$ [cm]	36.8	45.8	54.7	60.4

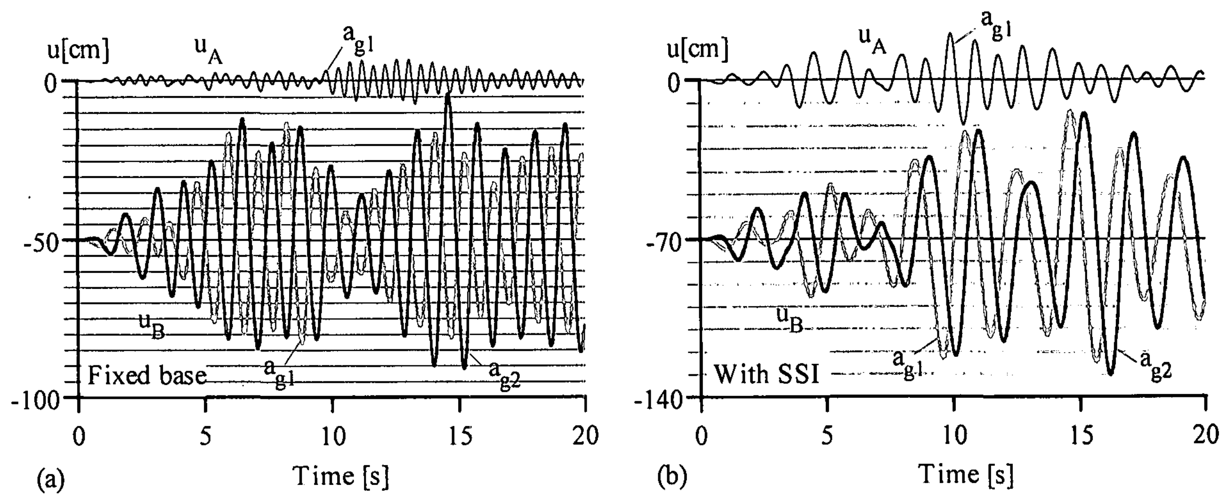

Fig. 11(a) and (b). Effect of SSI and spatially varying ground excitation on the girder responses u_A and u_B without pounding

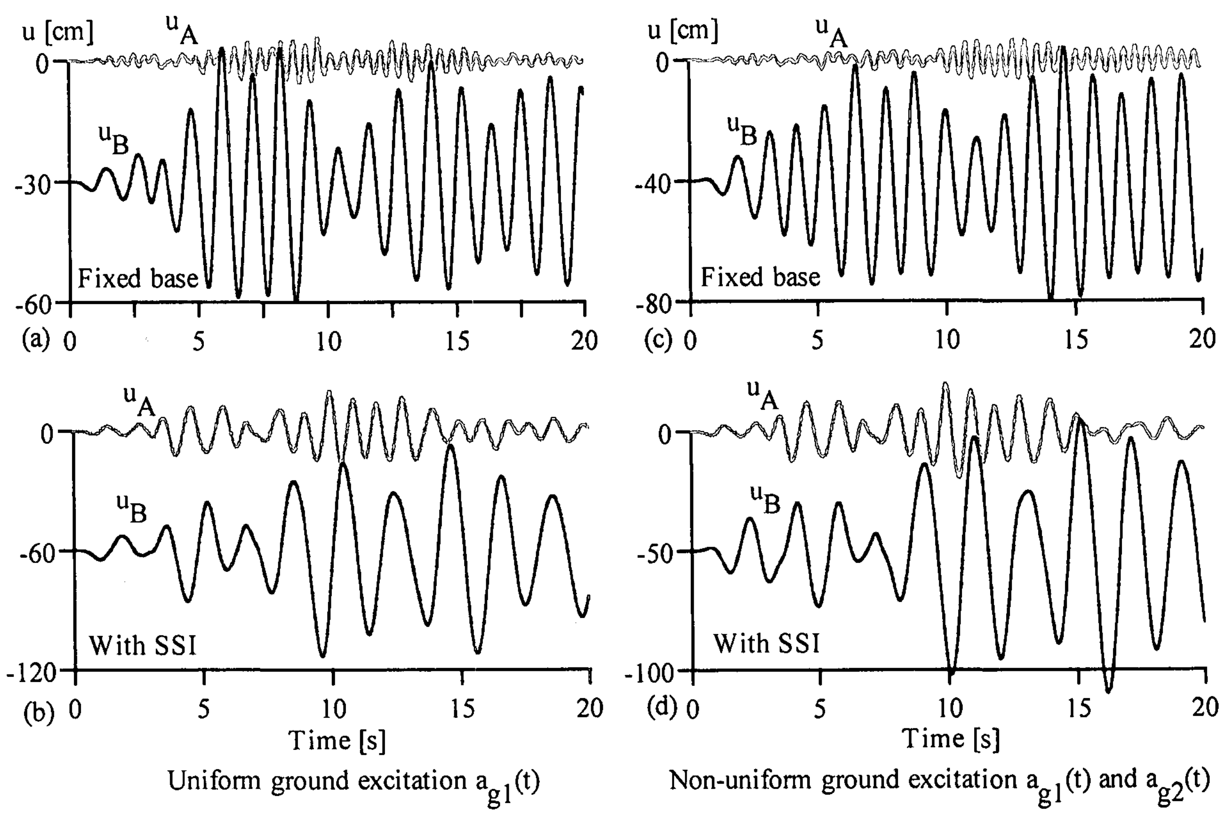

Fig. 12(a)-(d). Effect of SSI, spatially varying ground excitation and poundings on the girder responses u_A and u_B

4. Conclusions

In the numerical analysis the pounding response of two adjacent bridge girders to the near-source ground motions is considered.

The investigation reveals:

If uniform ground motions can be accepted, soft soil should not be neglected if the ratio between the natural frequencies of the adjacent structures is larger than 1.1.

The non-uniform ground excitation will strongly affect the maximum relative displacement, which determines the pounding potential of the adjacent structures, when the natural frequencies of the neighbouring structures are very close to each other. In this case soft soil will additionally amplify the relative displacement between the structures.

In case of non-uniform ground excitation soft soil has also a strong amplification effect, if the natural frequency ratio is larger than 1.6.

Soft soil amplifies in general the girder response. The existing soft soil, however, does not necessarily cause an amplification of the girder responses, since the magnitude of the response is also determined by the actual excitation, which is defined by the relationship between the frequency content of the ground motions and the lower natural frequency of the system *bridge with subsoil.*

The result of the bridge frame investigation shows that the non-uniform ground excitation causes in case of the bridge with fixed base a larger required distance to avoid pounding. If the soil-structure interaction is considered as well, smaller distance is necessary. The reason is that the non-uniform ground excitation causes an unequal shift of the occurrence of the peak response of the adjacent girders. Therefore pounding will occur on another occasion, and causes in the considered case a smaller relative displacement.

The conclusion is valid only for the considered assumptions. More investigations on the influence of the non-uniform ground excitation and soil-structure interaction in relation to the dynamic characteristics of the bridge are necessary for a better understanding of the relationship between the near-source ground motion characteristics and pounding behaviour of bridge girders.

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