Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 597

CONSEQUENCE OF NON-UNIFORM SITE FOR BRIDGE RESPONSE

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ABSTRACT

This work emphasizes the significance of spatial variation of ground motions, unequal soil-structure interaction (SSI) and the interrelation between seismic waves, bridge structural properties and non-uniform soil site. The spatially varying ground motions are simulated stochastically based on New Zealand design spectra. The bridge structures, footing and subsoil are described using a combined finite element and boundary element method. Amplification and reduction of activated forces in bridge structures due to different assumptions of the ground excitation and the soil-bridge structure system reflect the significance of the combined influence of structural, soil and seismic wave properties. Neglect of a realistic assumption can underestimate the damage potential of bridge structures in earthquakes.

Introduction

Damages to bridges have been observed in almost all major earthquakes in the past, e.g. Kobe earthquake in 1995 (Kawashima and Unjoh 1996), Chi-Chi earthquake in 1999 (EERI 1999), and Wenchuan earthquake in 2008 (Lin et al. 2008). Collapses occur when bridge decks lose their seat due to large opening relative movements at the supports. Pounding induced girder damages at the girder ends take place when closing relative movements are larger than the gap.

In past decades many researches have been done to understand the cause of girder relative movements. However, most investigations are performed under the assumption of uniform ground excitations and fixed base structures, e.g. (Ruangrassamee and Kawashima 2001). Research outcomes have also been incorporated in design regulations, e.g. (CALTRANS 2006). However, these outcomes are based on investigations with the previously mentioned assumption of uniform ground excitations and fixed base structures. Consequently, the relative responses are believed to be controlled mainly by dynamic properties of the participating bridge structures. It is, however, well known that seismic waves will arrive at distant bridge support locations at the different time and with some coherency loss. The ground excitations of adjacent bridge structures therefore cannot be the same. The Japanese specification (JRA 2004) is probably the only one that considers the influence of spatial variation of ground motions, although only empirically.

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To avoid bridge girder pounding recent researches (Bi et al. in print, Chouw and Hao 2008b) have suggested the usage of modular expansion joints that have the ability to accommodate large relative movement between adjacent bridge girders without causing any pounding.

To reveal the consequence of influence parameters uniform ground motions, time delay and spatial variation in ground excitations, unequal soil-structure interaction and non-uniform site effect are considered in this work.

Bridge-soil model and ground motions

Fig. 1 shows the considered adjacent bridge structures with different pier heights. The left and right girders have the length of 36.55 m and 63.45 m, respectively. The footing length is 9 m. For simplicity the multiple piers of each bridge structure are modeled numerically as a collective pier (dash line in Fig. 1). It is assumed that the distance between these collective piers is 50 m and the gap between bridge girders is 3 cm. To limit the number of influence parameters the bridge structures, their footings and subsoil should remain elastic. The material data is given in Table 1.

For the numerical analysis the bridge structures with their footings and the subsoil are described in the Laplace domain by finite elements and boundary elements, respectively. The fundamental frequencies of the left and right bridge structures with an assumed fixed base are 2.14 Hz and 0.9 Hz, respectively. The material damping of the bridge structures is described by a complex Young's modulus with the real and imaginary parts $E_1 = 0.1$ and $E_n = 10^{28}$, respectively (Hashimoto and Chouw 2003). The equivalent damping ratio is about 1.4 %.

Table 1. Material data.

Bridge	Left			Right		
Member	Mass (10^3 kg/m)	EA (10^8 kN)	EI (10^8 kN m^2)	Mass (10^3 kg/m)	EA (10^8 kN)	EI (10^8 kN m^2)
Girder	151	63.42	50.49	217.5	63.42	50.49
Pier	5.26	1.407	1.546	7.89	2.111	2.32
Footing	91.5	768.6	1024.8	91.5	768.6	1024.8

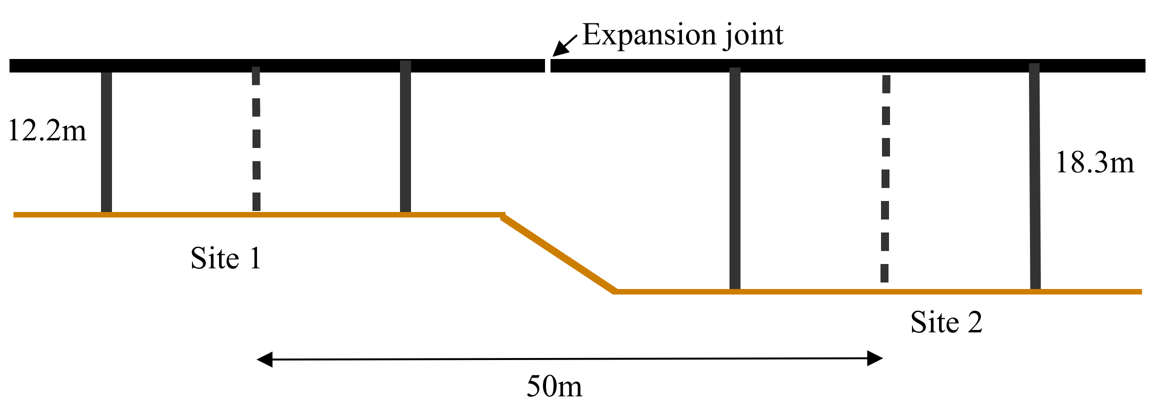

Figure1. Bridge structures with different sites.

The ground is assumed to be a half space. It is assumed that the soil density is 2 kNs^2/m^4, the Poisson's ratio ν is 0.33 and the shear wave velocities are respectively 200 m/s and 400 m/s for medium and hard soil sites which correspond to subsoil classes C and A according to New Zealand standard (NZS1170.5 2004). To limit the number of considered parameters it is assumed that the soil has no material damping. Only radiation damping due to propagating waves is considered.

After transforming the wave equation into the Laplace domain the dynamic soil stiffness $\tilde{\mathbf{K}}_{cc}^{sn}$ is defined. The structural members including footings are described by continuous-mass model (Kodama and Chouw 2002). The dynamic stiffness of each bridge structures is obtained by adding the stiffness of each structural member. The governing equations of the bridge structure with subsoil in the Laplace domain are

$$\begin{bmatrix} \tilde{\mathbf{K}}_{bb}^{bn} & \tilde{\mathbf{K}}_{bc}^{bn} \\ \tilde{\mathbf{K}}_{cb}^{bn} & \tilde{\mathbf{K}}_{cc}^{bn} + \tilde{\mathbf{K}}_{cc}^{sn} \end{bmatrix} \begin{bmatrix} \tilde{\mathbf{u}}_{b}^{bn} \\ \tilde{\mathbf{u}}_{c}^{bn} \end{bmatrix} = \begin{bmatrix} \mathbf{0} \\ \tilde{\mathbf{P}}_{c}^{n} \end{bmatrix} \tag{1}$$

where $\tilde{\mathbf{P}}_{c}^{n} = \tilde{\mathbf{K}}_{cc}^{sn}\, \tilde{\mathbf{u}}_{g}^{n}$. $\tilde{\mathbf{u}}_{g}^{n}$ is the ground motion at the footing-soil interface. *n*, *b*, *s*, *c* stand for the left or right bridge structure, bridge, soil and contact degree-of-freedom, respectively. A transformation of the results from the Laplace to the time domain gives the time history of the bridge responses.

The nonlinear behaviour of the soil-structure system is described by piecewise linear behaviour, i.e. contact or no contact condition. Pounding and separation of bridge girders and the resulting unbalanced forces for correcting the change from one behaviour to the following one are determined in the time domain, while the response is calculated in the Laplace domain. Details about the nonlinear algorithm for analysing the soil-structure system in Laplace and time domain are given in (Chouw 2002 and Chouw and Hao 2008a).

The spatially non-uniform ground motions are simulated stochastically based on New Zealand design spectra (NZS1170.5 2004) using the coherency loss function

$$\gamma_{ij} = \exp\left(-\beta\, d_{ij}\right)\, \exp\left(-\alpha\, d_{ij}^{1/2} f^{2}\right) \exp\left(-i\, 2\pi\, f\, d_{ij} / c_{a}\right) \tag{2}$$

where β is a constant, d_{ij} is the distance between the two locations *i* and *j* in the wave spreading direction, *f* is the frequency in Hz and c_a is the apparent wave velocity. α is a function in the following form

$$\alpha(f) = a / f + b\, f + c \qquad for\ f \le 10\,\mathrm{Hz} \tag{3}$$

When $f > 10$ Hz, the α function is a constant and equals to the value at 10 Hz. In the considered cases it is assumed that the ground motions are highly correlated and the apparent wave velocity c_a is 500 m/s. *a*, *b*, *c* and β are 3.583 10^{-3}, -1,811 10^{-5}, 1.177 10^{-4} and 1.109 10^{-4}, respectively. Details about the ground motion simulation procedure are given in (Hao 1989, Hao et al. 1989).

Fig. 2 shows the spatial variation of ground motions for two different site conditions: (1) uniform medium soil and (2) hard soil at site 1 and medium soil at site 2.

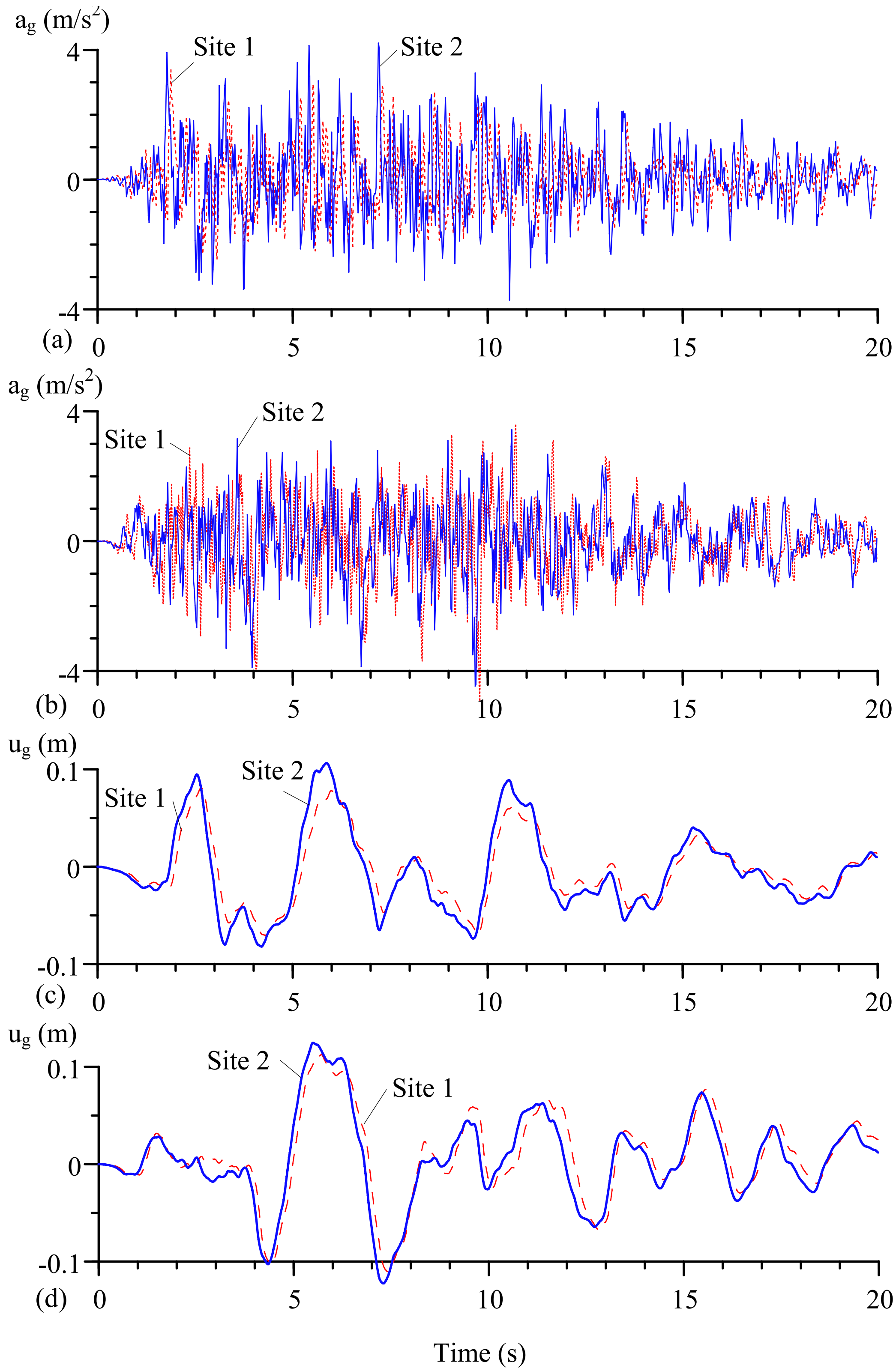

Figure 2. Site ground motions. (a) Hard and medium soil ground accelerations, (b) medium soil ground accelerations, (c) hard and medium soil ground displacements and (d) medium soil ground displacements.

Numerical results

Influence of load assumption and SSI

In structural analysis it is often assumed that structures are fixed at their base and adjacent structures experience the same ground excitation. In the case of a long extended structure, e.g. pipelines and bridges, this assumption can have significant consequence for activated forces in the structure. Because of wave propagation seismic waves will not be able to arrive at two adjacent bridge piers at the same time. Hence, adjacent bridge structures will have unequal ground excitations.

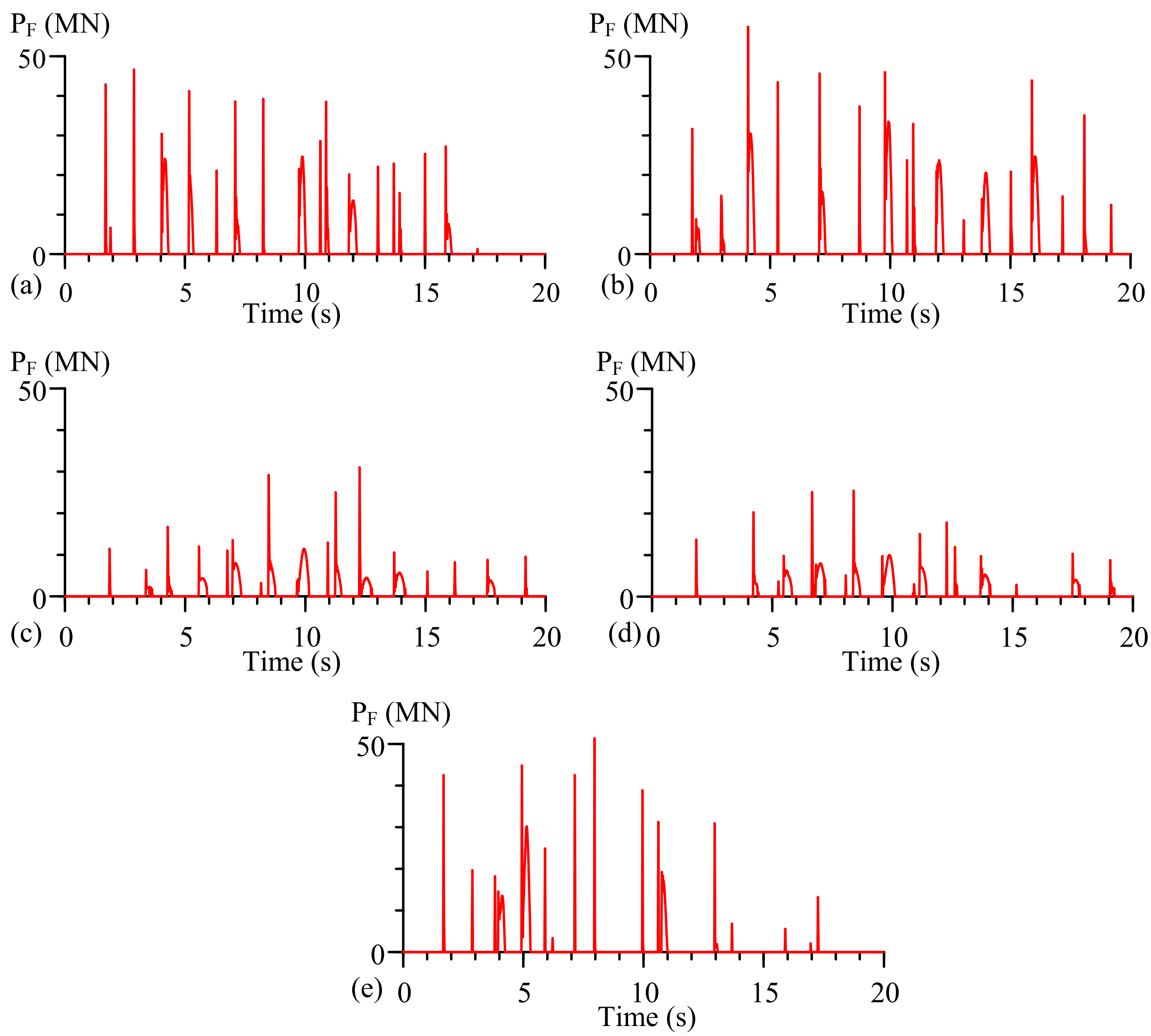

Figure 3. Pounding forces due to (a) uniform ground excitation, (b) uniform ground excitation with 0.1s time delay at site 2 without SSI effect, (c) uniform and (d) non-uniform ground excitation with SSI effect and (e) non-uniform ground excitation without SSI.

Fig. 3 shows the activated pounding forces due to medium soil ground motions. If uniform ground motions (site 1 ground motions in Figs. 2(b) and 2(d)) and fixed base bridge structures are assumed, contact forces P_F in Fig. 3(a) occur with the maximum force of 46.7 MN at 2.88 s.

To incorporate the influence of propagation of seismic waves, some researchers have included the effect of wave spreading as a time delay. In this considered case the apparent wave velocity is 500 m/s and the distance is 50 m. The ground motions at the right bridge pier will occur 0.1 s later than those at the left bridge pier. In Fig. 3(b) the pounding forces due to uniform ground motions with 0.1 s time delay are displayed. Although the number of poundings in the considered time window is the same and similar pounding development can be observed, the time delay causes a larger maximum pounding force of 57.6 MN at 4.08 s.

If spatially varying ground motions are considered, pounding development at the girder ends as displayed in Fig. 3(e) takes place. The bridge structures are assumed to be fixed at their base. The consequence of the spatial variation of the excitations can be clearly seen. Even though the maximum pounding force of 51.4 MN, which occurs at 7.96 s, is of same magnitude, smaller number of poundings can be observed.

Figs. 3(c) and (d) show respectively the pounding force developments due to uniform and non-uniform ground motions including SSI effect. The corresponding maximum contact forces are 31 MN and 25.5 MN, respectively, which occur at 12.26 s and 8.38 s. A comparison with the results in Figs. 3(a) and (e) shows that SSI clearly reduces the magnitude of pounding forces.

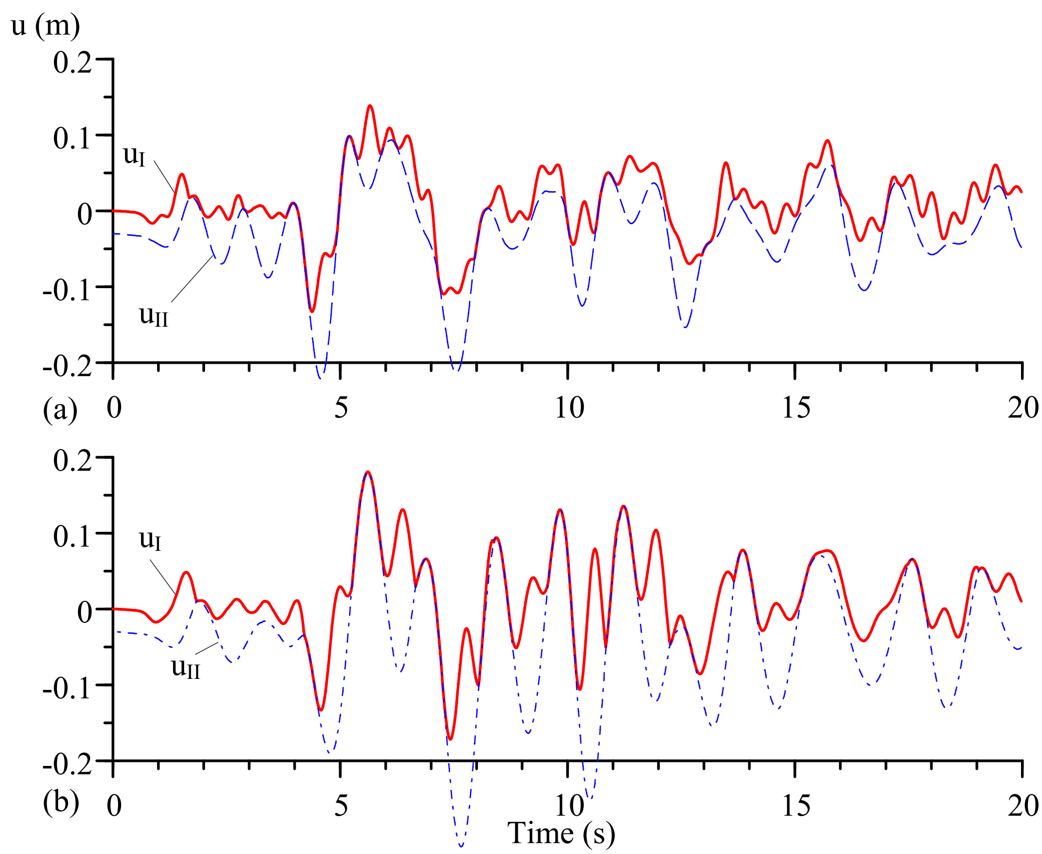

Figure 4. Structural displacements due the spatially varying medium soil ground excitation (a) without and (b) with SSI.

Damage potential due to pounding between adjacent bridge girders is determined not only by the maximum contact force but also by the number of strong poundings. The results show that in the considered cases the spatial variation of ground motions has a significant influence on the pounding development. Consequently, it is important for a proper estimation of pounding induced damage in earthquakes. A consideration of supporting ground has in the considered cases a beneficial effect on this pounding force development.

Figs. 4(a) and 4(b) show the displacement time histories u_I (solid line) and u_{II} (dash line) of the left and right girders without and with SSI effect, respectively. The influence of subsoil on the girder responses can be clearly seen in longer vibration periods. In contrast to the activated pounding forces P_F in Fig. 3 SSI causes larger amplitudes of the girder displacements.

In Fig. 5 the bending moment M_{II} developed at the right bridge pier support due to the spatially varying medium soil ground motions is compared. Pounding effect is considered. The dotted and solid lines are the bending moment without and with SSI effect, respectively. Larger bending moment occurs when SSI effect is considered. A consideration of subsoil causes more flexible structures, and an increase in girder movements activates larger bending moment at the support.

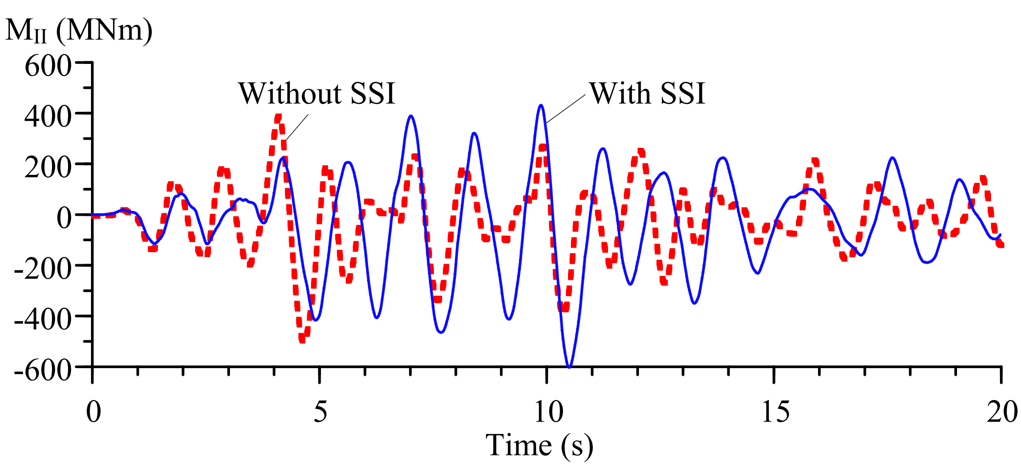

Figure 5. SSI effect on bending moment development at right pier support due to the medium soil ground excitations.

Influence of unequal SSI

In the previous investigations the fundamental frequency ratio f_{II}/f_I of the left and right bridge structures with an assumed fixed base is 0.42. To reveal the influence of SSI the bending stiffness of the girder and pier of the right bridge structure is increased so that the frequency ratio f_{II}/f_I is 0.99.

Fig. 6(a) displays the girder displacements u_I (solid line) and u_{II} (dotted line) of the left and right bridge structures with a fixed base assumption, respectively. Since both structures have almost the same fundamental frequencies, an assumption of uniform medium ground excitations cause no pounding. Both structures respond in phase. This is the main reason that current design regulations, e.g. (CALTRANS 2006), recommend that adjacent structures should have the same or similar fundamental frequencies. However, this recommendation will provide confidence of safety which does not exist. Fig. 6(b) shows the displacements u_I (solid line) and u_{II} (dashed line) of the left and right bridge girders due to the uniform medium soil ground motions without pounding effect. In contrast to the results in Fig. 6(a) SSI effect is considered. Even though both bridge structures have almost the same fixed-base fundamental frequencies (f_{II}/f_I = 0.99) and both structures experience the same ground excitation, it can be easily seen that pounding will already occur at 1.76 s with subsequent pounding possibilities. The reason is that different slenderness due to unequal height of the adjacent bridge piers causes different SSI. This unequal interaction leads to different vibration behaviour of the adjacent structures. Although both structures are excited by the same ground motions, relative movements between the adjacent bridge girders occur. They cause then poundings. Fig. 6(c) displays the actual pounding forces P_F in the considered time window which cannot be observed if SSI effect is neglected.

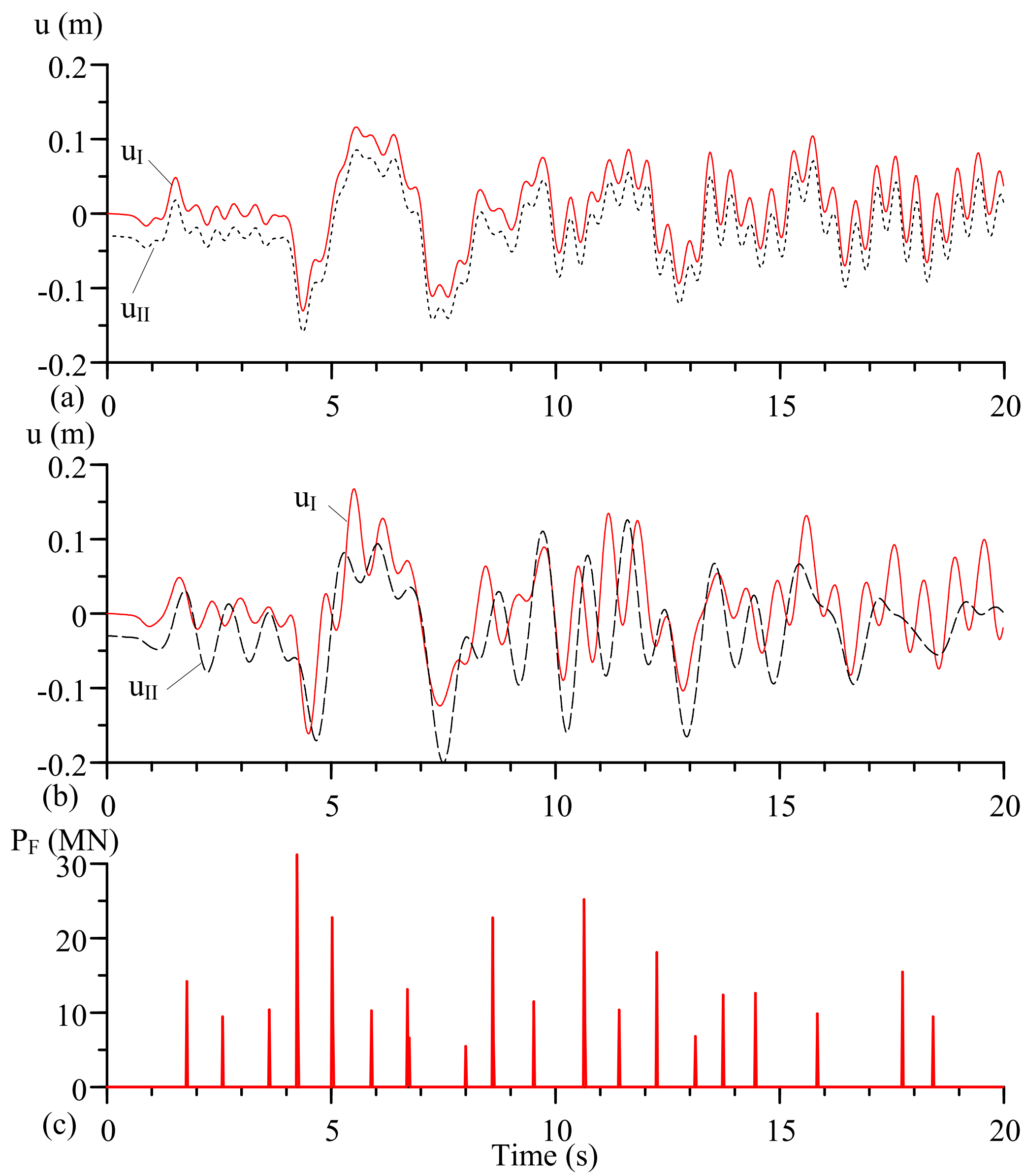

Figure 6. Structural displacements due to uniform medium soil ground excitation (a) without, (b) with SSI effect and (c) activated pounding forces with SSI.

Influence of non-uniform site

In previous sections medium soil is assumed for both sites. In this section the effect of non-uniform site is considered. It is assumed that the left site is hard soil, while the right site is medium soil.

Fig. 7 shows the development of relative displacement u_{rel} between the left and right bridge girders including SSI effect. The solid and dotted lines are u_{rel} of the case of non-uniform and uniform sites, respectively. The maximum u_{rel} in the case of non-uniform and uniform soil sites occur respectively at 3.74 s and 10.58 s with the corresponding values of 21.98 cm and 28.51 cm. Pounding occurs when u_{rel} exceeds the gap size of 3 cm. The results show that not only the maximum values are not the same, also the number of poundings.

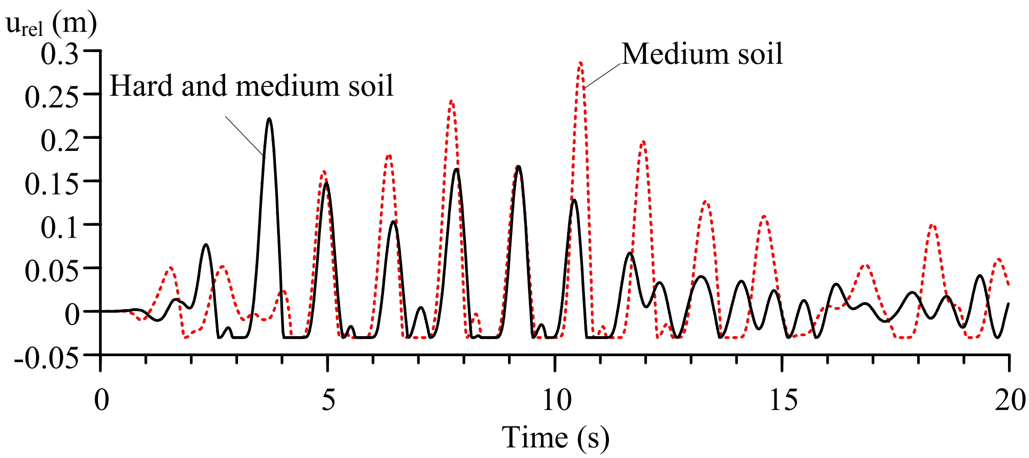

Figure 7. Non-uniform site effect on relative response development with SSI.

The influence of non-uniform site can also be seen in the different development of the bending moment M_{II} at the pier support of the right bridge (Fig. 8). The dotted and solid lines are M_{II} when non-uniform and uniform sites are considered, respectively. Pounding effect is incorporated. In the considered cases the non-uniform site causes smaller bending moment.

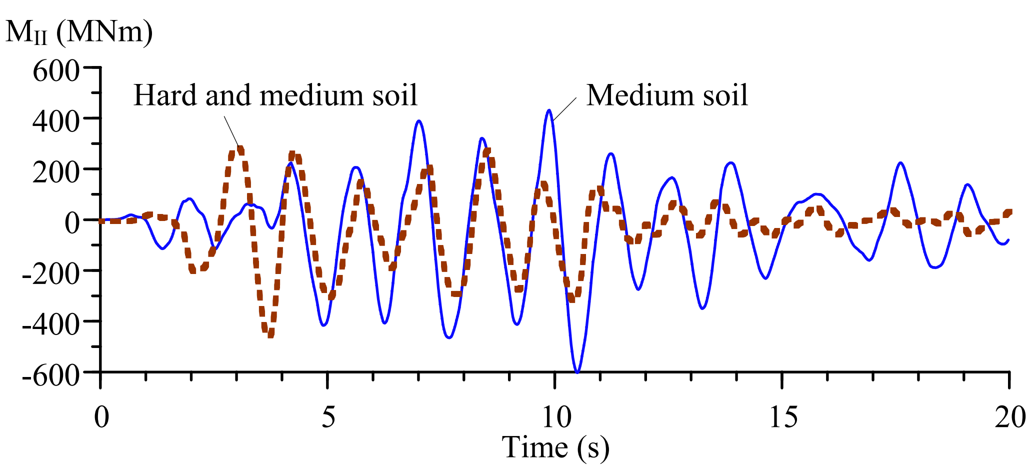

Figure 8. Non-uniform site effect on bending moment development with SSI.

Conclusions

Two adjacent bridge structures are considered to address the significance of the combined influence of structural, soil and ground excitation properties on seismic response of bridge structures.

The investigation reveals that the most commonly assumed fixed-base structures and uniform ground excitation will not be able to reflect a realistic damage potential due to pounding between adjacent bridge girders.

An adjustment of fundamental frequencies of adjacent structures with assumed fixed base cannot ensure a reduction of girder pounding potential.

A simultaneous consideration of non-uniform site, ground excitations, soil-structure interaction and the dynamic properties of bridge structures is required to enable a realistic prediction of pounding induced damage at bridge girder ends.

Acknowledgments

This research was supported by FRDF under the Grant No. 3622738/9572.

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