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Comments on bridge girder seating length under current design regulations

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This paper investigates the adequacy of the required seating length of bridge girders in the current Japanese bridge design code under strong earthquakes. The seating length was developed without considering the effect of soil-structure interaction (SSI). In this study, spatially varying earthquake ground motion time histories are simulated corresponding to the Japanese design spectrum, and any two of them are compatible with an empirical coherency loss function. Two bridge structures on half space are considered. The study reveals that although the Japanese design specification is currently the most advanced and strictest regulation and implicitly includes the effect of spatial variation of ground motions, the required seating length may still not be adequate, especially when the neighbouring bridge structure is flexible and when both structures experience different SSI.

Key Words: *design code, relative displacement, spatially varying ground motion, soil-structure interaction, pounding*

1. Introduction

Damages to bridge structures are often related to the relative responses between the adjacent structures. Large closing relative movements of girders can cause pounding damages at the girder ends, and large opening relative movements of bridge decks can result in unseating of bridge girders. Even though in almost all seismic design regulations the minimum seating length of bridge girders is specified, collapse of bridge girders is still observed, like in the 1989 Loma Prieta earthquake, the 1994 Northridge earthquake, the 1995 Kobe earthquake, the 1999 Chi-Chi Taiwan earthquake and the 1999 Kocaeli and Düzce Turkey earthquake.

Relative responses occur not only because the adjacent bridge structures have different dynamic properties. They can also develop because the adjacent bridge structures experience unequal ground excitations. The supporting subsoil can additionally affect the overall vibrations of the bridge structures, and consequently their relative responses. How strong the spatial variation of the ground motions is depends on the distance, the soil properties between the piers of the adjacent bridge structures and the characteristics of the propagating waves.

Despite many efforts spent on developing and revising design codes most of the regulations still have an empirical nature. Many researches on pounding responses have been published in past two decades. At the last world conference on earthquake engineering in Vancouver alone more than 50 works were presented. In most of the investigations, e.g. Andrawes and DesRoches[1)], Izuno et al.[2)], Tirasit and Kawashima[3)], uniform ground excitations and structures with fixed base were assumed. Works considering non-uniform ground motions were often limited to phase-delay effect, e.g. Zhu et al.[4)]. Investigations considering spatially varying ground excitations and SSI were rare. The soil effect was often represented by frequency-independent soil stiffness, e.g. Wang et al.[5)] , Oshima et al.[6)] and Lou and Zerva[7)]. The simultaneous effect of spatial variation of ground excitations and subsoil was considered so far by the authors [8)-11)]. Investigations on the so-called relative displacement response spectra, the maximum relative displacement as a function of frequency ratio of the adjacent bridge structures, are still limited. Jeng and Kasai[12)] proposed an approach that allows a consideration of the influence of time lag of the ground excitations. Ruangrassamee and Kawashima[13)] performed extensive studies on various influence factors, e.g. mass ratio and gap size. However, the studies so far did not include the coherence loss of the ground motions and SSI effect, which are included in this paper.

In this study some of current American, Taiwan and Japanese design regulations, i.e. AASHTO[14)], CALTRANS[15)], JRA[16)], concerning the required seating length for bridge girders are also considered. In all existing design regulations up to now a rational consideration of the effect of pounding, non-uniform ground motions and SSI is not implemented. To illustrate the consequence of these factors the most recent Japanese design code is evaluated, and the required girder seating length considering the simultaneous influence of SSI, girder pounding and spatial variation of ground excitations is discussed.

2. Current design regulations

Eqs. (1)-(4) define the minimum seating length $S_{E,\,min}$ for bridge girders in metre according to current American codes AASHTO[14], CALTRANS[15], Taiwan specification and Japanese design regulation JRA[16], respectively. They are similar. l and H are respectively the effective span to the adjacent expansion joint and the average height of the substructure in metre. S_k is the skew of the bridge support in degree measured from the line normal to the span.

The Japanese specification is currently probably the most advanced regulation. The relative displacement u_{rel} in Eq. (5) considers the frequency ratio f_1/f_2 of the adjacent structures, and also the effect of the relative ground displacement u_G, which depends on the soil strain ε_G and the distance L between the substructures in metre (Eq. (6)). For hard, medium and soft soil ε_G has the value of 0.0025, 0.00375 and 0.005, respectively. In the specification u_{rel} is defined for adjacent reference structures with certain fundamental frequencies without a consideration of the effect of pounding and unseating prevention measures. The larger seating length obtained from Eqs. (4) and (5) should be used.

CALTRANS:

$$S_{E,\,min} = (0.3 + 0.0025\ l + 0.01 H)\,(1 + \frac{S_k^2}{8000}) \tag{1}$$

AASHTO:

$$S_{E,\,min} = (0.2 + 0.0017\ l + 0.0067\ H)\,(1 + \frac{S_k^2}{8000}) \tag{2}$$

Taiwan:

$$S_{E,\,min} = 0.7 + 0.0025\ l + 0.01 H \tag{3}$$

Japan:

$$S_{E,\,min} = 0.7 + 0.005\ l \tag{4}$$

$$S_E = u_{rel} + u_G \geq S_{E,\,min} \tag{5}$$

$$u_G = \varepsilon_G L \tag{6}$$

3. Applied numerical approaches

3.1 Ground excitation simulation

For numerical evaluation of the relative displacements of adjacent bridge structures spatially varying ground motions based on the Japanese design spectra[17] for hard, medium and soft soil conditions (Fig. 1) are simulated stochastically. In order not to underestimate the effect due to uncertainties involved in the ground motion spatial variations and also to consider a possible large pier distance, a relatively large distance of 100 m between the pier footings is chosen. In the simulation empirical coherency loss functions are employed. These functions are derived by Hao[18] and Hao et al.[19] from about 1000 strong motion time histories in the dense seismograph SMART-1 arrays. The empirical coherency loss functions are given by

$$\left|\gamma(f, d^l_{ij}, d^t_{ij})\right| = \exp(-\beta_1 d^l_{ij} - \beta_2 d^t_{ij}) \\ \exp\{-[\alpha_1(f)\sqrt{d^l_{ij}} + \alpha_2(f)\sqrt{d^t_{ij}}\,]\, f^2\} \tag{7}$$

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

$$\alpha_i(f) = \frac{a_i}{f} + b_i f + c_i, \quad i = 1, 2 \tag{8}$$

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 during the event 45 is employed. Ground motions during the event 45 are considered as highly correlated. Without losing generality intermediately cross-correlated ground motions with an apparent velocity c_a of 500 m/s are simulated using following modified empirical coefficients:

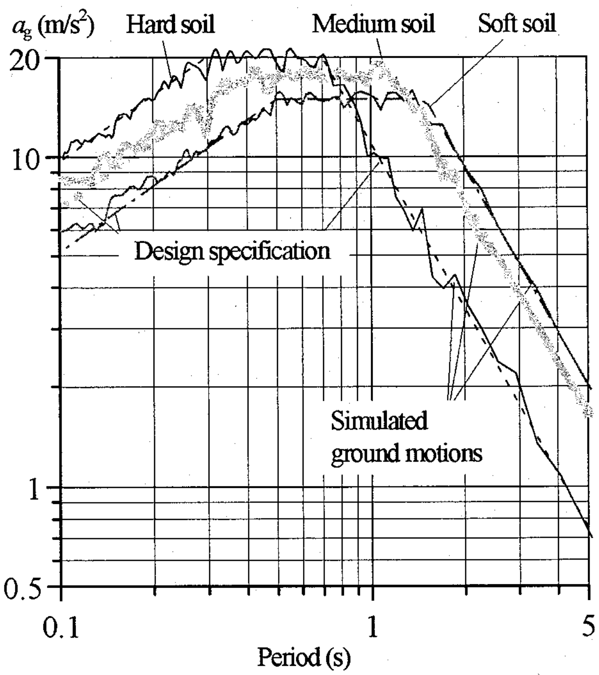

Fig. 1. Japanese design spectra[17] and spectra of the simulated ground motions

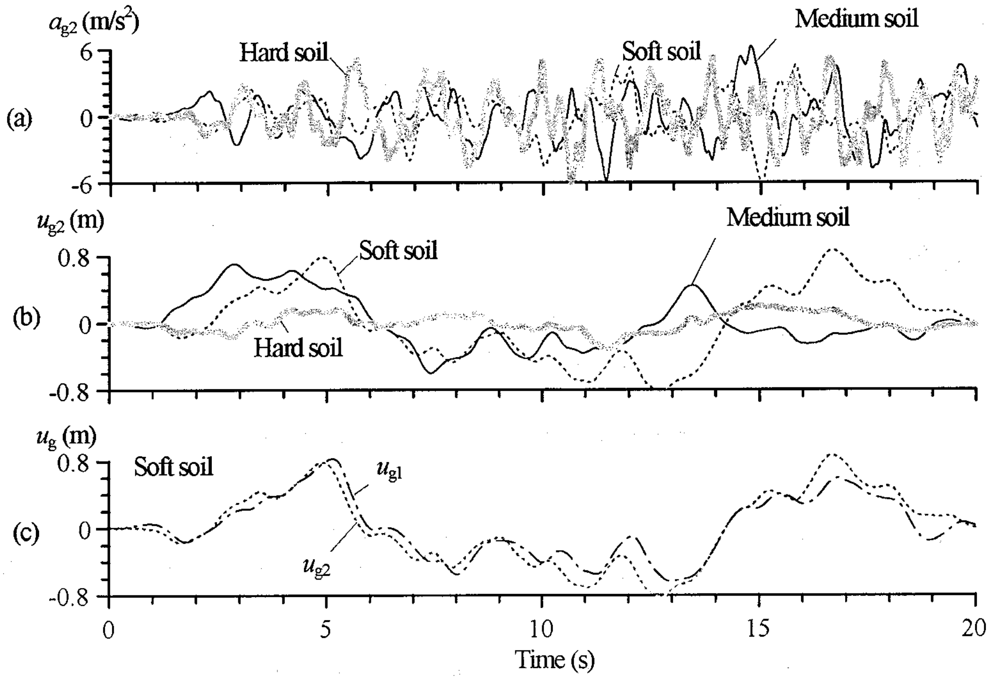

Fig. 2(a)-(c) Ground excitations. (a) Ground acceleration a_{g2}(t) and (b) displacement u_{g2}(t) at the left pier support, and (c) spatially varying ground displacements u_{g1}(t) and u_{g2}(t) at right and left pier supports

$\beta_1 = 3.7\times10^{-4}$, $\beta_2 = 2.24\times10^{-4}$, $a_1 = 1.19\times10^{-2}$, $b_1 = -1.1811\times10^{-5}$, $c_1 = 1.177\times10^{-4}$, $a_2 = 1.721\times10^{-2}$, $b_2 = -7.583\times10^{-6}$, $c_2 = -1.905\times10^{-4}$.

Figs. 2(a) and 2(b) show the simulated ground accelerations a_{g2}(t) and their corresponding displacements u_{g2}(t) at the left pier support, respectively (see also Fig. 3). Although the peak ground accelerations (PGAs) for hard, medium and soft soil conditions are almost the same, about 6 m/s^2, the peak ground displacements (PGDs) are very different because of the different frequency content of ground motions for the different soil conditions. With increasing soil stiffness the PGD decreases. This difference in the ground displacement amplitude can have strong influence on the relative displacement, because it depends on dynamic and also quasi-static responses of the adjacent bridge structures. Fig. 2(c) shows the ground displacements u_{g1}(t) and u_{g2}(t) at the right and left pier supports. Their non-uniformity reflects the quasi-static contribution of the ground motions to the total relative displacement.

3.2 Soil-structure modelling

Fig. 3 shows the considered adjacent bridge structures. For simplicity the girder displacement is described by a single-degree-of-freedom (SDOF). l, L_1 and L_2 are all in metres. Each surface foundation has the dimension of 9.0 m x 9.0 m. The soft and medium subsoil are assumed to be a half-space with shear wave velocities c_s of 100 m/s and 200 m/s, respectively. The soil has the density ρ of 2000 kg/m^3 and the Poisson's ratio ν of 0.33. For the hard soil condition the ground is assumed to be rigid. To limit the influence factors it is assumed that only the radiation damping due to wave propagation in the half-space is considered. In this study it is assumed that the right and left bridge piers experience the ground motions u_{g1}(t) and u_{g2}(t), respectively. In the case of uniform ground excitation both bridge piers are excited by u_{g1}(t). Each bridge structure has the damping ratio of 5 %.

In the numerical analyses the bridge structures and their foundations are described by a finite element method, and the subsoil by a boundary element method. By coupling the two subsystems the dynamic stiffness of the whole system can be obtained. The dynamic stiffness of the structural members can be obtained by solving the equation of motion analytically. Continuous-mass model formulation is applied. By adding the stiffness of each structural member the dynamic stiffness $\tilde{\boldsymbol{K}}^{b}$ of the bridge structures can be defined. The tilde indicates a vector or matrix in the Laplace domain. Details of the formulation are given in the reference (Chouw[20]).

The dynamic stiffness of the subsoil is obtained by transforming the wave equation into the Laplace domain. By using the full-space fundamental solution and by assuming the distribution of displacements and tractions along the boundaries the relationship between tractions and displacements at the contact areas between the foundations and subsoil are defined. An introduction of the element area leads then to the dynamic stiffness of the subsoil at the contact area. The governing equation for each bridge structure and subsoil is obtained by coupling the two subsystems.

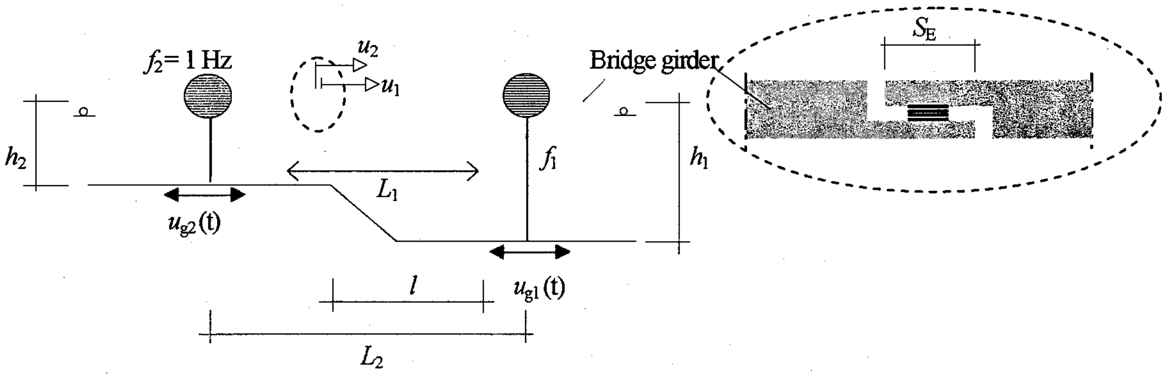

Fig. 3. Adjacent bridge structures with their simplified model

$$\begin{bmatrix} \tilde{K}_{bb}^{bn} & \tilde{K}_{bc}^{bn} \\ \tilde{K}_{cb}^{bn} & \tilde{K}_{cc}^{bn} + \tilde{K}_{cc}^{sn} \end{bmatrix} \begin{Bmatrix} \tilde{u}_{b}^{bn} \\ \tilde{u}_{c}^{bn} \end{Bmatrix} = \begin{Bmatrix} \tilde{P}_{b}^{bn} \\ \tilde{P}_{c}^{bn} \end{Bmatrix} \tag{9}$$

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

$$\{\tilde{P}(s)\} = \int_0^\infty \{P(t)\} e^{-st} dt \tag{10}$$

where $s = \delta + i\omega$ is the Laplace parameter and $i = \sqrt{-1}$, the linear response $\{\tilde{u}(s)\}$ of both bridge structures can then be defined using Eq. (9). A transformation of the results back to the time domain (Eq. (11)) gives the time history of the structural responses, and the girder relative displacement u_{rel} (t) = u_1(t) – u_2(t) can be calculated. In this study only traditional expansion joint is considered, and a girder gap of 5 cm is assumed.

$$\{u(t)\} = \frac{1}{2\pi i} \int_{\delta - i\omega}^{\delta + i\omega} \{\tilde{u}(s)\} e^{st} ds \tag{11}$$

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. Since now the two structures are in contact, the condensed stiffness has to be added to the stiffness of the uncondensed subsystem. 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 the first 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. The corrective term is obtained from Eq. (9) 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 SSI approach are described in the reference (Chouw[21)]).

4. Effect of spatially varying ground excitation and SSI

Figs. 4(a) and (c) show respectively the girder relative displacements due to soft-soil ground motions without and with the SSI effect. The corresponding displacements including the quasi-static responses owing to non-uniform ground displacements are displayed in Figs. 4(b) and (d). Without pounding effect the largest required seating length of the structures with assumed fixed bases is 40.23 cm (Fig. 4(a) at 10.44 s). The SSI amplifies the maximum required seating length to 45.33 cm (Fig. 4(c) at 16.64 s). If the influence of the spatial variation of the ground displacements is considered as well, the required seating lengths for bridge structures without and with SSI effect are 64.64 cm (Fig. 4(b) at 16.5 s) and 80.18 cm (Fig. 4(d) at 16.62 s), respectively. In all considered cases poundings reduce the required seating length.

Most current codes estimate the relative displacement response of adjacent bridge structures without considering the effects of SSI, pounding and earthquake ground motion spatial variations. In these code estimations the relative response will indeed be zero, if the two bridge structures have the same vibration frequency. However, the relative displacement, or required seating length will not be zero, if the simultaneous influence of spatially varying ground motions, SSI and pounding is considered, as the results shown in Fig. 4. This observation indicates that the current design recommendation should be reexamined.

Fig. 5 shows the other significant influence of SSI. While in the fixed-base case the structural response is mainly determined by the fundamental frequency of the structure, the response with SSI is additionally affected by the slenderness ratio of the bridge piers and foundation properties. Because of their different slenderness two neighbouring bridge structures -even with the same fixed-base fundamental frequency- will experience different SSI effect, and consequently relative

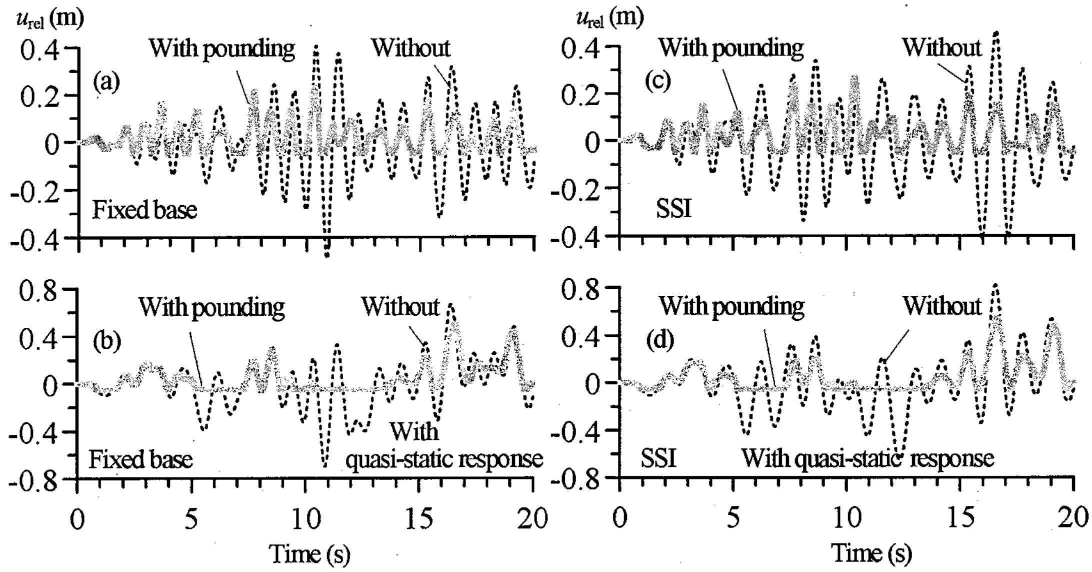

Fig. 4(a)-(d). Non-uniform ground motion effect on the relative displacement u_{rel} (t).
(a)-(b) Fixed-base structures, and (c)-(d) structures with subsoil. $f_1 = f_2 = 1$ Hz and $h_1 = h_2 = 9$ m

displacements between the girders occur. This influence is not considered in the current design regulations, either. To illustrate this effect two structures with the same fixed-base fundamental frequency of 1.0 Hz are considered. It is assumed that both structures experience the same soft-soil ground motions.

Since both structures have the same excitations and same fundamental frequency, there is no relative displacement between adjacent girders. This is true, however, only if the fixed-base structures are assumed, or when both structures have the same slenderness -if SSI is considered- (thick-grey line in

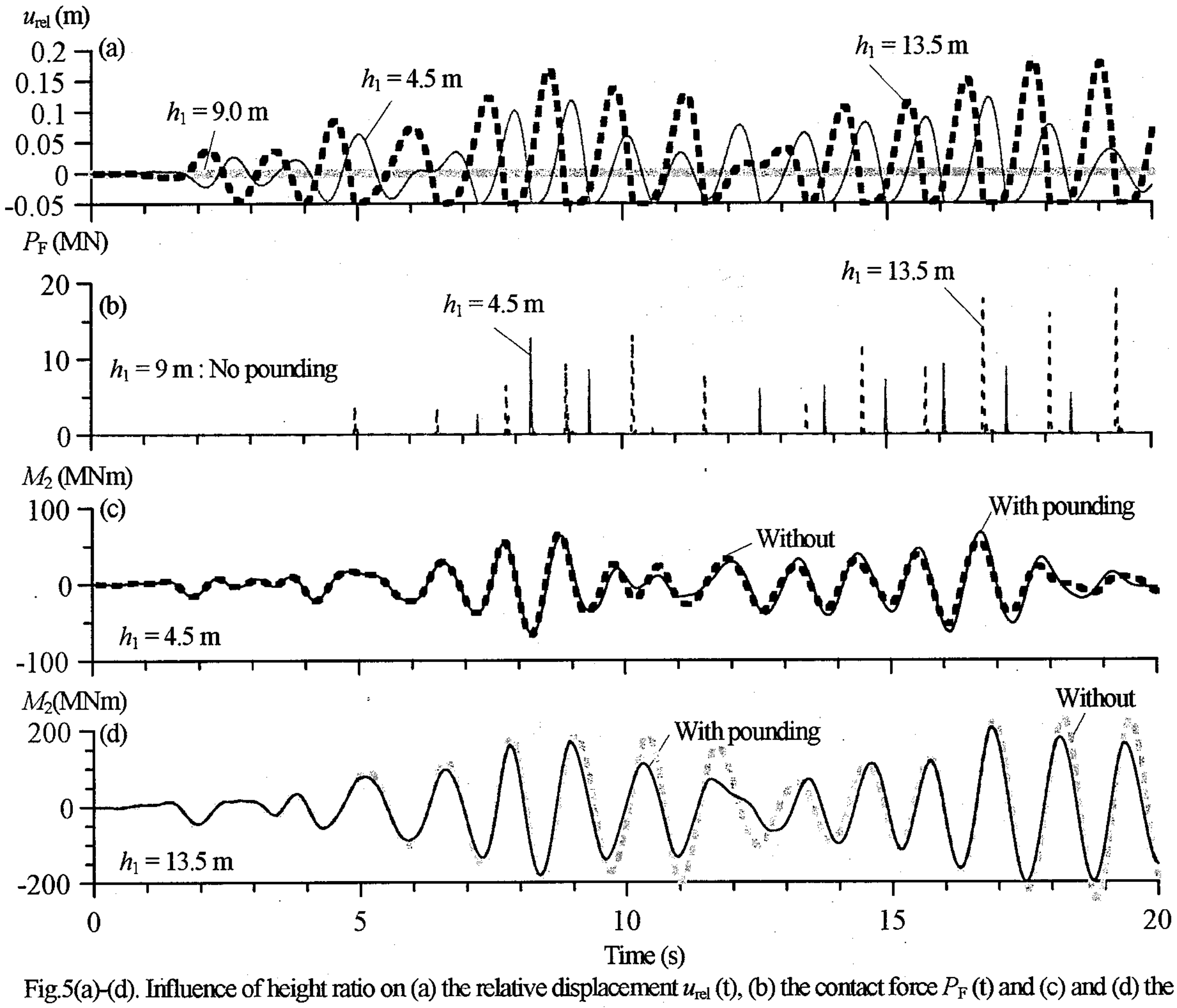

Fig.5(a)-(d). Influence of height ratio on (a) the relative displacement u_{rel} (t), (b) the contact force P_F (t) and (c) and (d) the bending moment M_2 (t) at the right pier support for $h_2 = 9$ m, $f_1 = f_2 = 1$ Hz and with soil-structure interaction

Fig. 5(a) for $h_1 = h_2 = 9$ m). This is because in the later case both structures experience the same SSI effect. If the heights of the bridge structures are not the same, they will have different SSI, therefore result in relative responses. While the overall vibration behaviour of a tall (slender) bridge structure is mainly determined by the rocking stiffness of the foundation-soil system, the behaviour of a low bridge structure is defined by the horizontal soil stiffness. Therefore bridge structures with different slenderness will not respond in phase, even though they have the same fixed-base fundamental frequency. In the considered case a height ratio h_1 / h_2 of 0.5 will cause a required seating length of 12.0 cm (thin-solid line at 16.96 s), while a height ratio of 1.5 will produce a necessary seating length of 17. 9 cm (dotted line at 17.8 s).

This influence of different SSI is often overlooked in structural engineering. Instead structures are almost always assumed fixed at rigid ground. The results show the significance of a consideration of SSI. With the commonly assumed fixed base pounding between adjacent girders does not occur as shown in Fig. 5(b). Consequently, girder damage potential due to pounding cannot be predicted properly.

The influence of different SSI can also be seen in the development of bending moment M_2(t) at the right bridge pier support in Figs. 5(c) and 5 (d). The solid and dotted lines are the bending moments with and without pounding effect, respectively. As expected a slender structure has larger bending moment. While in the case of height ratio h_1 / h_2 of 1.5 the bending moment development conforms to the common presumption that pounding will reduce the bending moment, in the case of h_1 / h_2 of 0.5 pounding can amplify the bending moment. The result shows that contrary to what is commonly presumed; pounding does not necessarily always reduce bending moments. Possible different interaction between the two structures with their supporting subsoil should therefore be considered.

5. Comparison

To compare with the values defined by the Japanese design regulation[16)] the mean values of the calculated relative displacements are normalized by the maximum displacement of the reference left bridge girder. For each soil type 30 sets of stochastically simulated ground motions are considered. Figs. 6(a) and 6(b) show the dimensionless relative displacement spectra due to uniform and non-uniform ground accelerations, respectively. Pounding, quasi-static responses and SSI are not considered. The values of the spectra represent then how much larger or smaller the opening relative displacement becomes in comparison with the maximum displacement of the reference left bridge girder displacement. While the frequency f_2 of the reference structure is kept constant, the frequency f_1 of the right structure is varied. The relative displacement spectrum of the Japanese design code for a reference structure with the fundamental frequency f_2 of 1 Hz is presented as a bold-grey line. It is obtained from the responses due to 63 strong ground motions of earthquakes in Japan with magnitude 6.5 or above. Since the Japanese design spectra (Fig. 1) used in this study for simulating the spatially varying ground motions are also based on the strong motions recorded in Japan, it is expected that the obtained relative displacement spectra and the design code values will correspond well.

Fig. 6(a) shows that in the lower frequency range the design values indeed correspond well with those due to medium-soil ground motions. At the frequency ratio $f_1 / f_2 = 1.0$ both the numerical results and the code value are equal to zero, since under the assumption of fixed-base structures and uniform ground excitation both structures respond in phase. In the frequency ratio above 1.0 the design values agree well with the responses due to hard-soil ground motions.

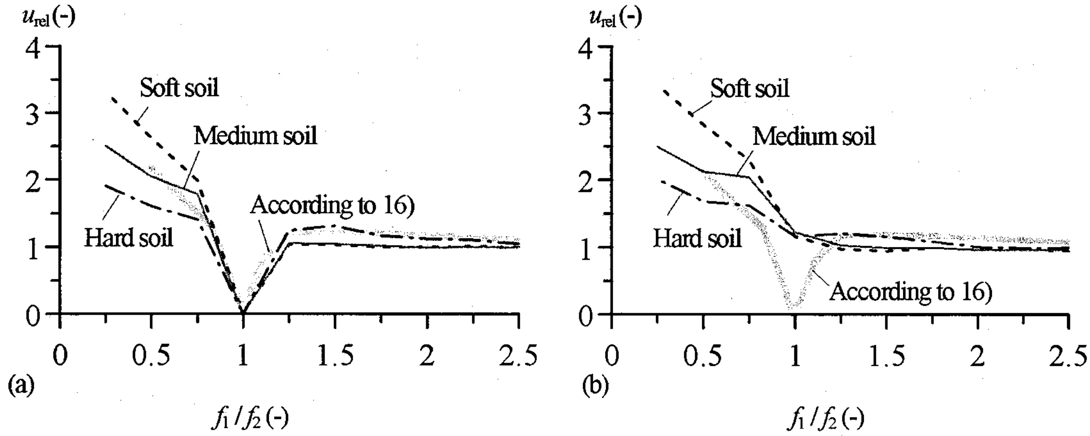

Fig. 6(a) and (b). Normalized relative displacement spectra due to
(a) uniform and (b) non-uniform ground excitation without quasi-static response and pounding effect
($h_1 = h_2 = 9$ m, $f_2 = 1$ Hz, fixed-base structures)

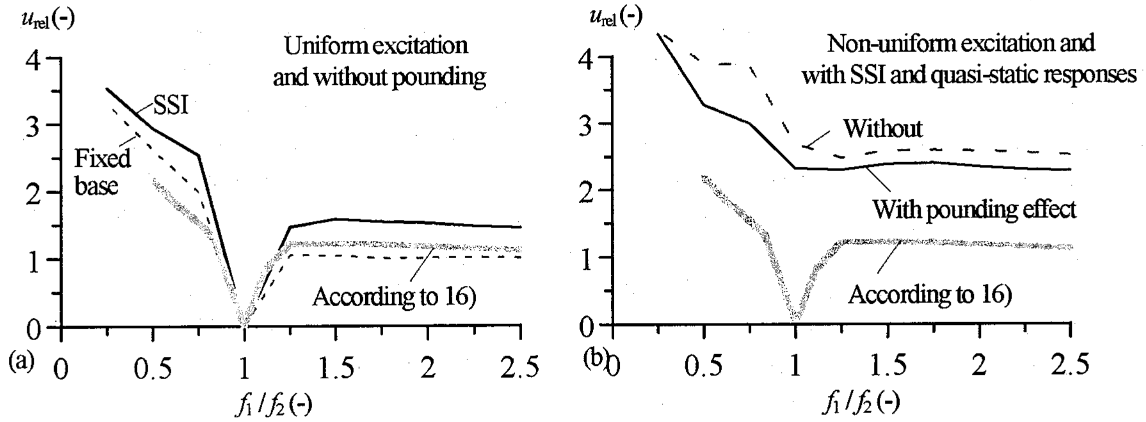

Fig. 7(a) and (b). Normalized relative displacement spectra due to (a) uniform and (b) non-uniform soft-soil ground excitation ($h_1 = h_2 = 9$ m, $f_2 = 1$ Hz, gap = 5 cm)

If non-uniform ground motions are considered, however, the design values clearly underestimate the required seating length, especially in the frequency ratio range around 1.0 where according to the recommendation of current design regulations similar or equal fundamental frequencies suppose to prevent bridge girder from unseating owing to their negligible relative displacements. If the soil is soft, the design values also underestimate the required seating length for the lower frequency ratios. The results show that the influence of spatial variation of the ground motions should be considered, especially when both structures have similar dynamic properties.

In Fig. 7(a) the design code values (bold-grey line) are displayed again with the required seating length (dotted line) under uniform soft-soil ground motions and fixed-base structure condition. An additional consideration of SSI effect (black-solid line) amplifies the required seating length in almost whole frequency ratio range. If the quasi-static responses due to spatially varying ground displacements are considered as well, the required seating length becomes even larger (compared solid-black line in Fig. 7(a) with dash line in Fig. 7(b)), not only at the frequency ratio range around 1.0 but in the whole frequency ratio range. Pounding reduces the spectrum values in almost the whole frequency range except when the neighbouring structure is much more flexible, e.g. at $f_1 / f_2 = 0.25$. However, they are still much larger than the design code values, because the relative displacement in the design regulation still does not include the soil strain effect. This effect is considered not until the seating length S_E is defined (Eq. 5).

The influence of the height ratio h_1 / h_2 can be seen in Fig. 8. $h_1 = 13.5$ m causes the largest spectrum values (thin dash line). $h_1 = 9$ m produces also similar spectrum values (bold-black

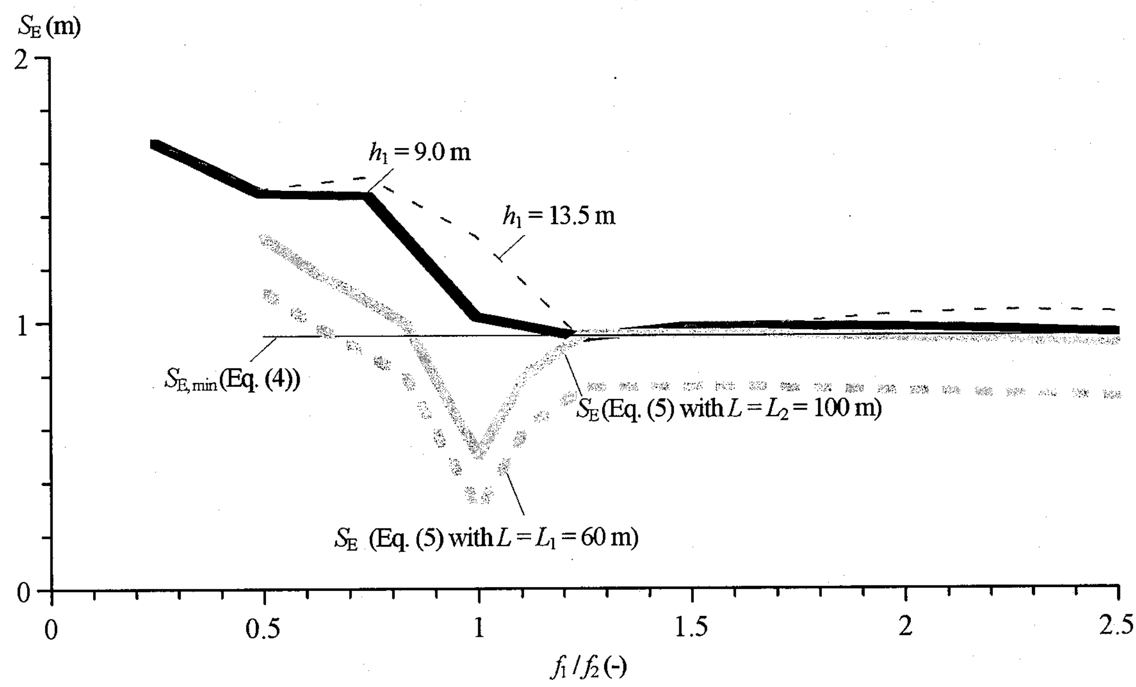

Fig. 8. Comparison between the required seating length according to Japanese design code and the mean value of the calculated seating length S_E with $h_1 = 9.0$ m and 13.5 m and without pounding effect

line). However, in the frequency ratio range between 0.5 and 1.2 and above 1.8, the slender adjacent bridge structure causes larger required seating length. These values are obtained with non-uniform soft-soil ground motions. The quasi-static responses and the SSI influence are also considered. However, in order to compare with the design code values pounding effect is excluded.

Assume the height H and the effective bridge span l to be 9 m and 50 m, respectively. Eqs. (1), (2), (3) and (4) give then the values $S_{E,\,min}$ of 0.515 m, 0.345 m, 0.915 m, and 0.95 m, respectively. ASSHTO provides the smallest required seating length, and the Japanese design code required the largest minimum seating length. In Fig. 8 the horizontal line represents the minimum values $S_{E,\,min}$ according to the Japanese design code. For an assumed distance between the substructures $L = L_1$ = 60 m the seating length according to Eq. (5) is plotted as bold-dash line. The influence of the relative ground displacement in the Japanese design values can be seen in the non-zero value at the frequency ratio of 1.0. All relative displacement values increase with a constant quasi-static contribution of 0.3 m. This consideration of the relative ground displacement reflects the most advanced knowledge considered in the current Japanese design regulations, and so far no other design regulations have included this significant influence factor. In reality, however, the influence of spatially varying ground displacements on the response of adjacent structures is not a constant. Hao[22] indicated that when the structures are stiff, the quasi-static response has a strong influence. If the subsoil effect is considered as well, the influence of the relative ground displacements becomes more complex, since the effects of the frequency ratio, possible non-uniform SSI and the frequency content of the ground motions are also involved simultaneously. The results show that in the whole frequency-ratio range the required seating length according to the current Japanese design specification is smaller than the calculated values. This is also the case, if a larger distance $L = L_2$ = 100 m is chosen.

6. Conclusions

In this study the adequacy of the minimum required seating length according to the current design regulations for preventing bridge girders from unseating during strong earthquakes is investigated. A distance between the adjacent pier supports is assumed to be 100 m.

The results of the considered cases reveal:

Current Japanese design specification is compared to other specifications the most advanced existing regulation, and it provides the largest required seating length. It includes the influence of the frequency ratio of the neighbouring structures, and also implicitly accounts for the effect of spatially varying ground displacements.

Although the current Japanese design specification already specifies the largest required seating length, it still underestimates the necessary seating length, especially when the frequency ratio is low and the adjacent structure is slender.

Significant factors not considered in the current design regulations are: non-uniform SSI influence and spatially varying ground motions, as well as pounding between adjacent girders.

Further investigations are deemed necessary, especially on the simultaneous influence of these factors including spatially varying ground conditions.

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