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Influence of SSI and frequency content of non-uniform ground motions on bridge girder poundings

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The study addresses the effect of the characteristics of Japanese design spectra on the pounding potential of two bridge girders. The spatially varying ground motions for soft; medium and hard soil condition are simulated stochastically. The effect of non-uniform ground displacements and accelerations, as well as soil-structure interaction is studied. The common design criterion for pounding mitigation is evaluated. The study reveals that a consideration of uniform ground excitation can strongly underestimate the pounding potential, especially when the ground motions have low dominant frequencies, and the soil is soft. The closer are the vibration frequencies of the adjacent girders, the more prominent is the non-uniform ground motion effect. The design recommendation for adjusting the structural frequencies should not be applied, if the ground is soft and a uniform ground excitation cannot be ensured.

Key Words: frequency content, girder pounding, spatially varying excitation, soil-structure system, design specification

1. Introduction

Poundings between adjacent structures occurred in almost all major earthquake events in the past, such as the 1989 Loma Prieta earthquake[1], the 1994 Northridge earthquake[2], the 1995 Kobe earthquake[3] or the 1999 Chi-Chi earthquake[4]. Especially after the 1985 Mexico earthquake damages due to structural poundings have attracted the interest of many researchers. In order to avoid poundings or at least to minimize the pounding effect on structures many investigations have been performed. In many design specifications, e.g. Caltrans Seismic Design Cretaria[5], recommendations for pounding mitigation are given. Hao and Shen[6] proposed numerical procedures for determining the required gap to avoid poundings between buildings. Ruangrassamee et al.[7], Zhu et al[8] and Oshima et al.[9] investigated possible measures for reducing pounding effect at bridge girders. In the case of buildings poundings only the participating buildings are affected. In the case of bridges, pounding damages might result in closure of the bridge that could affect the whole region. Buildings collide each other mainly due to insufficient distances between them and due to their different dynamic properties. Because of their close distance it can be assumed that the adjacent buildings experience the same ground excitation. The out-of-phase vibrations are mainly caused by their different dynamic behaviour. Therefore many design regulations recommend an adjustment of structural natural frequencies, so that the structures will vibrate as synchronous as possible. In the case of bridges often the piers are located far from each other. Since the seismic waves will never be able to reach the two distant pier locations at the same time, phase differences in the ground motions at the locations always exist. Depending on the soil conditions and wave characteristics it is very unlikely that the ground motions at those locations are perfectly correlated. Studies on the effect of non-uniform ground motions on buildings also exist[10, 11, 12]. Research on bridge girder poundings due to non-uniform ground excitations, however, is still very limited[13, 14], especially when the soil-structure interaction (SSI) effect[15, 16] is included. In most of the investigations of bridge girder poundings uniform ground excitations are assumed, e.g. Ruangrassamee et al.[7] and Zhu et al.[8]. If the subsoil effect is included, a frequency-independent soil stiffness is assumed, e.g. Zhu et al.[17]. The effect of soil-structure interaction, however, not only depends on the soil profile and material but also on the vibration frequencies of the foundation. The stiffness of the supporting soil at the foundation depends consequently on the vibration frequencies as well. In this study the influence of the frequency content of non-uniform ground motions and the SSI effect with frequency-dependent soil stiffness on the girder pounding potential, as well as the recommendation of design regulations for pounding mitigations are studied.

2. Bridge structure and spatial ground motion simulation

2. 1 Bridge structure

In the investigation the two bridge frames in Fig. 1 is considered. The left and right frames have two and three piers, respectively. The bridge is adopted from the work of DesRoches and Muthumar[18]. In their investigations each of the frames are described by a single-degree-of-freedom system with an assumed fixed base, and uniform ground excitations were assumed. In this study the bridge frames with multiple-piers (grey piers) are idealized by frames with single pier (black pier) in the middle and movable supports at each girder end. The properties of the single-pier bridge structures were chosen so that they have during the strong ground motions similar response behaviour as the multiple-pier bridge structures. A comparison of the responses of both model, the single-pier and multiple-pier model, was presented in the previous work[15]. The distance between the two single piers is 100 m. This work is a continuation of the previous studies[15, 16].

In the previous study[15] the effect of the ground displacements was not considered. In another investigation[16] the empirical near-source ground motion response spectrum proposed by Ambrasey and Douglas[19] was used. None of these studies considered the influence of the frequency content of the ground motions. In this work the influence of the frequency content of the spatially varying ground motions in relation with the ratio of the bridge-frame fundamental frequencies and SSI are considered.

The material data of both bridge frames is given in Table 1. The number of the structural members is indicated in Fig. 1. The material damping of the structural members is described by a complex modulus of elasticity[20]. For the chosen parameters the bridge frames have an equivalent damping ratio of about 1.4 %. The fundamental frequency of the left and right bridge frame with an assumed fixed base is 2.14 Hz and 0.905 Hz, respectively. The frequency ratio has therefore the value of 0.42.

For the soft and medium soil condition the subsoil is assumed to be a half-space with the shear wave velocity of 100 m/s and 200 m/s, respectively. The soil has the density of 2000 kg/m^3 and the Poisson's ratio of 0.33. To limit the number of the influence factors only the radiation damping is considered. In the case of hard soil condition a rigid base is assumed. The left and right bridge pier experiences the horizontal ground motions $a_{g1}(t)$ and $a_{g2}(t)$, respectively. In the case of uniform ground motions both piers have the excitation of $a_{g1}(t)$.

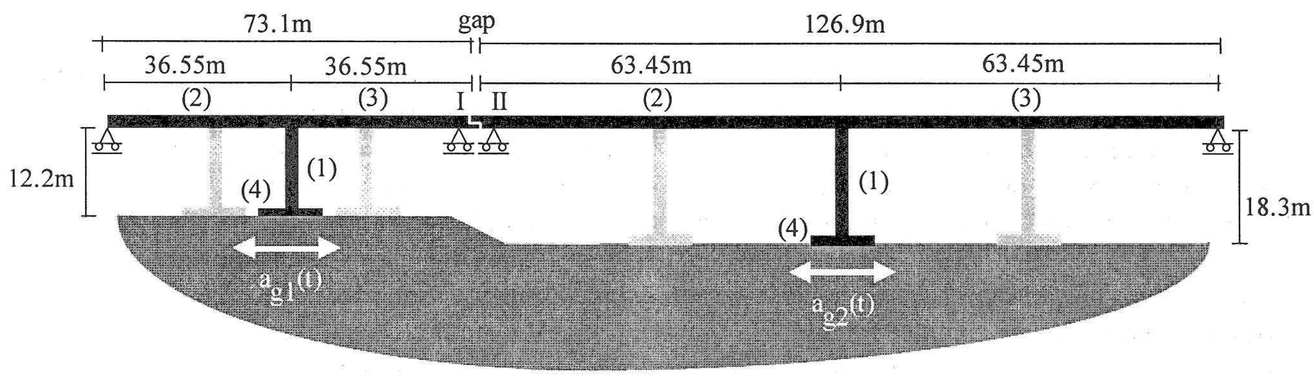

Fig. 1. Multiple-pier bridge frames and their idealization

Table 1. Material data of the idealized bridge

Bridge frame	Left				Right			
Member number	Mass [t/m]	EA [10^8 kN]	EI 10^8 kNm^2]	Length [m]	Mass [t/m]	EA [10^8 kN]	EI 10^8 kNm^2]	Length [m]
(1)	5.26	1.407	1.546	12.2	7.89	2.111	2.32	18.3
(2)	75.5	63.42	50.49	36.55	108.75	63.42	50.49	63.45
(3)	75.5	63.42	50.49	36.55	108.75	63.42	50.49	63.45
(4)	91.5	76.86	102.48	9.00	91.5	76.86	102.48	9.00

In numerical analysis the bridge frames with their foundations and the subsoil are described by finite elements and boundary elements, respectively. In order to determine the equation of motion of one of the bridge structures with subsoil in the Laplace domain we first transform the governing equation of the bridge structures

$$[\mathbf{M}]\{\ddot{\mathbf{u}}^b\}+[\mathbf{C}]\{\dot{\mathbf{u}}^b\}+[\mathbf{K}]\{\mathbf{u}^b\}=\{\mathbf{P}^b\} \tag{1}$$

into the Laplace domain

$$[\tilde{\mathbf{K}}^b]\{\tilde{\mathbf{u}}^b\}=\{\tilde{\mathbf{P}}^b\} \tag{2}$$

where $\tilde{\mathbf{K}}^b=(\mathbf{M}\,s^2+\mathbf{C}\,s+\mathbf{K})$ and $s = (\delta + i\ \omega)$ is the Laplace parameter and $i=\sqrt{-1}$. $\tilde{\mathbf{u}}^b$ is the response of the bridge structure in the Laplace domain. The tilde indicates a vector or matrix in the Laplace domain. In a common finite element formulation with lumped mass model or consistent mass model, **M**, **C** and **K** are respectively the mass, damping and stiffness matrix of the considered bridge structure. $\tilde{\mathbf{K}}^b$ is the dynamic stiffness of the bridge structure, which depends on the frequency ω. In this study we use the exact solution of the equation of motion in the axial and lateral direction of each structural members, the mass, damping and stiffness matrices are therefore no longer separated. This formulation is also called continuous-mass model. Details of the derivation of the element stiffness are given by Chouw[21]. The dynamic stiffness $\tilde{\mathbf{K}}^s$ of the subsoil is obtained using a full-space fundamental solution. The equation of motion of the bridge structure with subsoil in the Laplace domain is

$$\begin{bmatrix}\tilde{\mathbf{K}}^{bn}_{bb} & \tilde{\mathbf{K}}^{bn}_{bc}\\ \tilde{\mathbf{K}}^{bn}_{cb} & \tilde{\mathbf{K}}^{bn}_{cc}+\tilde{\mathbf{K}}^{sn}_{cc}\end{bmatrix}\begin{Bmatrix}\tilde{\mathbf{u}}^{bn}_b\\ \tilde{\mathbf{u}}^{bn}_c\end{Bmatrix}=\begin{Bmatrix}\tilde{\mathbf{P}}^{bn}_b\\ \tilde{\mathbf{P}}^{bn}_c\end{Bmatrix} \tag{3}$$

The indices b, s and c stand for bridge, subsoil and the contact-degree-of-freedom (CDOF) at the interface between the bridge foundation and the ground, respectively. n = I and II stand respectively for the left and right bridge structure with foundation and subsoil. With Eq. (3) and the transformed load $\tilde{\mathbf{P}}$ the linear response $\tilde{\mathbf{u}}$ of both bridge structures can be calculated. A transformation into the time domain gives the time history of the response **u**(t) of the bridge structures. In order to determine the time when the two girders collide, we calculate the closing relative displacements between the two girders at the locations I and II (see Fig. 1). If the relative displacement is larger than the existing gap size, pounding takes place. To incorporate the pounding effect into the linear response, we define the unbalanced forces using the relative displacement values and the girder stiffness at the pounding location. The response of the bridge structures to this unbalanced load is calculated in the Laplace domain using the dynamic stiffness of the whole system, since both bridge structures are now in contact. After transforming the obtained result into the time domain, the linear results can be corrected from the time step when pounding occurs. An examination of the corrected response reveals the instant when the girders separate. This is the case when the contact forces are larger than zero. Since tension cannot be transmitted between the girders, a further correction of the result is necessary. The unbalanced forces are now the contact forces acting in the opposite direction. After transforming this unbalanced load into the Laplace domain the corrective result can be calculated using Eq. (3). With this result the previous response can be corrected. We examine the result again for further poundings. In this numerical approach the nonlinear contact behaviour due to poundings and separations are described by subsequent piecewise linear behaviour, and the correction takes place in the time domain, while the result needed for the correction is calculated in the Laplace domain. Thus the analyses are performed alternately in the Laplace and time domain. Details of the non-linear procedure is given in the previous works[15), 22)].

2. 2 Non-uniform ground excitation

In this study the characteristic of the Japanese design spectra[23)] is considered. During the 1995 Kobe earthquake many strong ground motions were recorded in the near-source regions. The design spectra in Fig. 2 were defined in 1996 by simply enveloping the response spectrum values of the recorded strong ground motions. Three soil conditions are specified. For a soft soil site the dominant frequencies range from 0.67 Hz to 2 Hz. In the case of medium and hard soil they range from 0.83 Hz to 2.5 Hz, and from 1.42 Hz to 3.34 Hz, respectively.

Earthquake ground motions at any two locations on ground surface will not be the same owing to the seismic wave propagation[24)]. Much effort has been spent on study of spatial variations of seismic ground motions because such variation affects structural responses, especially large dimension structures such as bridges. Theoretical derivation of ground motion coherency loss is not straightforward. Usually, empirical coherency loss functions are derived from the recorded motions in dense seismograph arrays such as the SMART-1 array[25, 26, 27, 28)]. For example, an empirical coherency loss function[26)] was derived as

$$\left|\gamma(f,d^l_{ij},d^t_{ij})\right|=\exp(-\beta_1 d^l_{ij}-\beta_2 d^t_{ij})\\ \exp\{-[\alpha_1(f)\sqrt{d^l_{ij}}+\alpha_2(f)\sqrt{d^t_{ij}}]f^2\} \tag{4}$$

where d^l_{ij} and d^t_{ij} are projected distances in meters between locations i and j on ground surface in the wave propagation direction and its perpendicular direction, respectively; β_1 and β_2 are two constants, and $\alpha_1(f)$ and $\alpha_2(f)$ are two function defined as

$$\alpha_i(f) = \frac{a_i}{f} + b_i f + c_i, i = 1,2. \qquad (5)$$

The parameters β_1, β_2, and a_i, b_i, and c_i govern the coherency loss or cross correlation between ground motions at points i and j on ground surface. In this study, ground motion spatial variation model derived from data recorded at the SMART-1 array is employed to model ground motion spatial variations. In theory, the simulation can be done only provided the site under consideration has very similar geological and tectonic conditions. In our study, however, no particular site is investigated. Our intention is to demonstrate the importance of the effects of ground motion spatial variations on bridge girder pounding responses. Therefore the above empirical coherency loss function is employed, and the spatial ground motions are assumed to be intermediately cross-correlated in the analysis.

The parameters used are $\beta_1 = 3.7\ 10^{-4}$, $\beta_2 = 2.24\ 10^{-4}$, $a_1 = 1.19\ 10^{-2}$, $b_1 = -1.811\ 10^{-5}$, $c_1 = 1.177\ 10^{-4}$, $a_2 = 1.721\ 10^{-2}$, $b_2 = -7.583\ 10^{-6}$, $c_2 = -1.905\ 10^{-4}$, which are modified from the recorded motion at the SMART-1 array during Event 45[26].

Besides loss of coherency, seismic wave propagation also causes a phase shift between ground motions at spatial locations i and j. Previous studies revealed that spatial ground motion phase shift effect depends on a dimensionless parameter $f\,d/c_a$, where f is the structural vibration frequency, d is the separation distance between two structural supports i and j; and c_a is ground motion apparent wave propagation velocity[27, 28, 29, 30, 31]. It was found that in general c_a is also a function of frequency[25, 26]. A previous study found that relation between c_a and the frequency is rather random, and no conclusion could be drawn[26]. For this reason, in almost all the previous studies of ground motion spatial variation effects on structures, c_a was assumed as a constant. It depends on the wave propagation velocity of the site under consideration and the wave incident angle into the site. c_a equals to c_s if seismic wave propagates into the site horizontally and consists of mainly S-wave. Otherwise $c_a = c_s/\cos q$ and q is incident angle of the seismic wave into the site. It is infinity when seismic wave propagates vertically into the site. In this study, without losing generality, c_a is assumed to be 200 m/s, 500 m/s and 1000 m/s. For the three sites, although c_s is different, c_a could be the same because of different incident angle.

Using the above apparent velocity and empirical coherency loss function, spatially varying ground motion time histories are stochastically simulated and used as multiple inputs at the structural supports. The motions are simulated with a duration of 20.48 s and a time increment of dt = 0.01 s. Each simulated ground motion time history is compatible with the target design spectrum, which is the Japanese design spectrum for either soft, medium or hard soil condition[23]. Any two of the simulated ground motion time histories are compatible with the above empirical coherency function. For each case, three independent simulations are carried out to get three sets of stochastically independent spatially correlated ground motions. Three independent structural responses to the three sets of ground motions are calculated and ensemble mean structural responses are derived. To limit the influence factors in the investigation it is assumed that the bridge frames remain linear during the poundings. In order to justify this assumption only 50 % of the earthquake load is used in the analysis. Figs. 3, 4 and 5 shows respectively one set of the simulated ground accelerations corresponding to the Japanese design spectrum for the soft, medium and hard soil condition with the wave apparent velocity of 200 m/s, 500 m/s and 1000 m/s. Some of the corresponding ground displacements $u_g(t)$ are shown in Fig. 10. As shown, because the spectrum for hard soil condition has a relatively higher frequency content, although it associates with larger peak ground acceleration (PGA), the peak ground displacement (PGD) is smaller than the motions simulated according to the spectrum for soft soil condition, in which it has smaller PGA but larger PGD. Fig. 2 compares the response spectra of the simulated ground motions and the target design spectrum for the three soil conditions.

Comparison between the coherency loss of the simulated motions and the empirical coherency loss function is given in Fig. 6. As shown, the simulated motions are compatible with the Japanese design spectrum and the empirical coherency loss function. More detailed descriptions on ground motion spatial variations and simulation can be found in references[26, 27].

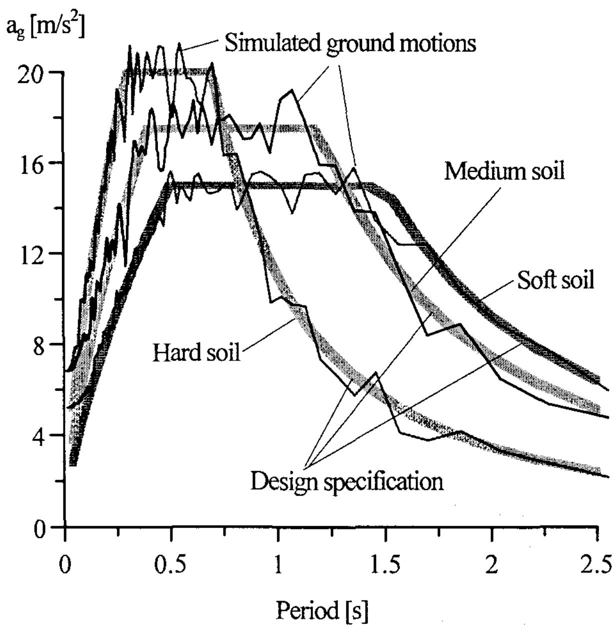

Fig. 2. Japanese design spectra and response spectra of the simulated ground motions

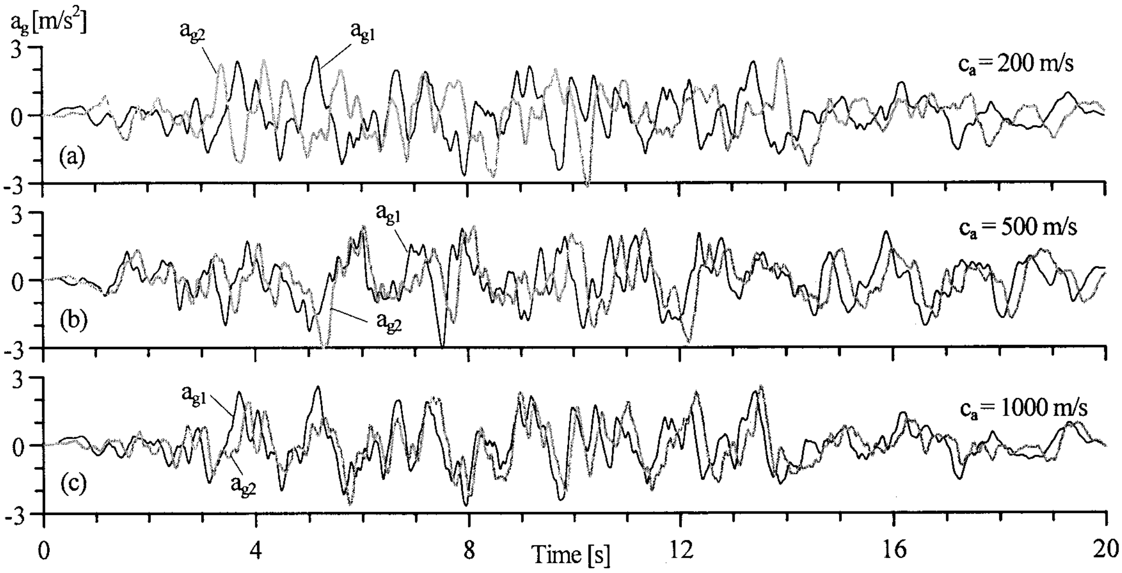

Fig. 3(a)-(c). Simulated ground accelerations with c_a = 200 m/s, 500 m/s and 1000 m/s for soft soil condition

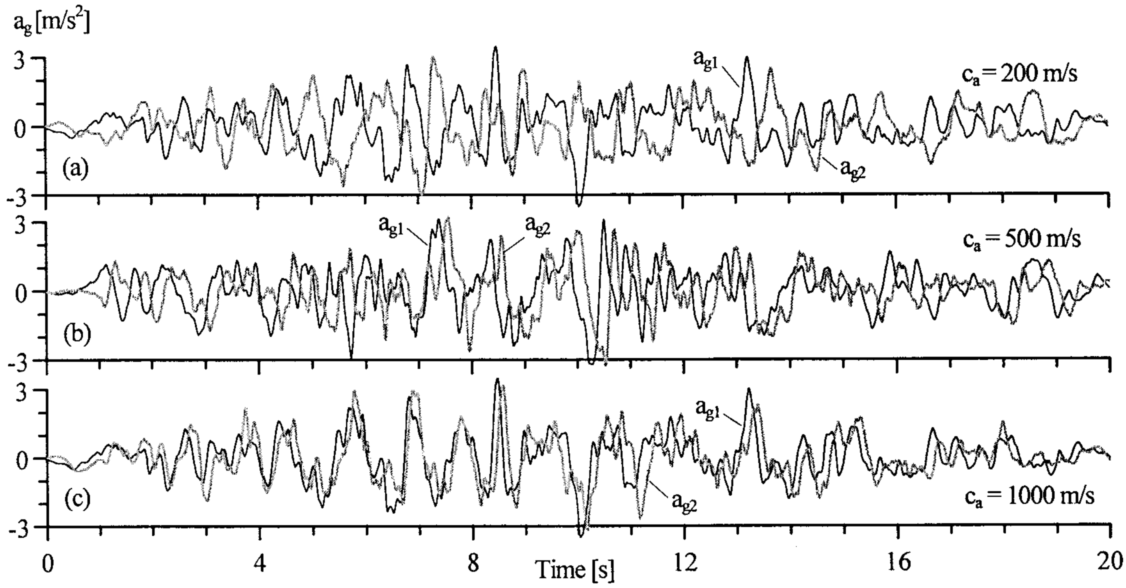

Fig. 4(a)-(c). Simulated ground accelerations with c_a = 200 m/s, 500 m/s and 1000 m/s for medium soil condition

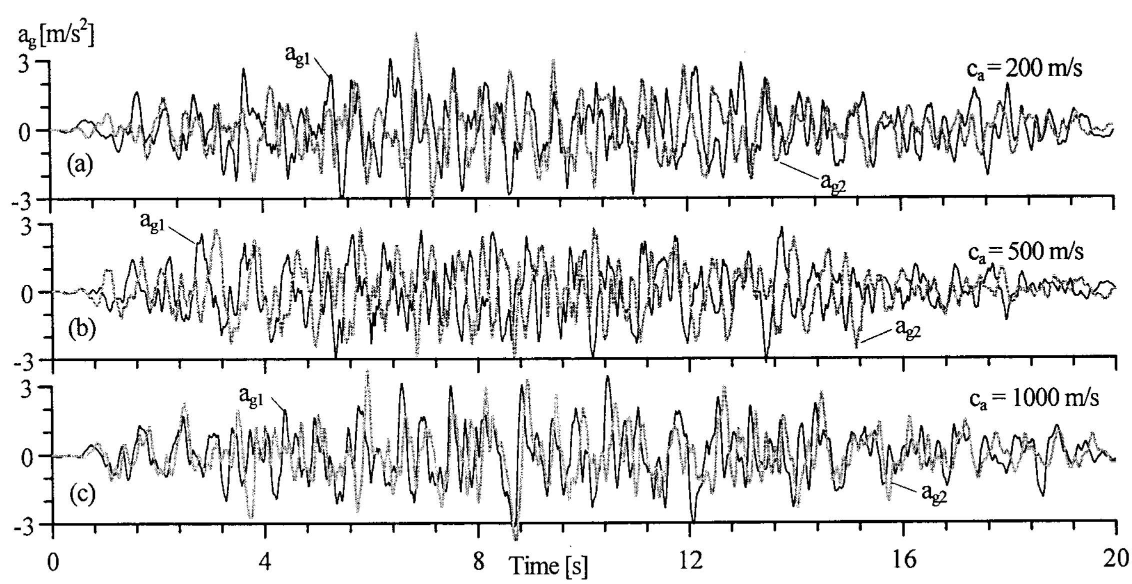

Fig. 5(a)-(c). Simulated ground accelerations with c_a = 200 m/s, 500 m/s and 1000 m/s for hard soil condition

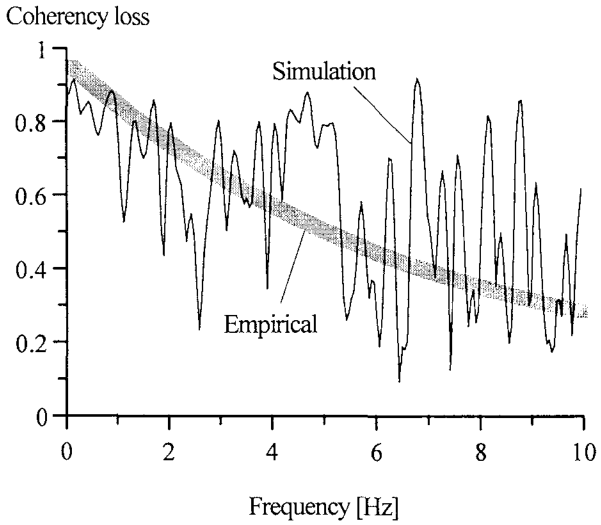

Fig. 6. Coherency loss function

3. Numerical results

3. 1 Influence of the frequency content of non-uniform ground motions and SSI on girder responses

In the case of a ground excitation of neighbouring buildings uniform ground motions are often assumed, since the buildings are very close to each other. Generally, the effect of the ground displacements on the building response can be neglected. Only the relative dynamic response is considered in the analysis.

In order to evaluate the recommendation of many current design regulations the flexural stiffness EI of right bridge frame is increased, so that both bridge frames have almost the same fundamental frequency. Fig. 7(a) and (b) show the relative dynamic response of the left and right girder u_I (grey line) and u_{II} (black dotted line) to the assumed uniform ground accelerations $a_{g1}(t)$ of the soft soil site with the wave apparent velocity c_a of 200 m/s (Fig. 3(a)) with and without SSI, respectively. The locations I and II are indicated in Fig. 1. The influence of the soft subsoil can be clearly seen in the longer period and larger amplitude of the responses. The distance between the responses u_I and u_{II} at the time 0 s represent the assumed gap between the girders to avoid pounding. The subsoil clearly causes larger required gap of 0.435 m, while in the case of an assumed fixed base the necessary gap is only 0.09 m. If the gap size is smaller than the assumed gap, without SSI pounding will take place at 11.2 s, and with SSI it will occur much earlier at 6.36 s.

In Fig. 8(a) and (b) the quasi-static response is included. It is the response of the bridge structures to the ground displacements $u_g(t)$. The total response becomes much larger than the relative dynamic response. In order to present the relative displacement between the two girders, the right girder response u_{II} is also plotted unshifted (black thin line). It can be clearly seen, especially in the fixed-base case, both girders respond in phase. In the case with subsoil both girders vibrate in phase globally. Even though the total response is much larger, the pounding potential is still determined by the out-of-phase relative dynamic responses. With SSI the relative response, which determines the required gap to avoid poundings, is clearly much larger.

The result shows that the design recommendation to mitigate the pounding effect by adjusting the fundamental frequencies is indeed correct; if uniform ground motions are considered. In the case of non-uniform ground motions the relative displacements are no longer determined by the relative dynamic responses, as Fig. 9(a) and (b) show. They are also determined by the non-uniform ground motions. A neglect of the characteristic of non-uniform ground motions can clearly underestimate the pounding potential between the bridge girders, and consequently the damage potential of the bridge. The effect of the quasi-static response will become more pronounced, when the ground motions have low dominant frequencies as the case with the soft and medium soil condition (see also Fig. 2).

The influence of the frequency content of the ground motions on the amplitude of the ground displacements $u_g(t)$ can be seen in Fig. 10. The lower the dominant frequencies of the ground motions are the larger the ground displacements $u_g(t)$ will be. The corresponding ground accelerations $a_g(t)$ are displayed in Fig. 3(a), 4(a) and 5(a). Since the soft and medium soil condition ground motions have similar frequency content in the lower frequency range below 0.8 Hz, both soil conditions have similar large ground displacements, much larger than the ones of the hard soil condition (compared Fig. 10(a) and (b) with Fig. 10(c)).

Suppose the two bridge frames do not experience ground accelerations, and the ground motions only consist of ground displacements. Fig. 10 displays the displacement of the corresponding bridge frame. $u_{g1}(t)$ (bold grey line) and $u_{g2}(t)$ (thin black line) are therefore equal to u_I and u_{II}, respectively. In order to display the required gap to avoid pounding induced by the ground displacements, the displacement $u_{g2}(t)$ is shifted corresponding to the required gap.

If uniform ground displacements are assumed, $u_{g2}(t)$ is then equal to $u_{g1}(t)$. In order to avoid confusion it is indicated by $u_{g2,\,uni}(t) = u_{g1}(t)$ (dotted grey line). It can then be clearly seen, no matter how large the uniform ground displacements are, they will not contribute to pounding, because both girders will move in phase. Non-uniform ground displacements can in contrast cause poundings, as Fig. 10(a) and (b) show in the out-of-phase vibrations (the grey line and the black line). The induced relative displacements between the girders are much larger than the ones caused by the ground displacements with the hard soil condition (Fig. 10(c)). In the case of the assumed gap size of 0.35 m the large ground displacements of the soft soil site will cause poundings. No pounding will take place for the medium soil condition. In the case of the hard soil condition the existing gap is clearly more than enough to avoid ground displacement induced poundings.

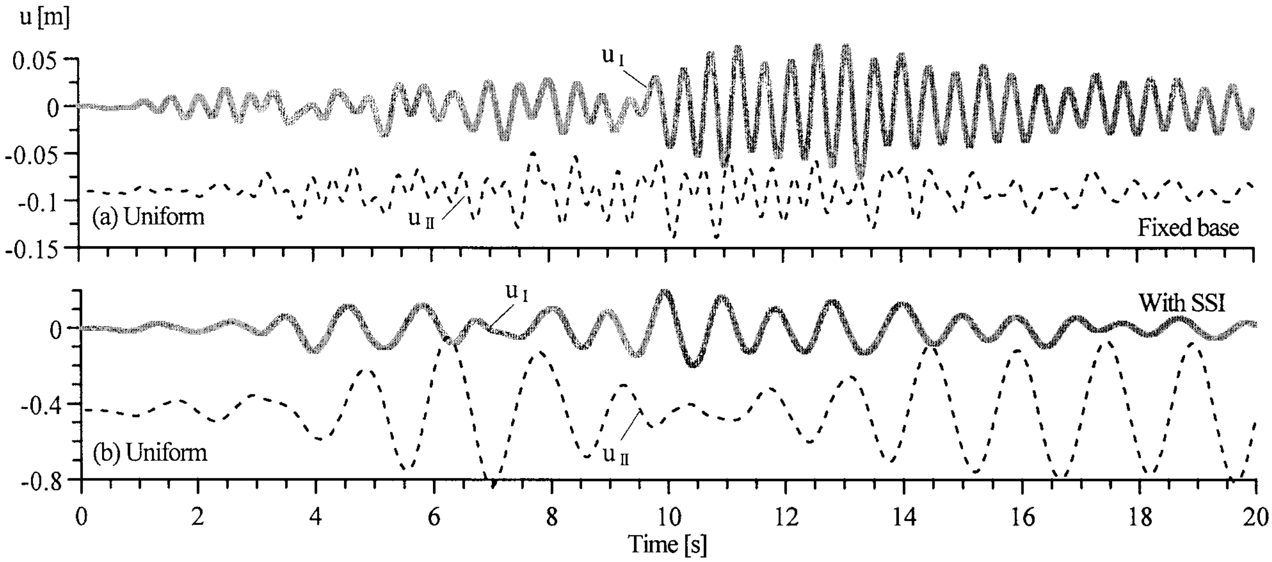

Fig. 7(a) and (b). Influence of uniform ground motions $a_{g1}(t)$ and soil-structure interaction on girder reponses.

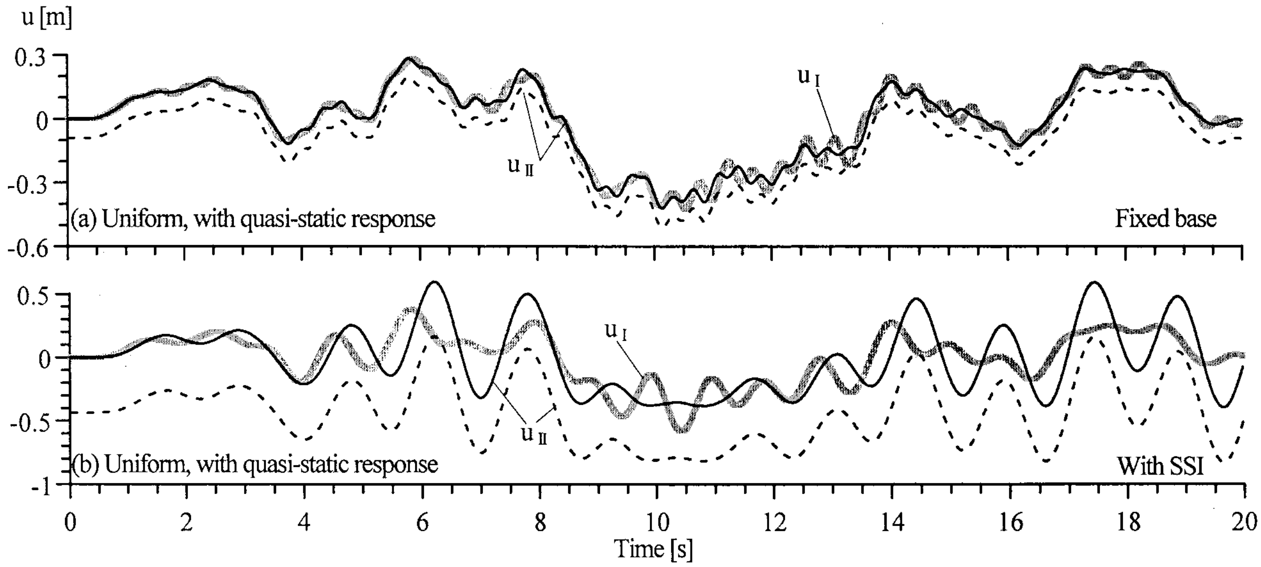

Fig. 8(a) and (b). Influence of uniform ground motions $a_{g1}(t)$, $u_{g1}(t)$ and soil-structure interaction on girder

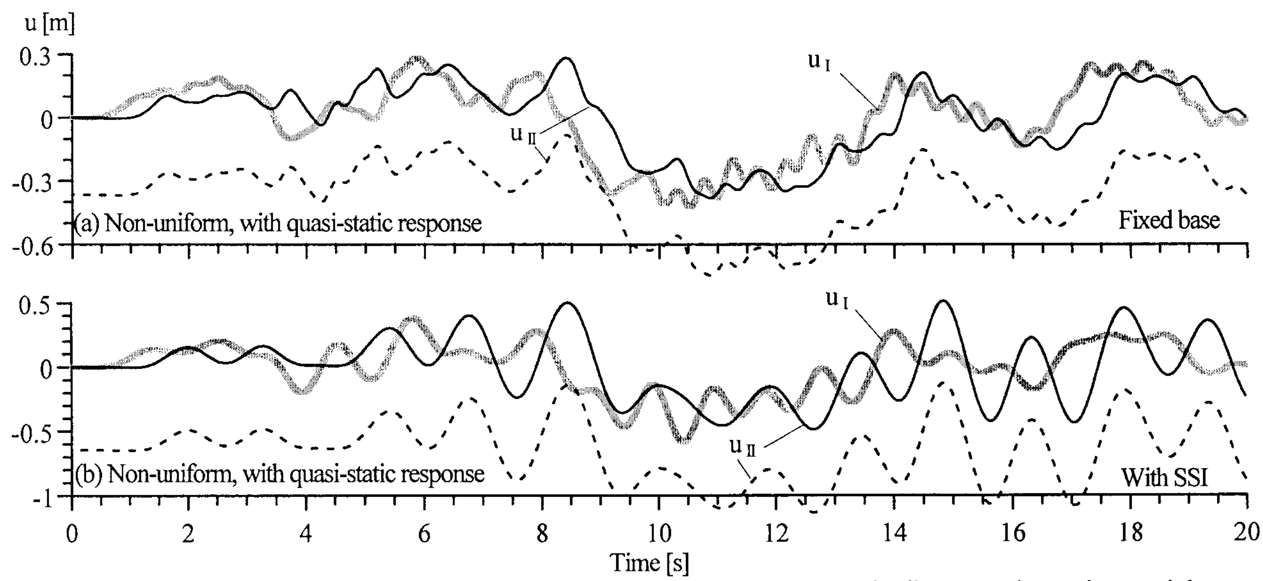

Fig. 9(a) and (b). Effect of non-uniform ground motions a_{g1}, a_{g2}, u_{g1}, u_{g2} and soil-structure interaction on girder

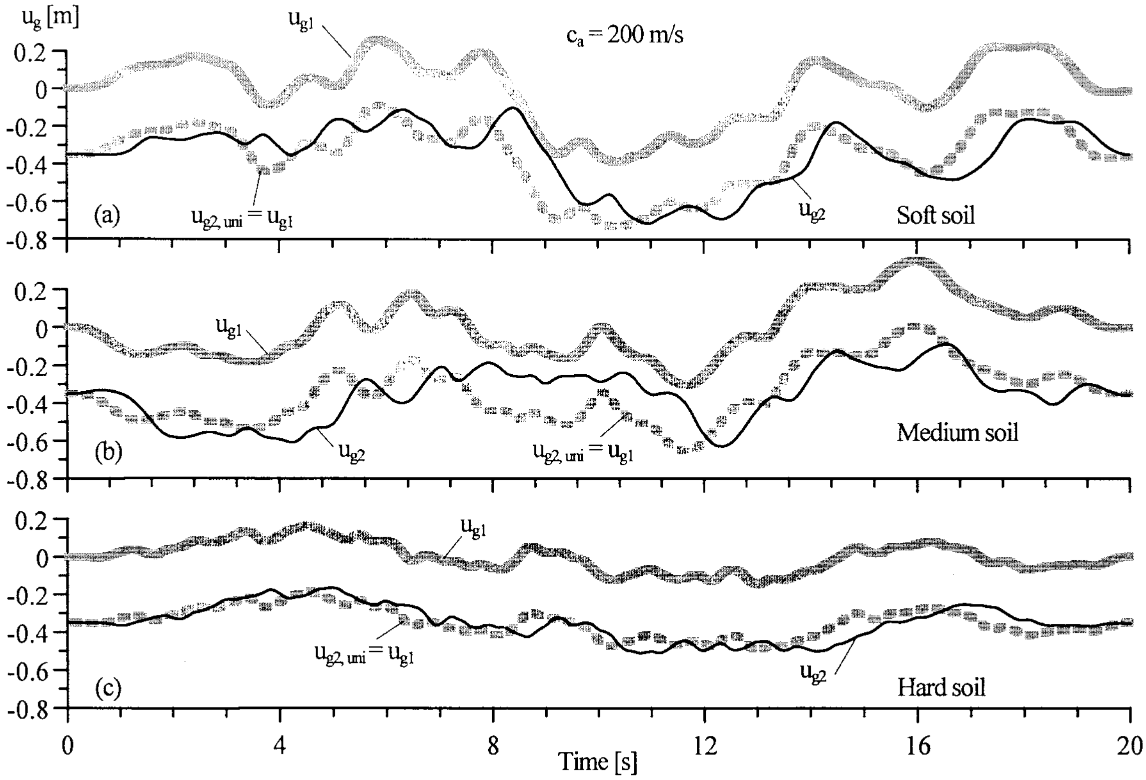

Fig. 10(a)-(c). Influence of the non-uniform ground displacements $u_{g1}(t)$ and $u_{g2}(t)$ on the pounding potential of the bridge girders. (a) Soft soil, (b) medium soil, and (c) hard soil condition

3. 2 Influence of the frequency content of non-uniform ground motions on the required gap

Fig. 11 shows the required gap to avoid poundings with respect to the site conditions. The SSI effect is neglected. The ratio of the fundamental frequencies f_{II} / f_I is 0.42. Thus, in the case of uniform ground motions the required gap d is mainly determined by the difference between the dynamic properties of both bridge frames. In the case of non-uniform ground motions the influences come mainly from the characteristics of the non-uniform ground motions. The results show that an assumption of uniform ground excitations will underestimate the required gap to avoid poundings.In all cases of non-uniform ground motions larger gap size is needed. In both uniform and non-uniform cases ground motions for hard soil condition causes smaller required gap even though their amplitudes are larger than those of the soft and medium soil condition (see Fig. 3, 4, and 5). As expected, in the case of non-uniform ground motions the lower the wave apparent velocity c_a is the larger the required gap will be.

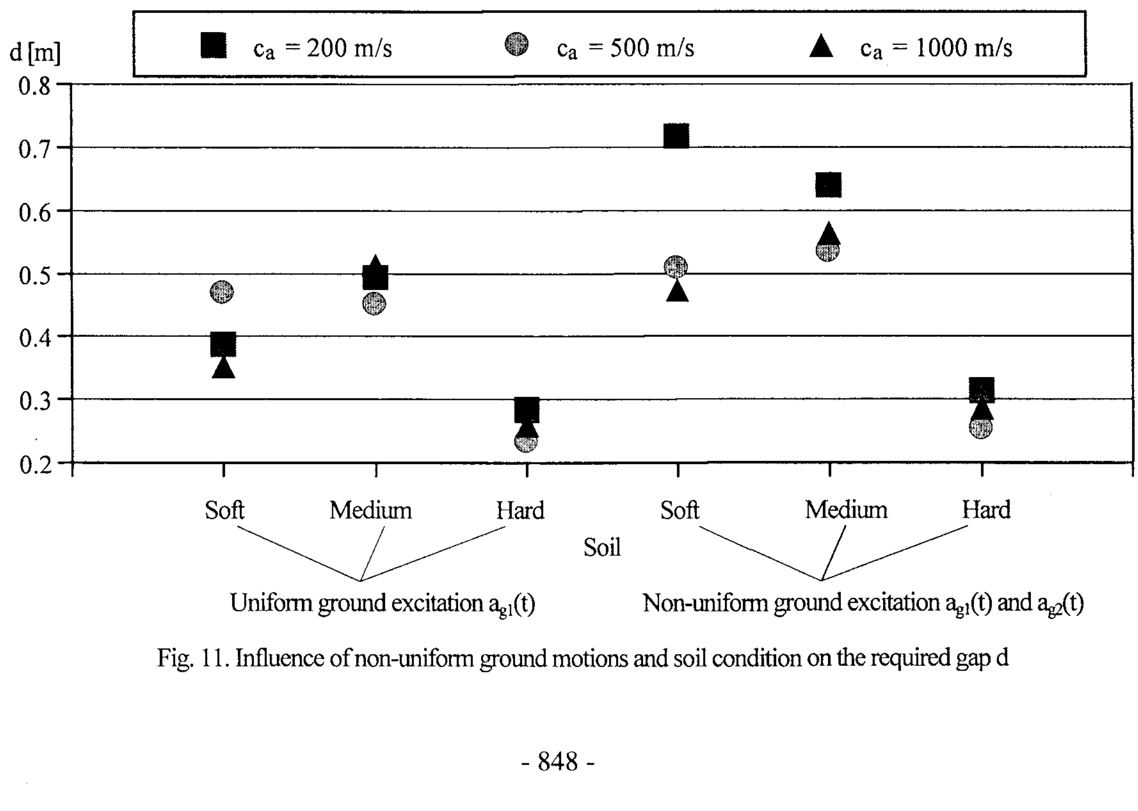

Fig. 11. Influence of non-uniform ground motions and soil condition on the required gap d

3. 3 Influence of the non-uniform quasi-static response on girder pounding responses

Fig. 12(a) and (b) show the response of both girders without and with a consideration of the quasi-static response, respectively. The non-uniform ground motions are the ones of the soft soil condition with the wave apparent velocity c_a of 200 m/s and the frequency ratio f_{II} / f_I is about 1. The soil-structure interaction is taken into account. The dotted lines are the responses without pounding effect. The corresponding linear responses are given in Fig. 7(b) and 9(b). It is assumed that the existing gap for the case without and with a consideration of the quasi-static response is 0.43 m and 0.64 m, respectively. If only the relative dynamic responses (the case without quasi-static response) are considered, the right girder with larger amplitude pushes the left girder away. The left girder experiences a slight damping due to the pounding. Pounding does not cause significant change in both girder responses. In contrast the non-uniform quasi-static response strongly affects the response of both girders after the pounding. The result shows that a neglect of the effect of the non-uniform ground displacements will underestimate the pounding potential and also the unseating of the bridge girders.

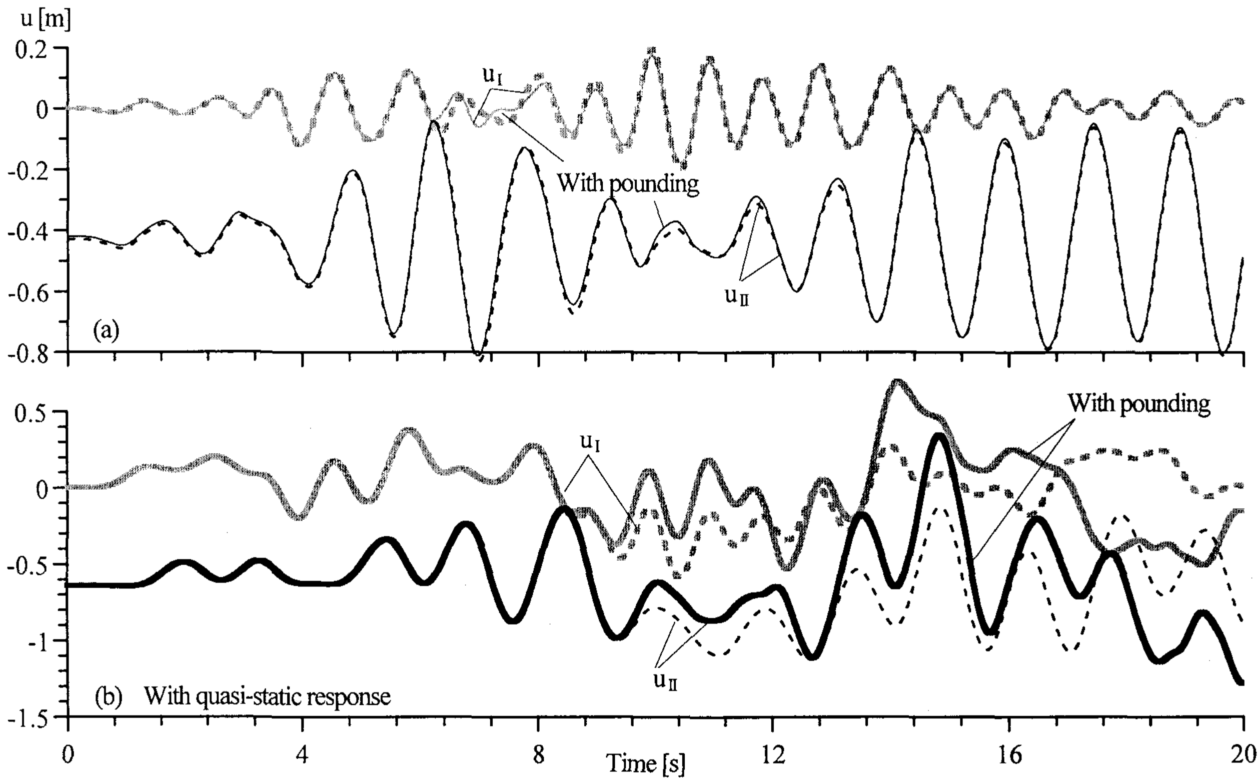

Fig. 12(a) and (b). Influence of quasi-static response on girder pounding behaviour. (a) Without and (b) with quasi-static response

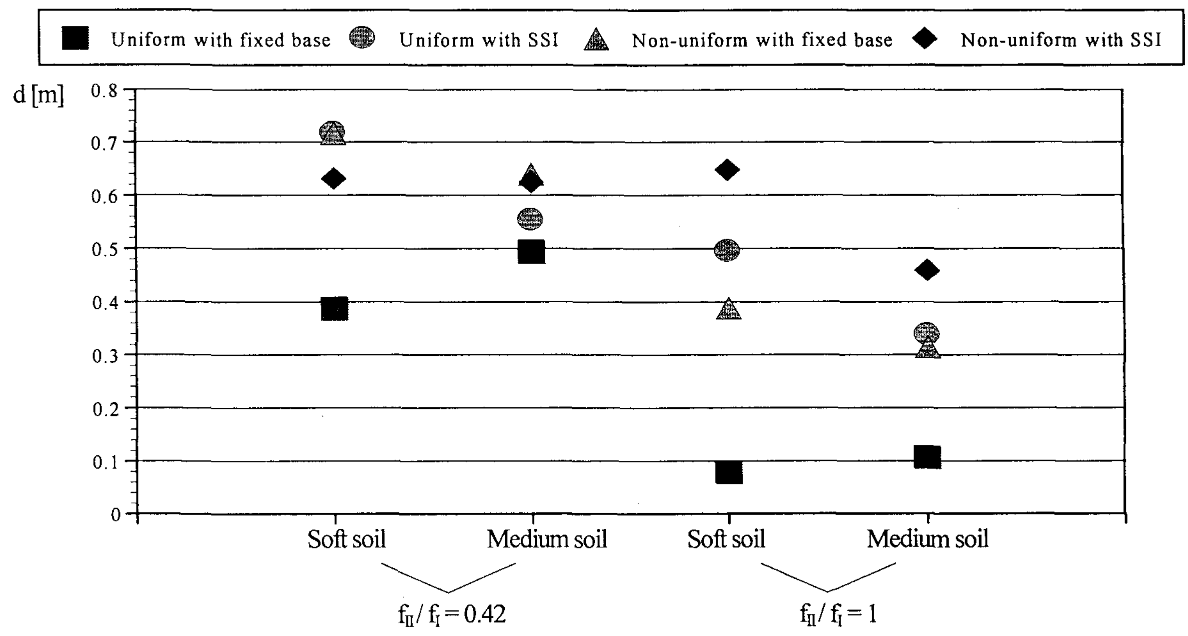

Fig. 13. Relationship between the frequency ratio of the bridge frames and the ground motion characteristics

3. 4 Influence of the frequency ratio f_{II} / f_I on the required gap

Fig. 13 shows the influence of the non-uniform ground motions, the soil-structure interaction and the frequency ratio f_{II}/ f_I on the required gap. In the case of uniform ground motions the fixed-base bridge frames with almost same fundamental frequency will vibrate in phase as shown in Fig. 8(a), consequently only small gap is needed. This confirms to the recommendation of many design regulations. A strong reduction of the required gap still can be observed if the SSI effect is included. However, SSI clearly causes larger required gap. A consideration of non-uniform ground excitations will causes larger required gap. If the additional SSI effect is included even larger gap is needed. The result shows that the recommendation to equalize the fundamental frequency of the adjacent structures is only beneficial when the uniform ground excitations are expected and the ground is rigid.

4. Conclusions

In the numerical analysis the pounding response of two adjacent bridge girders to the simulated non-uniform ground motions corresponding to the Japanese design specification for soft, medium and hard soil condition is considered. The wave apparent velocity c_a is assumed as either 200 m/s, 500 m/s or 1000 m/s. For the soft and medium soil condition the subsoil is assumed to be a half-space with the shear wave velocity c_s of 100 m/s and 200 m/s. For the hard soil condition rigid subsoil is considered. In order to obtain more reliable conclusions three sets of ground motions are simulated for each considered case.

The investigation reveals:

Non-uniform ground motions might result in bridge girder pounding responses which are very different from the responses due to uniform ground motions.

The large non-uniform ground displacements can strongly contribute to the relative displacements between the girders. A neglect of the quasi-static responses can therefore significantly underestimate the required gap to avoid pounding, especially when the ground excitations have low dominant frequencies where large non-uniform ground displacements can be expected. This obtained knowledge is important for bridges in near-source regions where the long-period pulses in the ground motions can be expected.

The recommendation for adjusting the fundamental frequencies of the adjacent bridge structures works, only when the occurrence of uniform ground motions can be ensured and the ground is very stiff. In the case of soft subsoil and non-uniform ground motions this design recommendation alone is insufficient to mitigate poundings, because the relative displacements of the adjacent structures with similar fundamental frequency as well as the pounding responses of the girders will strongly be determined by the non-uniform ground motions.

Even though the ground motions with high frequency content have larger PGA, the ground motions with low dominant frequencies might govern the girder pounding potential because of large quasi-static response. If the considered soil is soft and the non-uniform ground excitations have low dominant frequencies, SSI effect and spatial variation of the ground motions should be considered, since both factors can significantly affect the pounding responses of the bridge girders.

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